3MBSTTNK\EN\water_storage_tank_3mb.pdf
a1\PDF\a1.pdf
a2\PDF\a2.pdf
a3\PDF\a3.pdf
a4\PDF\a4.pdf
a5\PDF\a5.pdf
a6\PDF\a6.pdf
agkthesis\PDF\agkthesis.pdf
b1\PDF\b1.pdf
b2\PDF\b2.pdf
b3\PDF\b3.pdf
broc\PDF\broc.pdf
c1\PDF\c1.pdf
c2\PDF\c2.pdf
c3\PDF\c3.pdf
c4\PDF\c4.pdf
demthesis\PDF\demthesis.pdf
HYDRSTRM\PDF\HYDRSTRM.pdf
jpg\images.pdf
new_technology\PDF\new_technology.pdf
pdf_content\a1.pdf
pdf_content\a2.pdf
pdf_content\a3.pdf
pdf_content\a4.pdf
pdf_content\a5.pdf
pdf_content\a6.pdf
pdf_content\agkthesis.pdf
pdf_content\b1.pdf
pdf_content\b2.pdf
pdf_content\b3.pdf
pdf_content\broc.pdf
pdf_content\c1.pdf
pdf_content\c2.pdf
pdf_content\c3.pdf
pdf_content\c4.pdf
pdf_content\demthesis.pdf

pdf_content\HYDRSTRM.pdf
pdf_content\new_technology.pdf
pdf_content\rn_rwh04.pdf
pdf_content\rn_rwh05.pdf
pdf_content\ssrpr2.pdf
pdf_content\ssrpr3.pdf
pdf_content\ssrpr6.pdf
pdf_content\ssrpr8.pdf
pdf_content\thomast.pdf
pdf_content\tr_rwh01.pdf
pdf_content\tr_rwh02.pdf
pdf_content\tr_rwh04.pdf
pdf_content\tr_rwh06.pdf
pdf_content\tr_rwh07.pdf
pdf_content\tr_rwh09.pdf
pdf_content\tr_rwh10.pdf
pdf_content\tr10a.pdf
pdf_content\tr10b.pdf
pdf_content\tr11.pdf
pdf_content\tr12.pdf
pdf_content\tr13.pdf
pdf_content\tr14.pdf
pdf_content\tr15.pdf
pdf_content\tr16.pdf
pdf_content\tr20.pdf
pdf_content\tr21.pdf
pdf_content\tr22.pdf
pdf_content\tr23.pdf
pdf_content\tr24.pdf
pdf_content\tr25.pdf
pdf_content\tr26.pdf
pdf_content\tr27.pdf
pdf_content\tr28.pdf
pdf_content\tr29.pdf
pdf_content\tr30.pdf
pdf_content\tr33.pdf

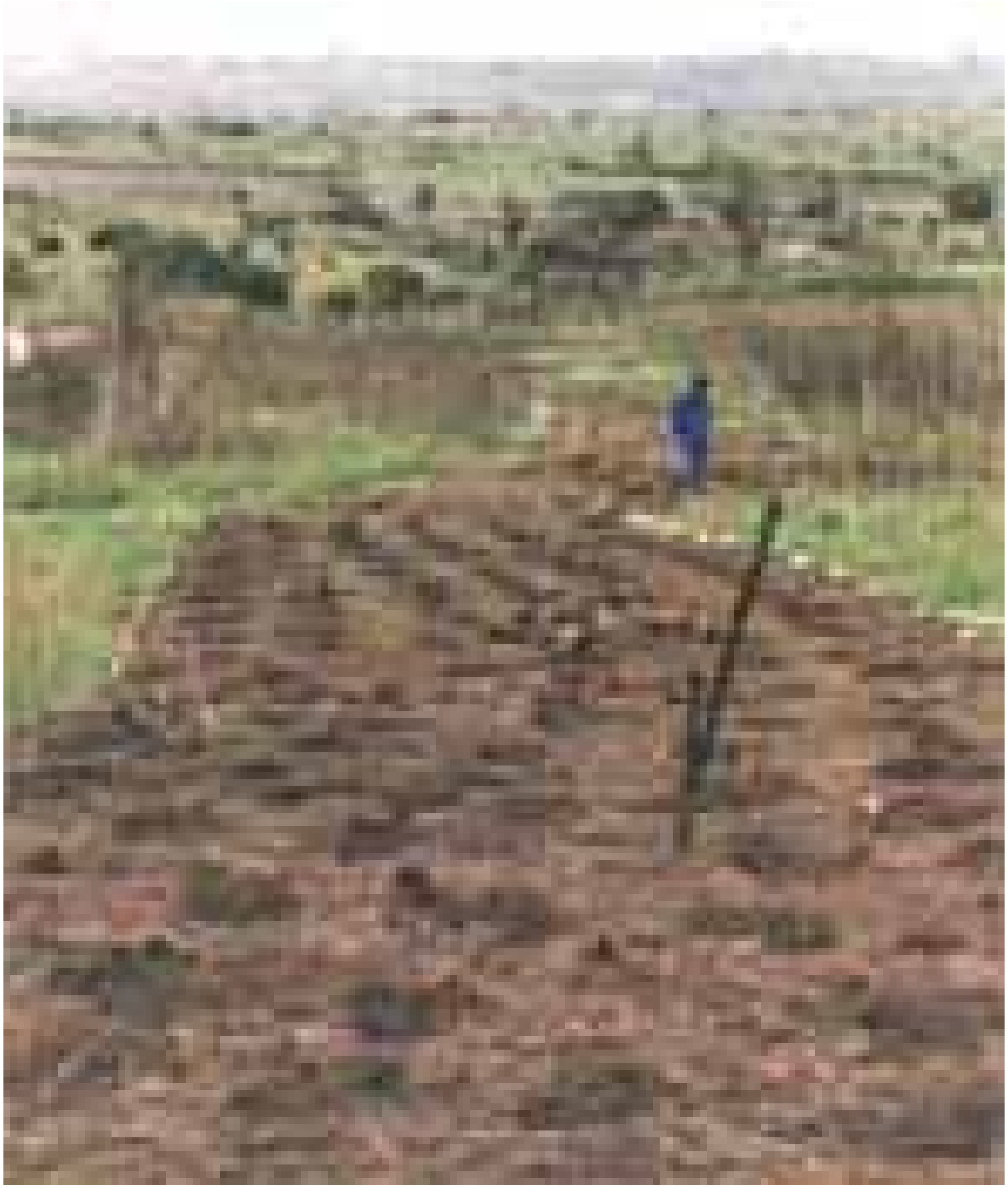
pdf_content\tr34.pdf
pdf_content\tr35.pdf
pdf_content\tr36.pdf
pdf_content\tr37.pdf
pdf_content\tr38.pdf
pdf_content\tr39.pdf
pdf_content\tr40.pdf
pdf_content\tr41.pdf
pdf_content\tr42.pdf
pdf_content\tr43.pdf
pdf_content\tr44.pdf
pdf_content\tr46.pdf
pdf_content\tr47.pdf
pdf_content\tr48.pdf
pdf_content\ugp001_turner.pdf
pdf_content\ugp011_constantine.pdf
pdf_content\ugp012_whitehead.pdf
pdf_content\ugp021_fernadez.pdf
pdf_content\ugp022_moeed.pdf
pdf_content\wp32.pdf
pdf_content\wp33.pdf
pdf_content\wp33_orig.pdf
pdf_content\wp34.pdf
pdf_content\wp35.pdf
pdf_content\wp38.pdf
pdf_content\wp39.pdf
pdf_content\wp40.pdf
pdf_content\wp41.pdf
pdf_content\wp42.pdf
pdf_content\wp43.pdf
pdf_content\wp44.pdf
pdf_content\wp45.pdf
pdf_content\wp46.pdf
pdf_content\wp52.pdf
pdf_content\wp53.pdf
pdf_content\wp54.pdf

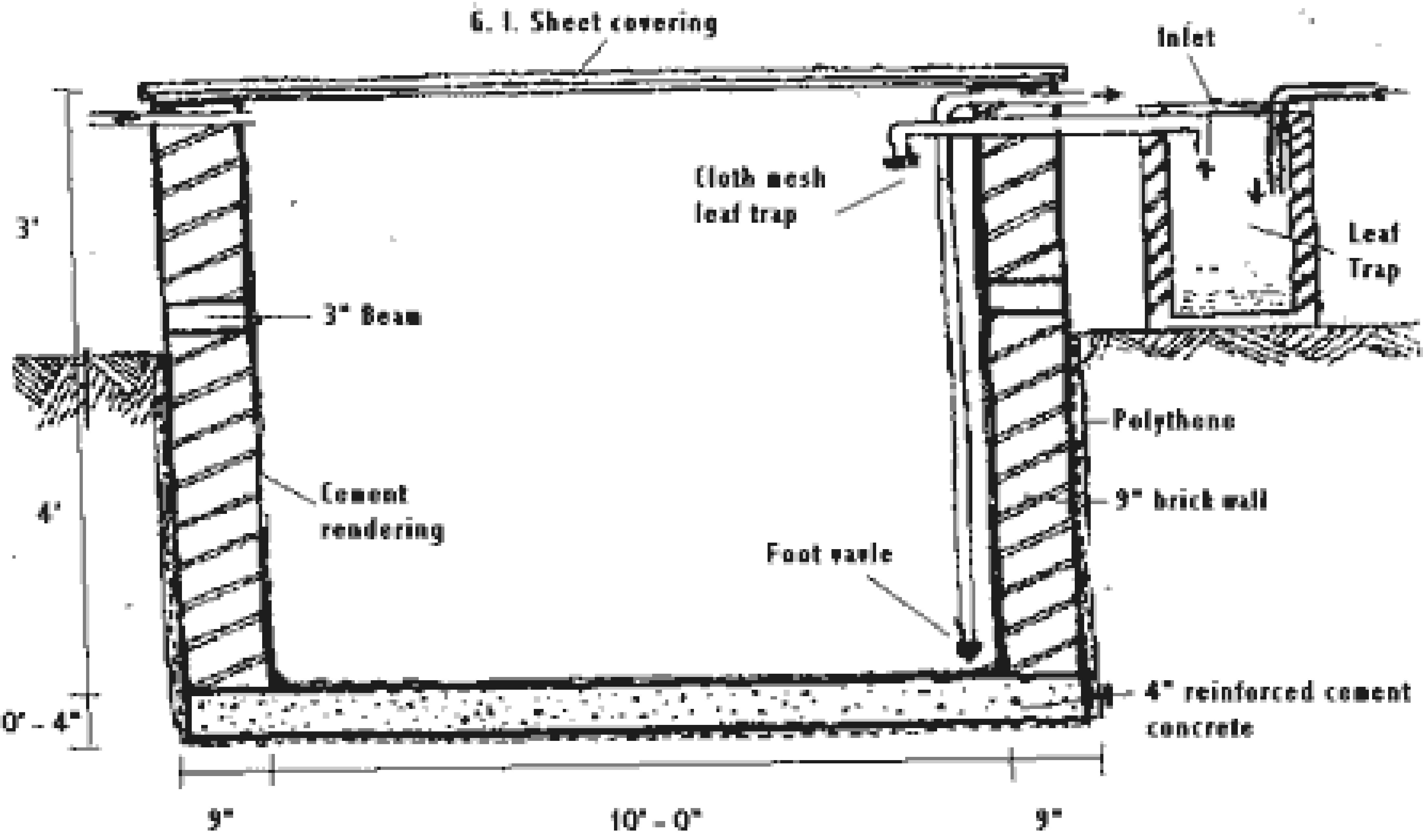
pdf_content\wp55.pdf
pdf_content\wp56.pdf
rn_rwh04\PDF\rn_rwh04.pdf
rn_rwh05\PDF\rn_rwh05.pdf
ssrpr2\PDF\ssrpr2.pdf
ssrpr3\PDF\ssrpr3.pdf
ssrpr6\PDF\ssrpr6.pdf
ssrpr8\PDF\ssrpr8.pdf
STUFF\a5\PDF\a5.pdf
thomast\PDF\thomast.pdf
tr10a\PDF\tr10a.pdf
tr10b\PDF\tr10b.pdf
tr11\PDF\tr11.pdf
tr12\PDF\tr12.pdf
tr13\PDF\tr13.pdf
tr14\PDF\tr14.pdf
tr15\PDF\tr15.pdf
tr16\PDF\tr16.pdf
tr20\PDF\tr20.pdf
tr21\PDF\tr21.pdf
tr22\PDF\tr22.pdf
tr23\PDF\tr23.pdf
tr24\PDF\tr24.pdf
tr25\PDF\tr25.pdf
tr26\PDF\tr26.pdf
tr27\PDF\tr27.pdf
tr28\PDF\tr28.pdf
tr29\PDF\tr29.pdf
tr30\PDF\tr30.pdf
tr33\PDF\tr33.pdf
tr34\PDF\tr34.pdf
tr35\PDF\tr35.pdf
tr36\PDF\tr36.pdf
tr37\PDF\tr37.pdf
tr38\PDF\tr38.pdf
tr39\PDF\tr39.pdf

tr40\PDF\tr40.pdf
tr41\PDF\tr41.pdf
tr42\PDF\tr42.pdf
tr43\PDF\tr43.pdf
tr44\PDF\tr44.pdf
tr46\PDF\tr46.pdf
tr47\PDF\tr47.pdf
tr48\PDF\tr48.pdf
tr_rwh01\PDF\tr_rwh01.pdf
tr_rwh02\PDF\tr_rwh02.pdf
tr_rwh04\PDF\tr_rwh04.pdf
tr_rwh06\PDF\tr_rwh06.pdf
tr_rwh07\PDF\tr_rwh07.pdf
tr_rwh09\PDF\tr_rwh09.pdf
tr_rwh10\PDF\tr_rwh10.pdf
ugp001_turner\PDF\ugp001_turner.pdf
ugp011_constantine\PDF\ugp011_constantine.pdf
ugp012_whitehead\PDF\ugp012_whitehead.pdf
ugp021_fernadez\PDF\ugp021_fernadez.pdf
ugp022_moeed\PDF\ugp022_moeed.pdf
wp32\PDF\wp32.pdf
wp33\PDF\wp33.pdf
wp33\PDF\wp33_orig.pdf
wp34\PDF\wp34.pdf
wp35\PDF\wp35.pdf
wp38\PDF\wp38.pdf
wp39\PDF\wp39.pdf
wp40\PDF\wp40.pdf
wp41\PDF\wp41.pdf
wp42\PDF\wp42.pdf
wp43\PDF\wp43.pdf
wp44\PDF\wp44.pdf
wp45\PDF\wp45.pdf
wp46\PDF\wp46.pdf
wp52\PDF\wp52.pdf
wp53\PDF\wp53.pdf

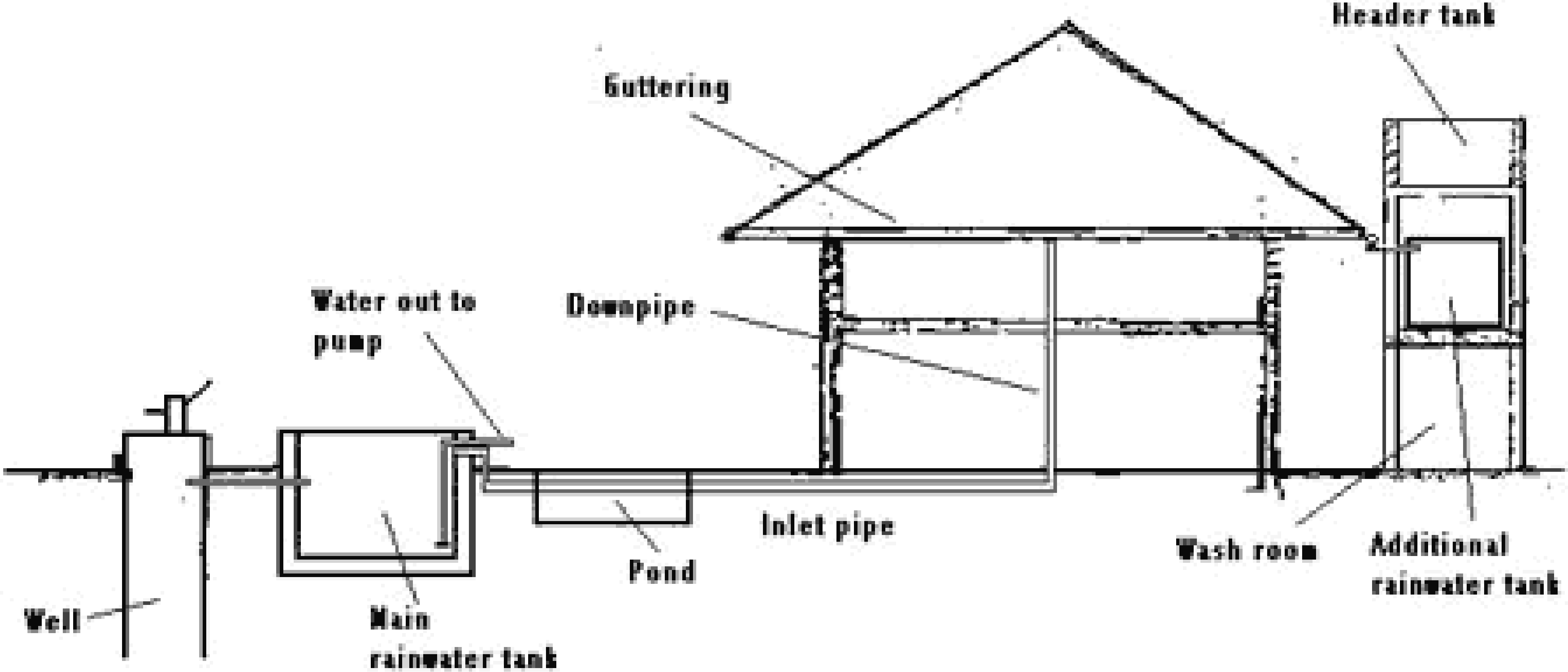
wp54\PDF\wp54.pdf
wp55\PDF\wp55.pdf
wp56\PDF\wp56.pdf

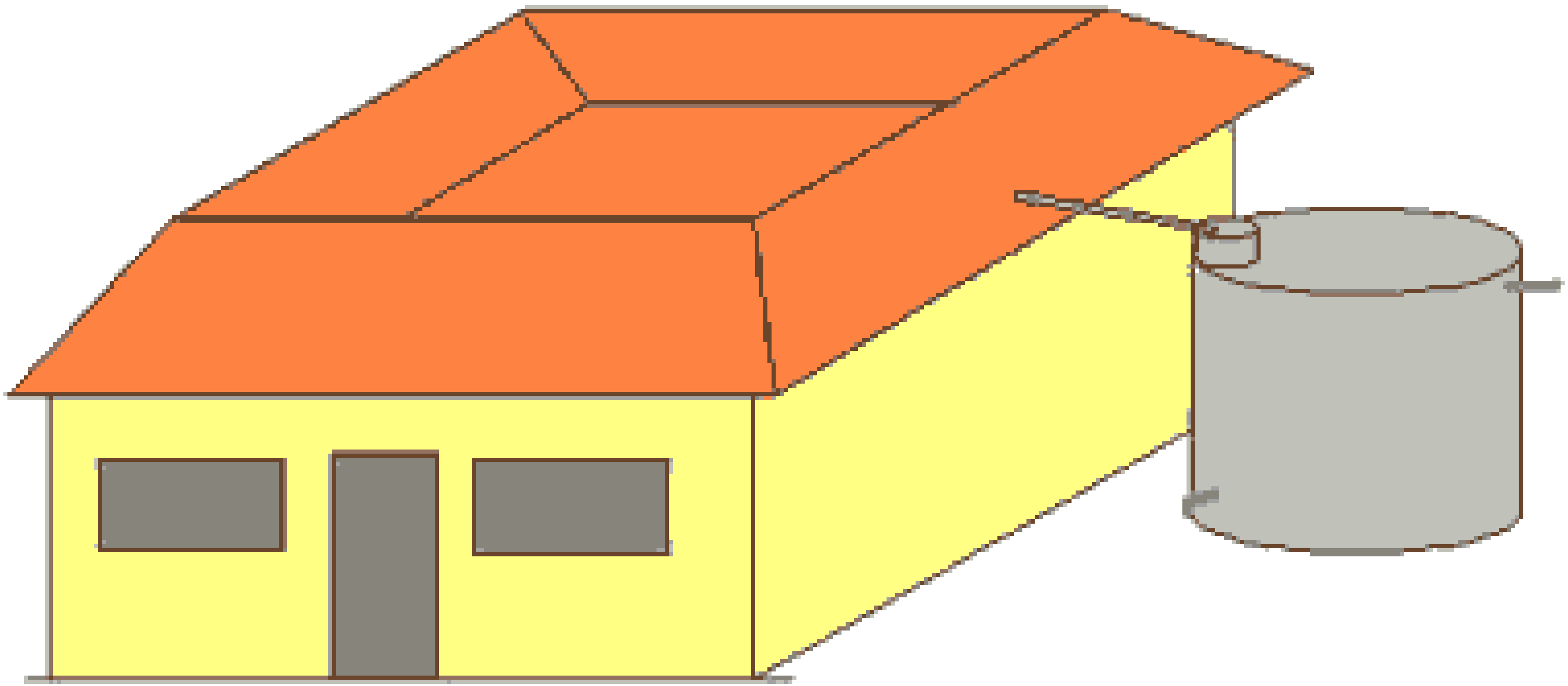

United Nations
Operation in Mozambique

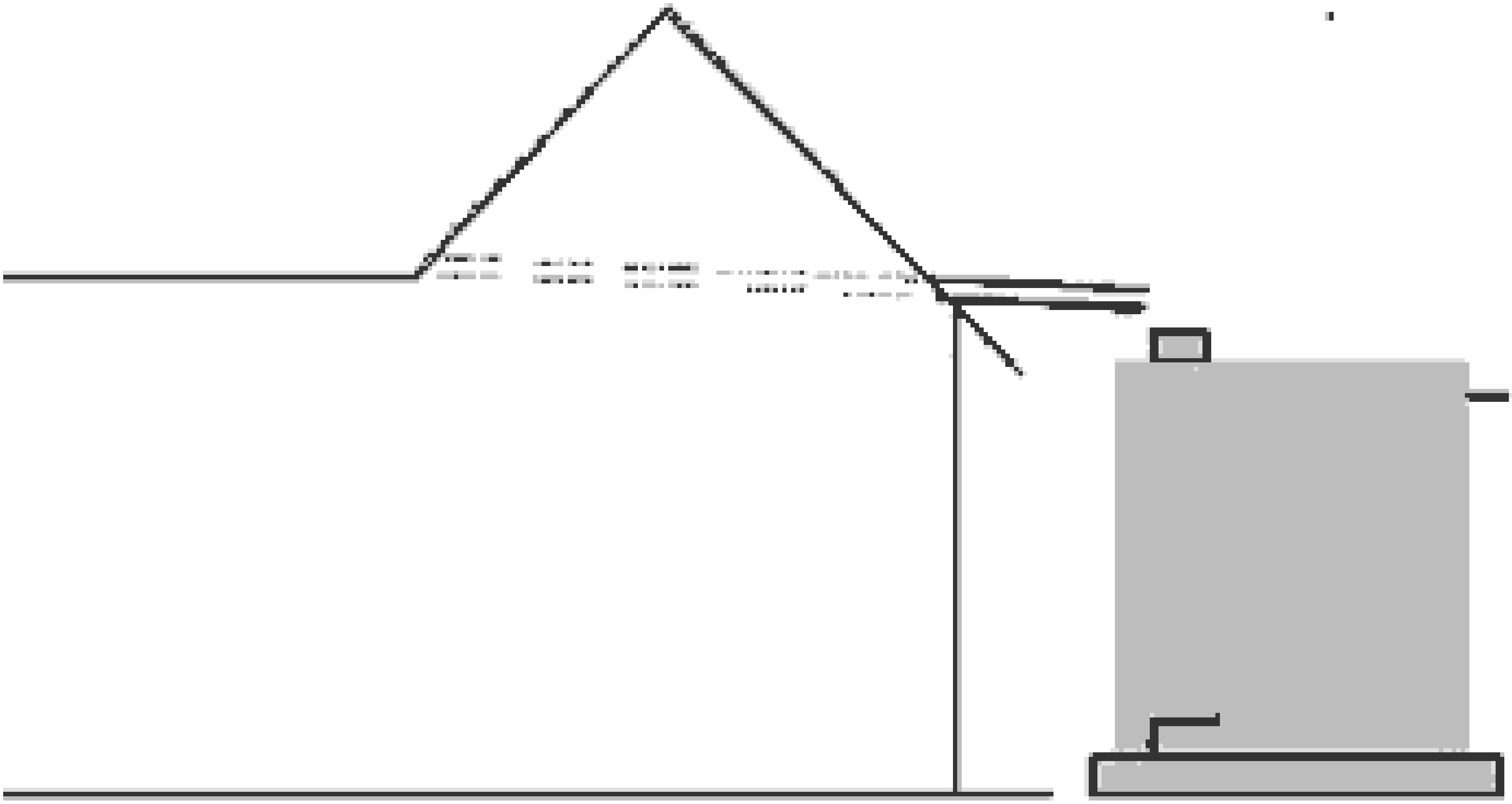


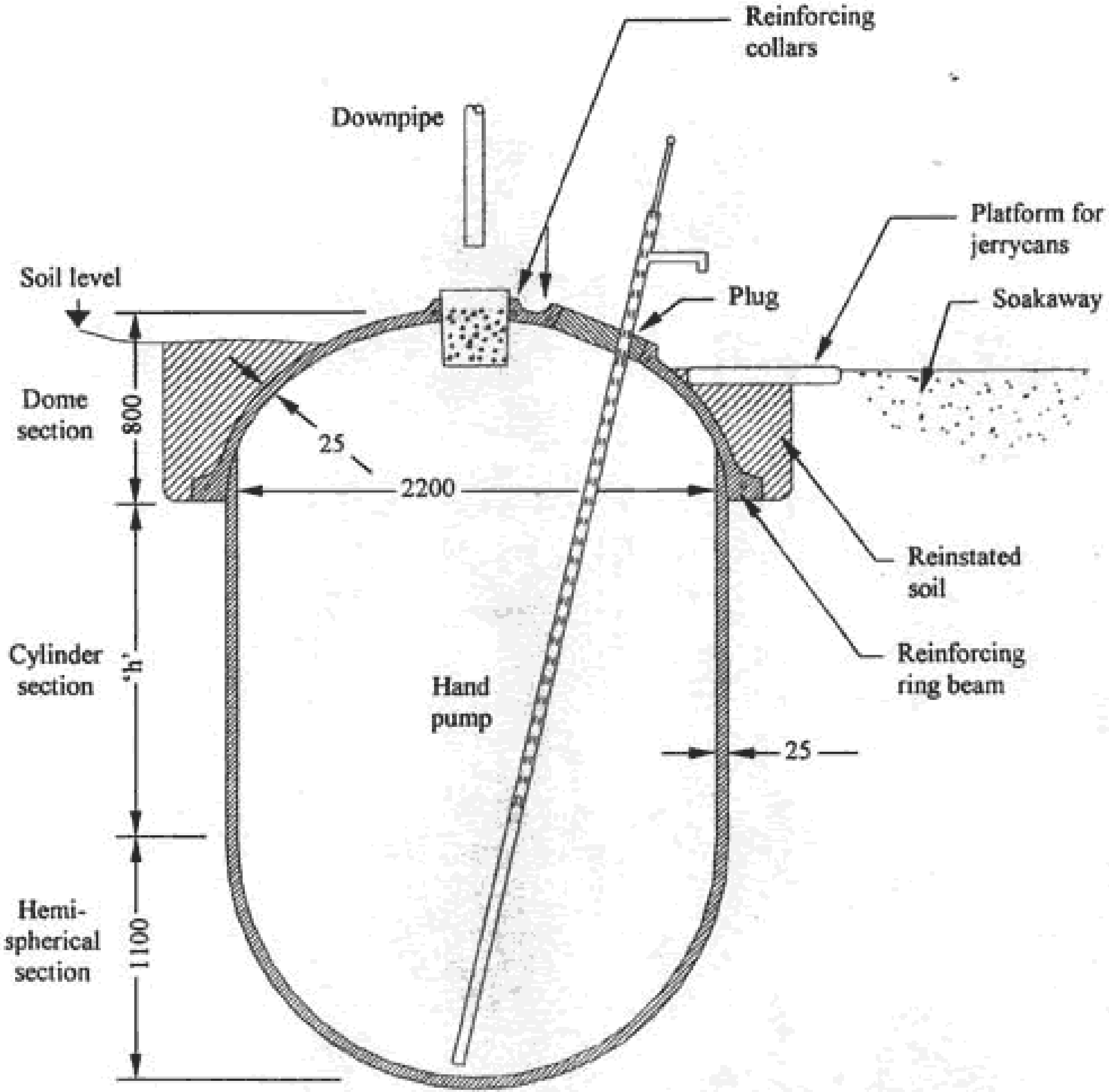


G. I. Sheet covering
Inlet
Cloth mesh
leaf trap
Leaf
Trap
3'
3" Beam
4'
Cement
rendering
Polythene
9" brick wall
Foot vaule
0' - 4"
4" reinforced cement
concrete
9"
10' - 0"
9"

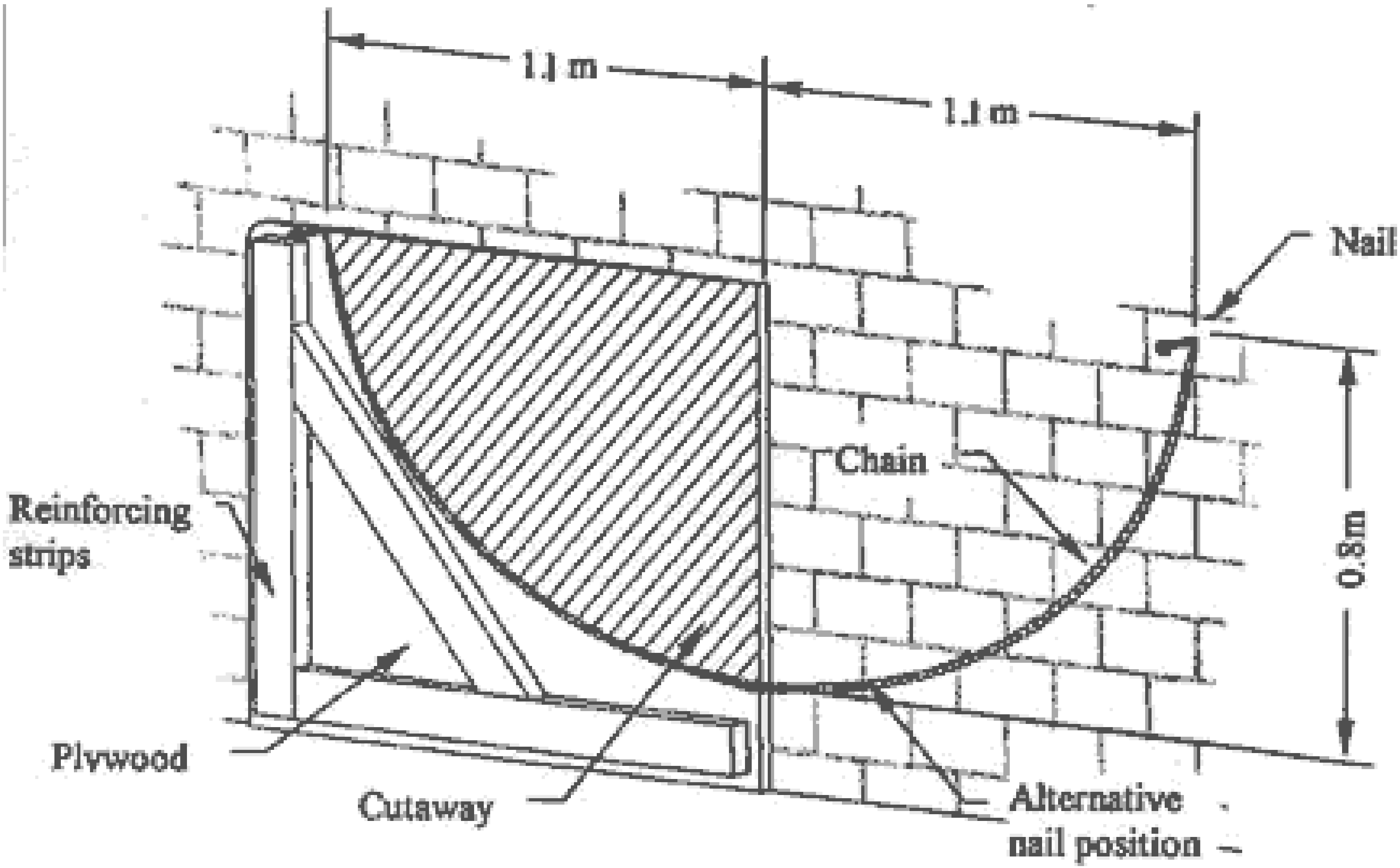

Header tank
Guttering
Water out to pump
Downpipe
Well
Main rainwater tank
Pond
Inlet pipe
Wash room
Additional rainwater tank



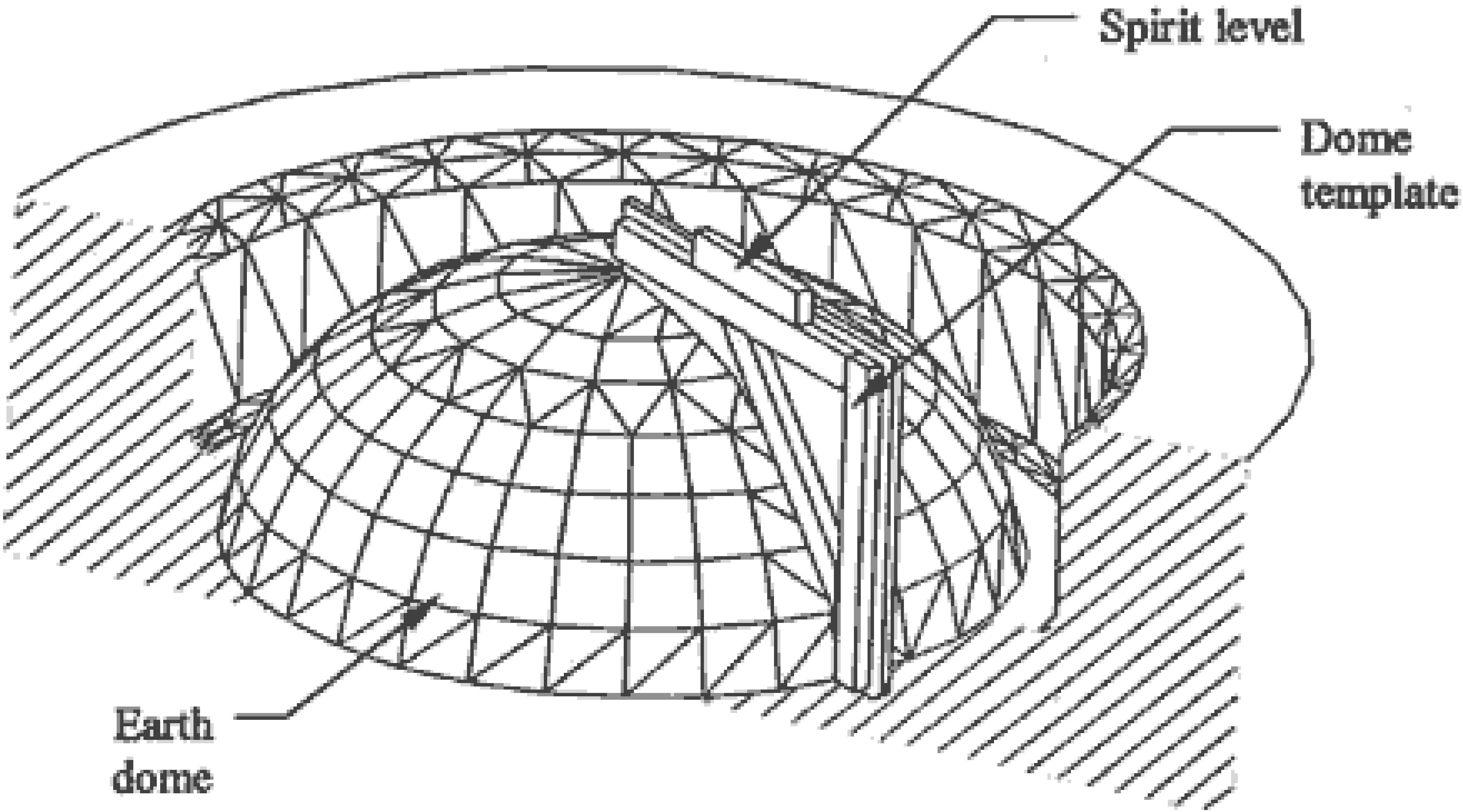


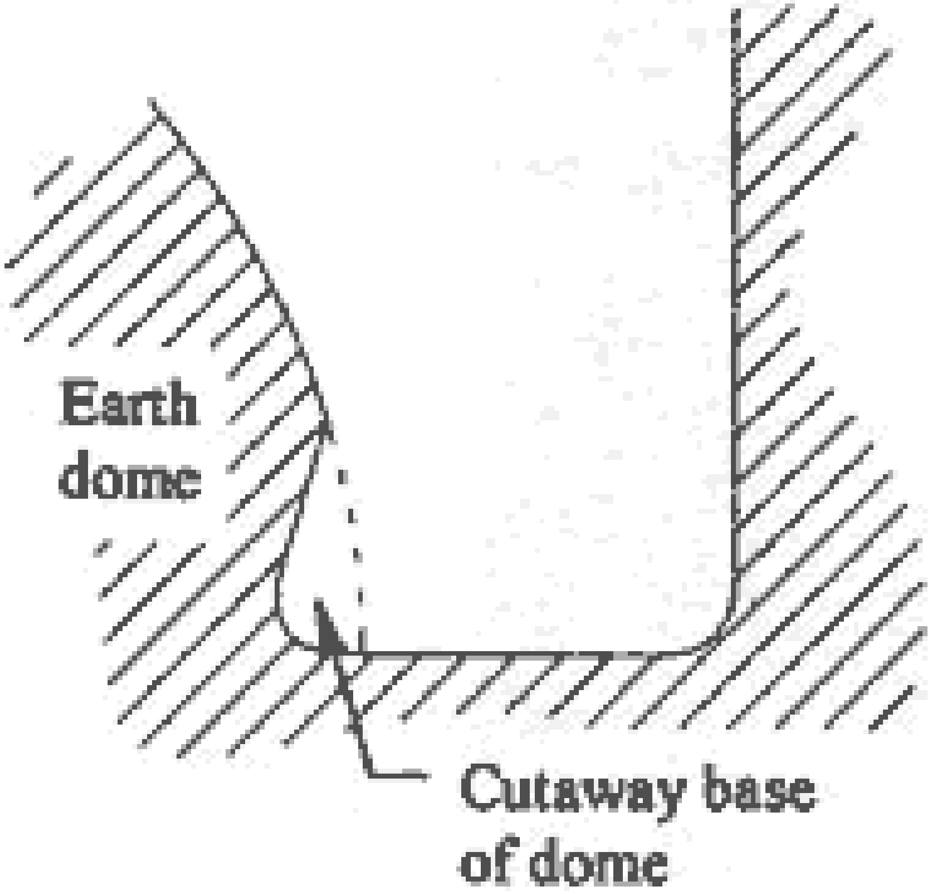


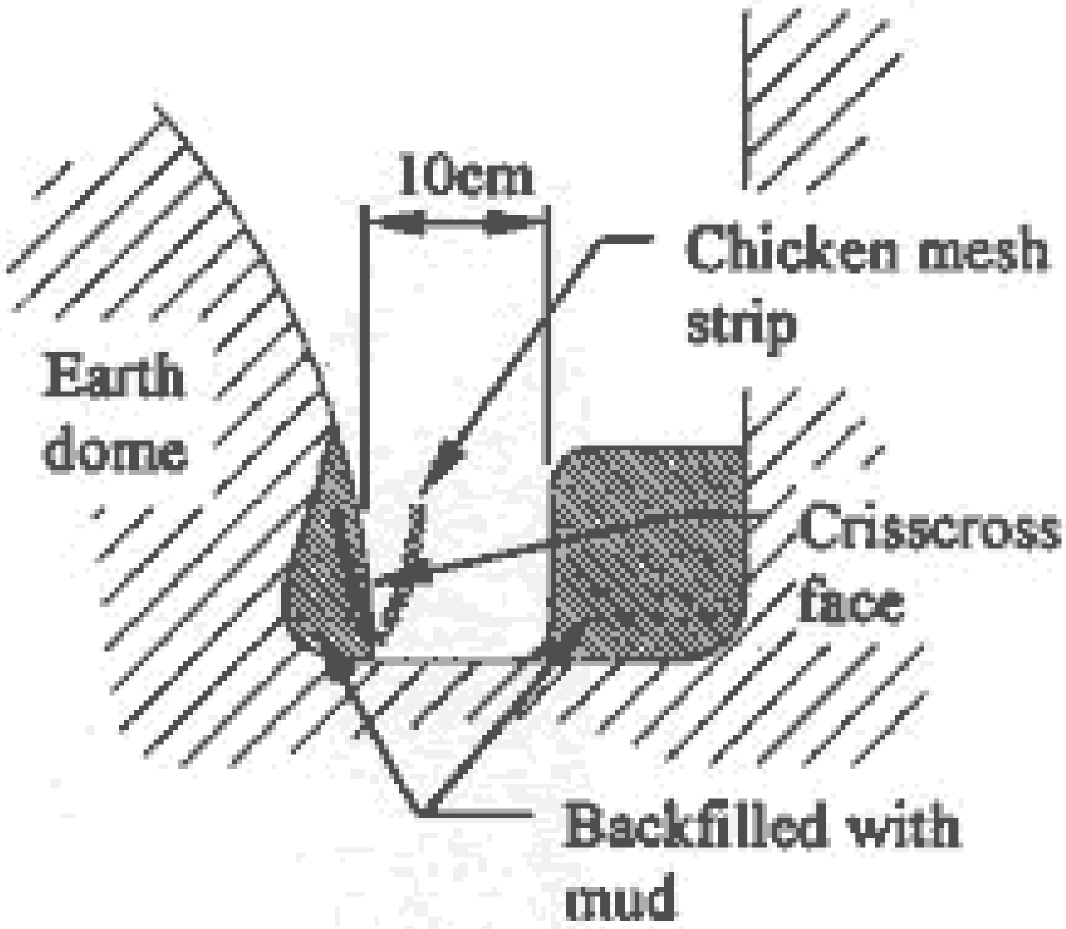
Reinforcing
collars
Downpipe
Platform for
jerrycans
Soil level
Plug
Soakaway
Dome
section
800
25
2200
Reinstated
soil
Cylinder
section
'h'
Hand
pump
Reinforcing
ring beam
25
Hemi-
spherical
section
1100

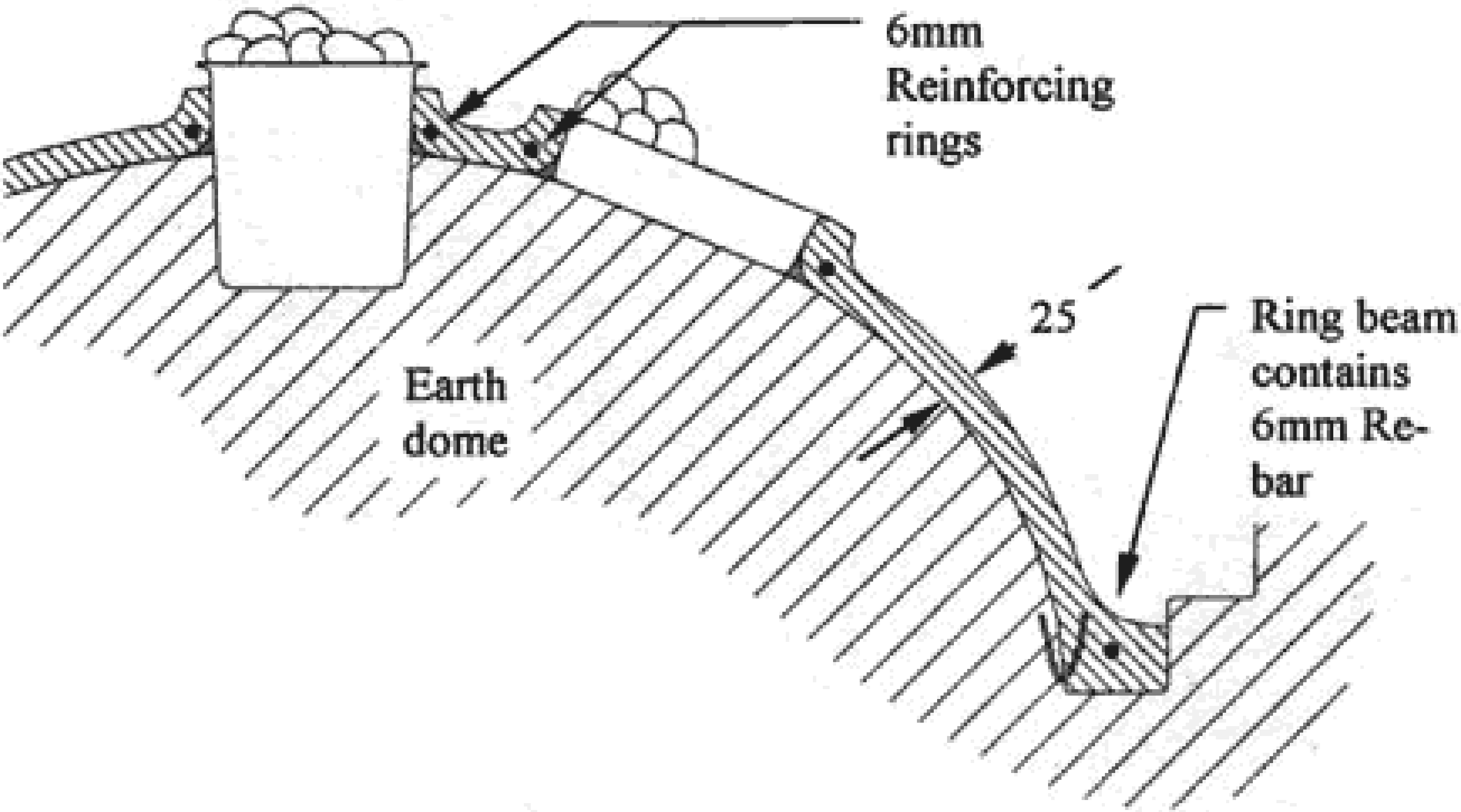


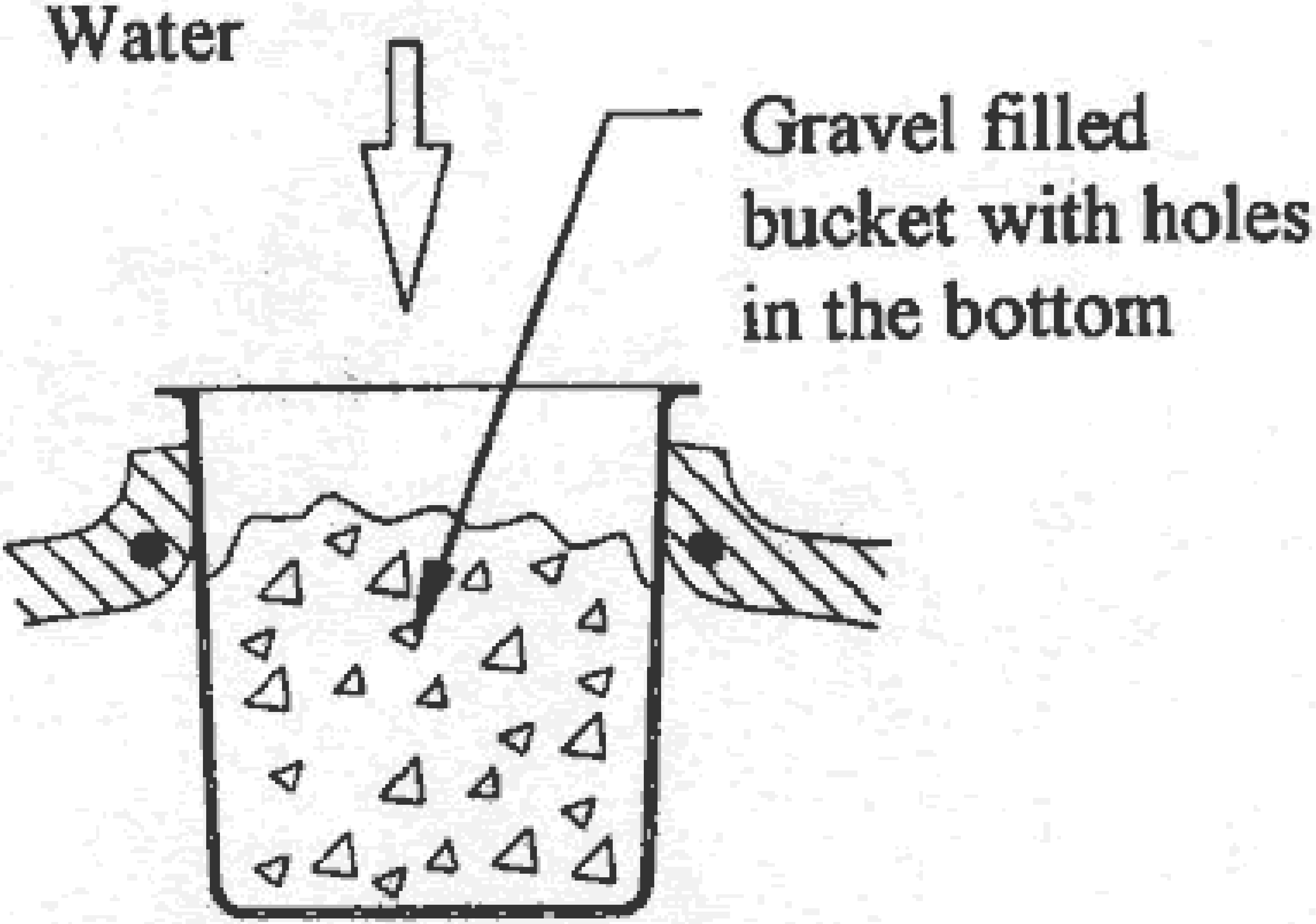
1.1 m
1.1 m
Nail
0.8m
Chain
Reinforcing strips
Plywood
Cutaway
Alternative nail position

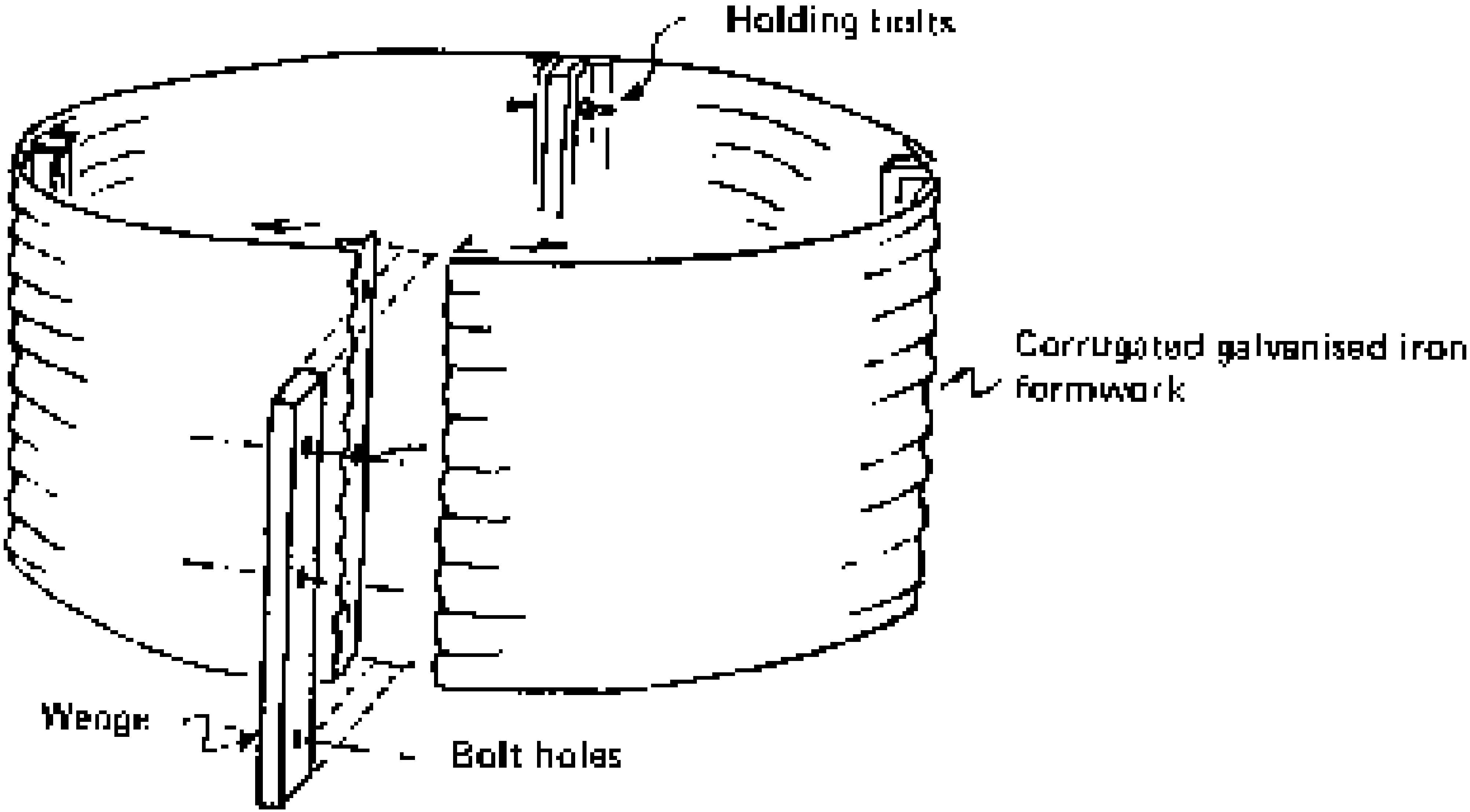

Spirit level
Dome
template
Earth
dome

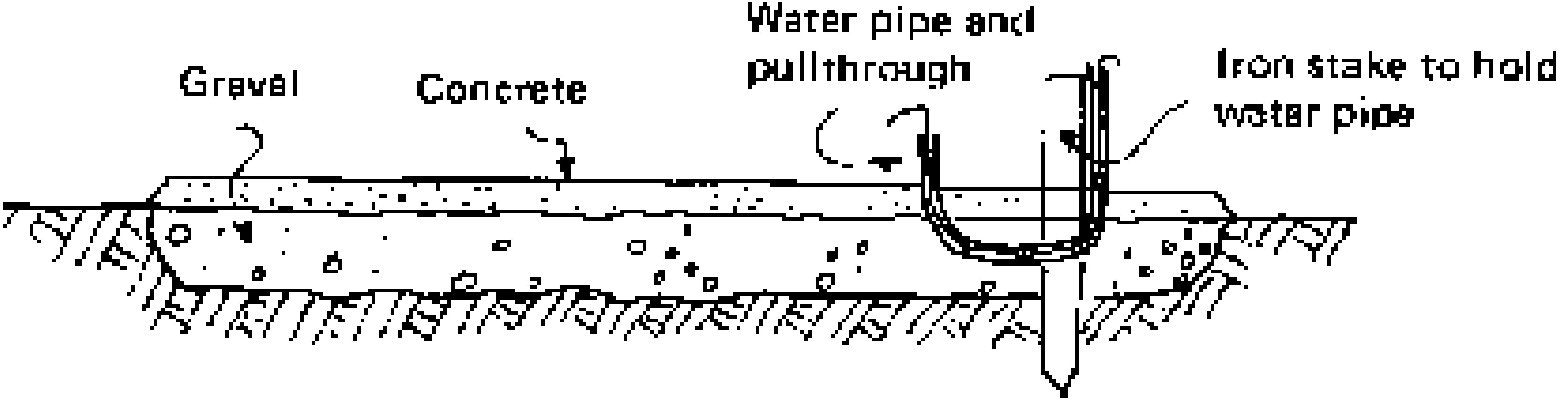

Earth dome
Cutaway base of dome

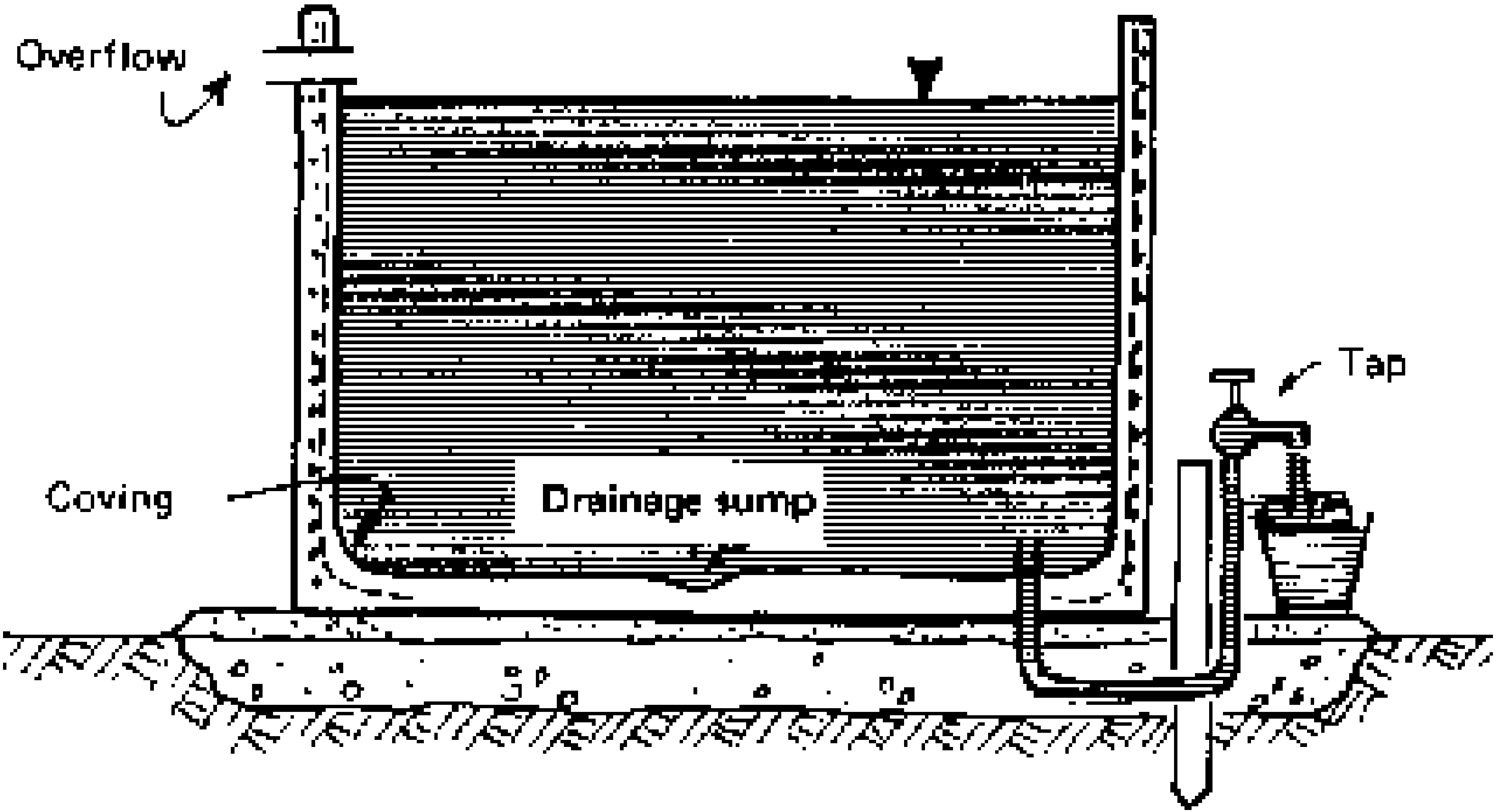

10cm
Chicken mesh strip
Earth dome
Crisscross face
Backfilled with mud

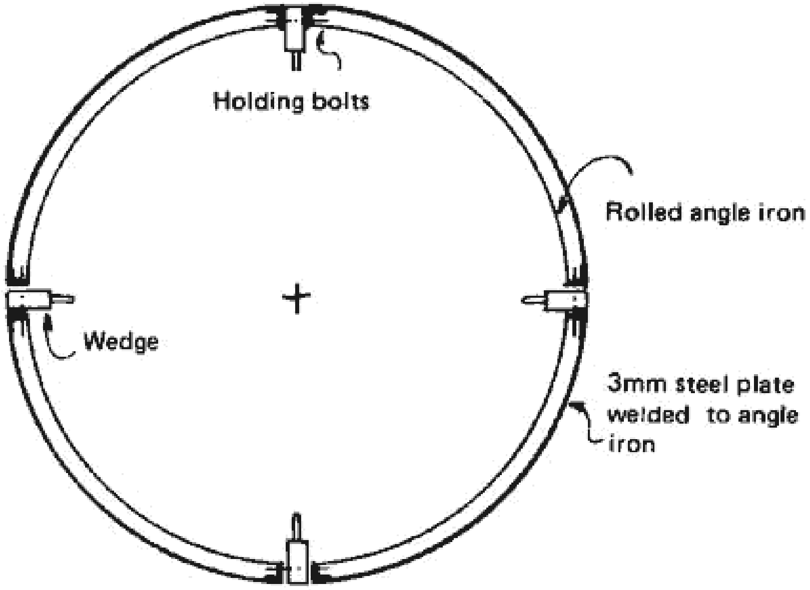

6mm
Reinforcing
rings
25
Ring beam
contains
6mm Re-
bar
Earth
dome


Water
Gravel filled
bucket with holes
in the bottom

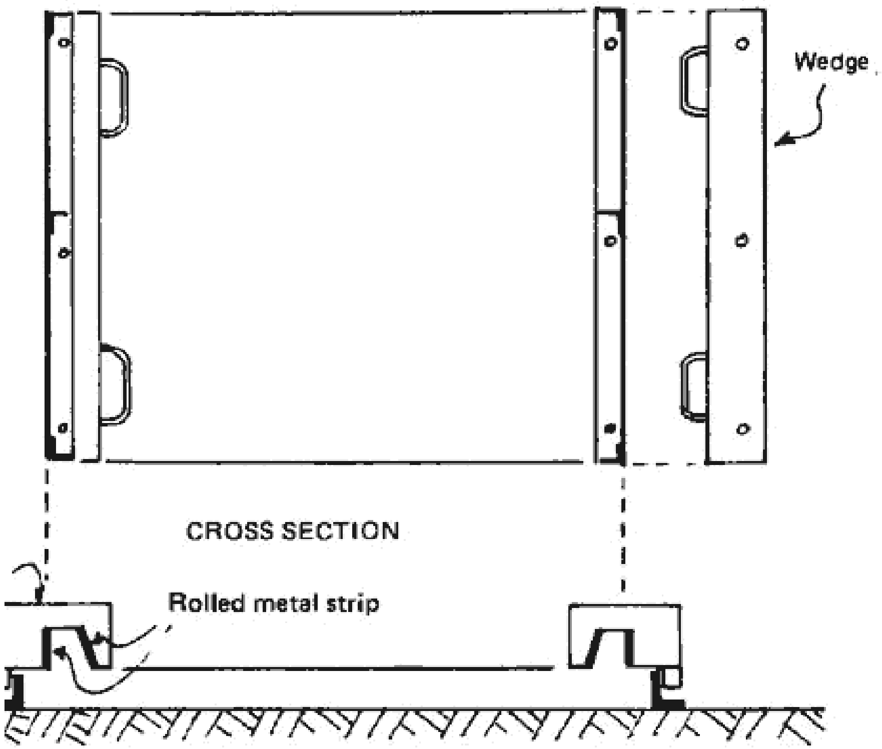

Holding bolts
Corrugated galvanised iron formwork
Wedge
Bolt holes


Gravel
Concrete
Water pipe and pullthrough
Iron stake to hold water pipe

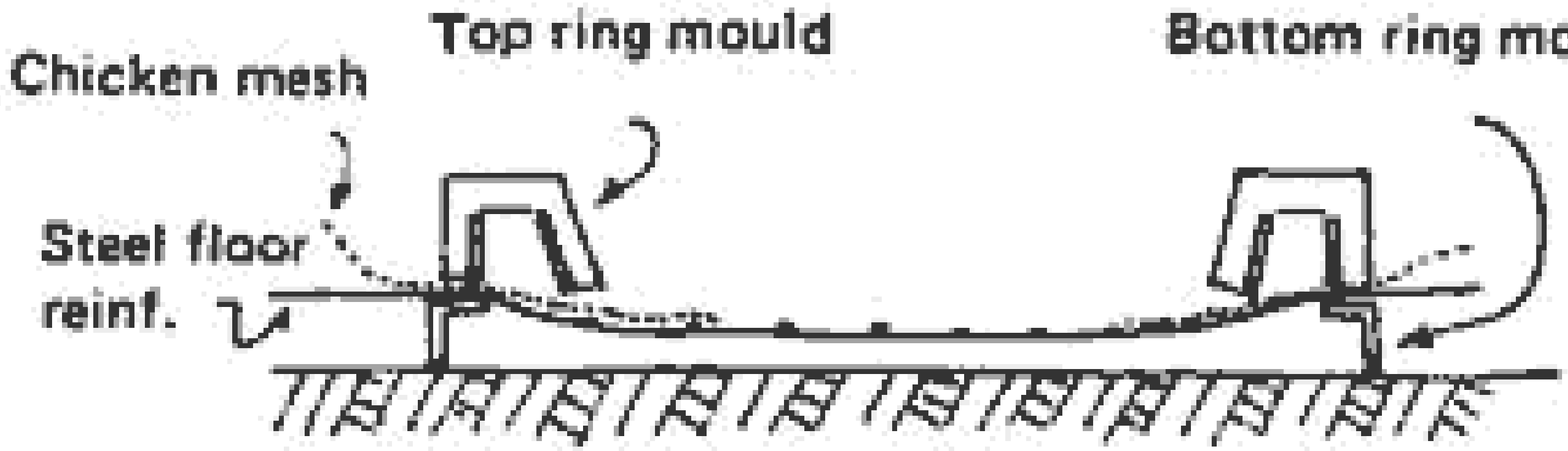

Overflow
Tap
Coving
Drainage sump



PLAN

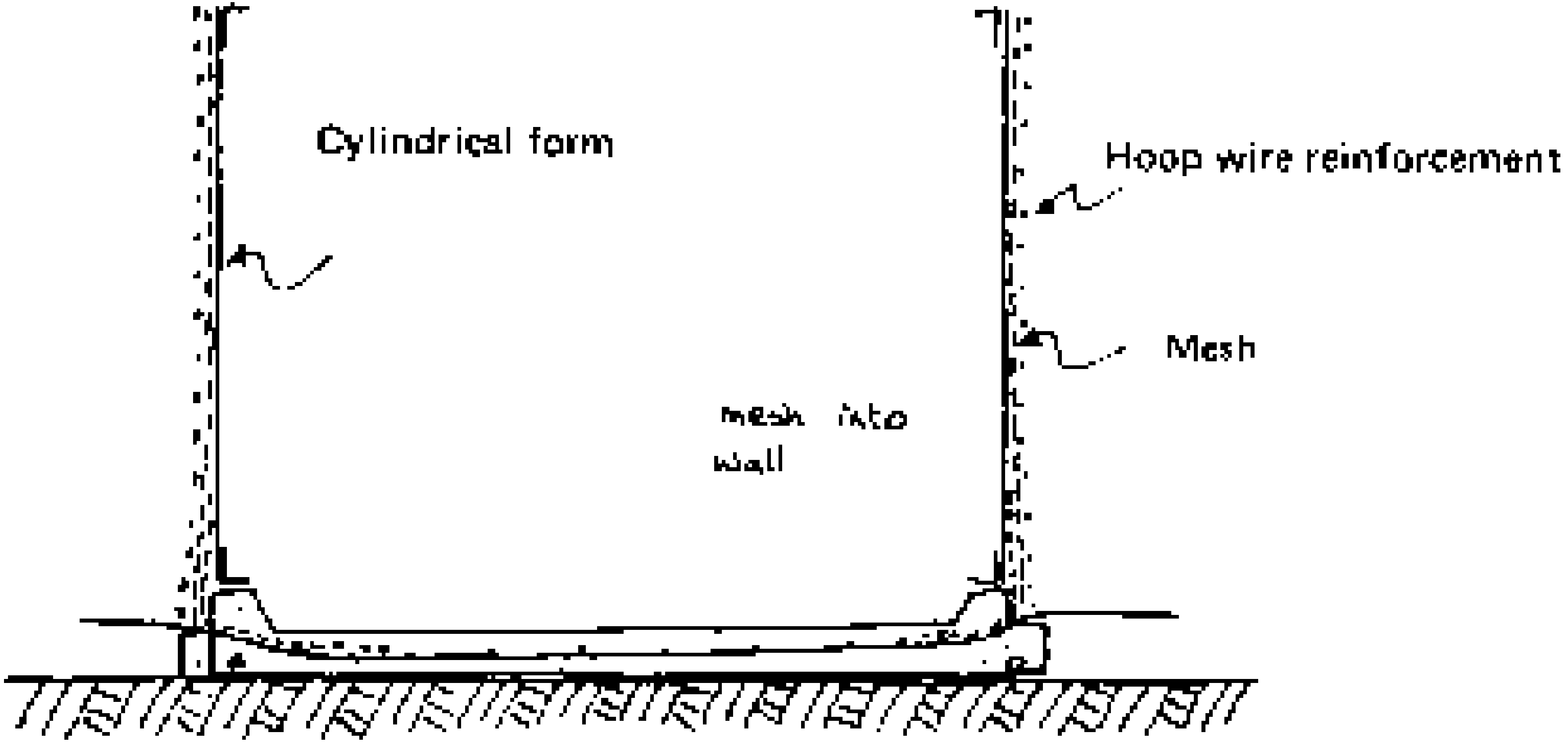


BASE MOULD

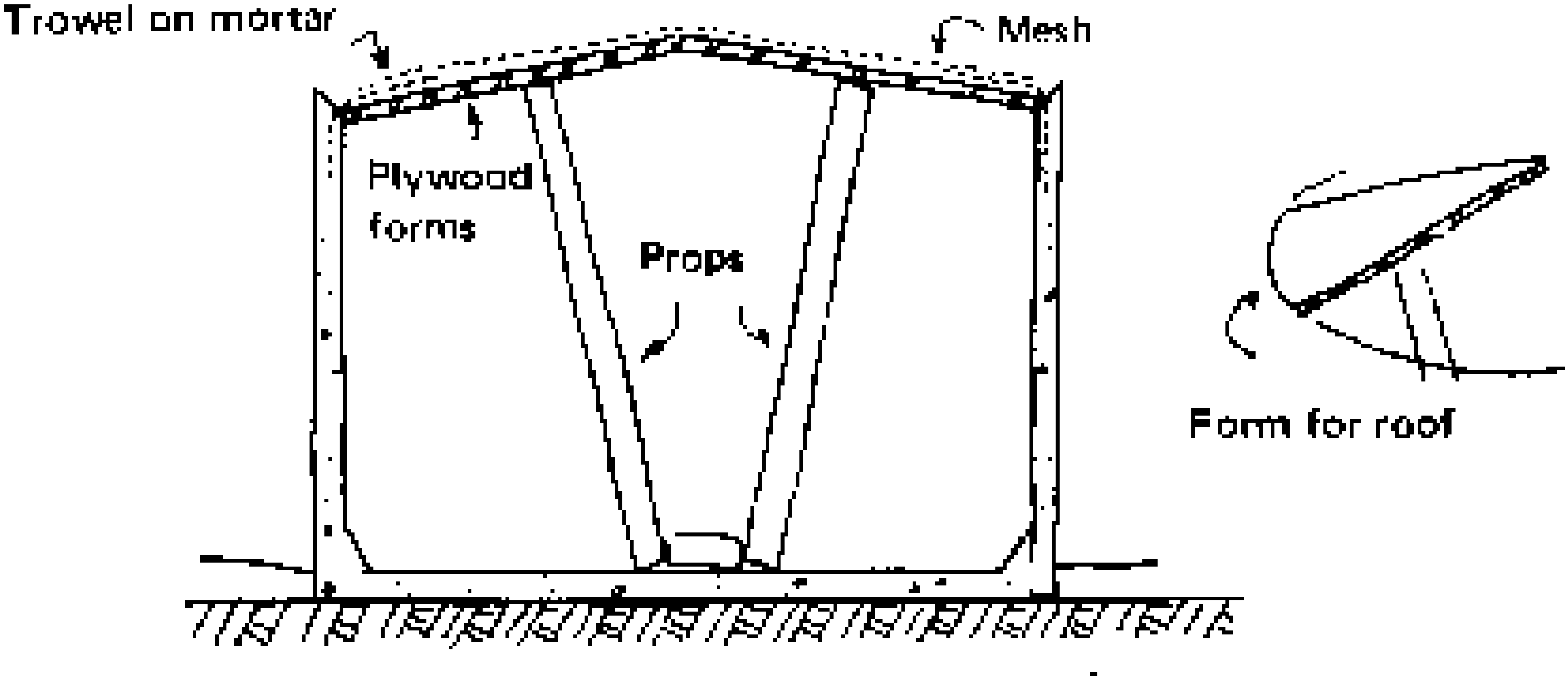


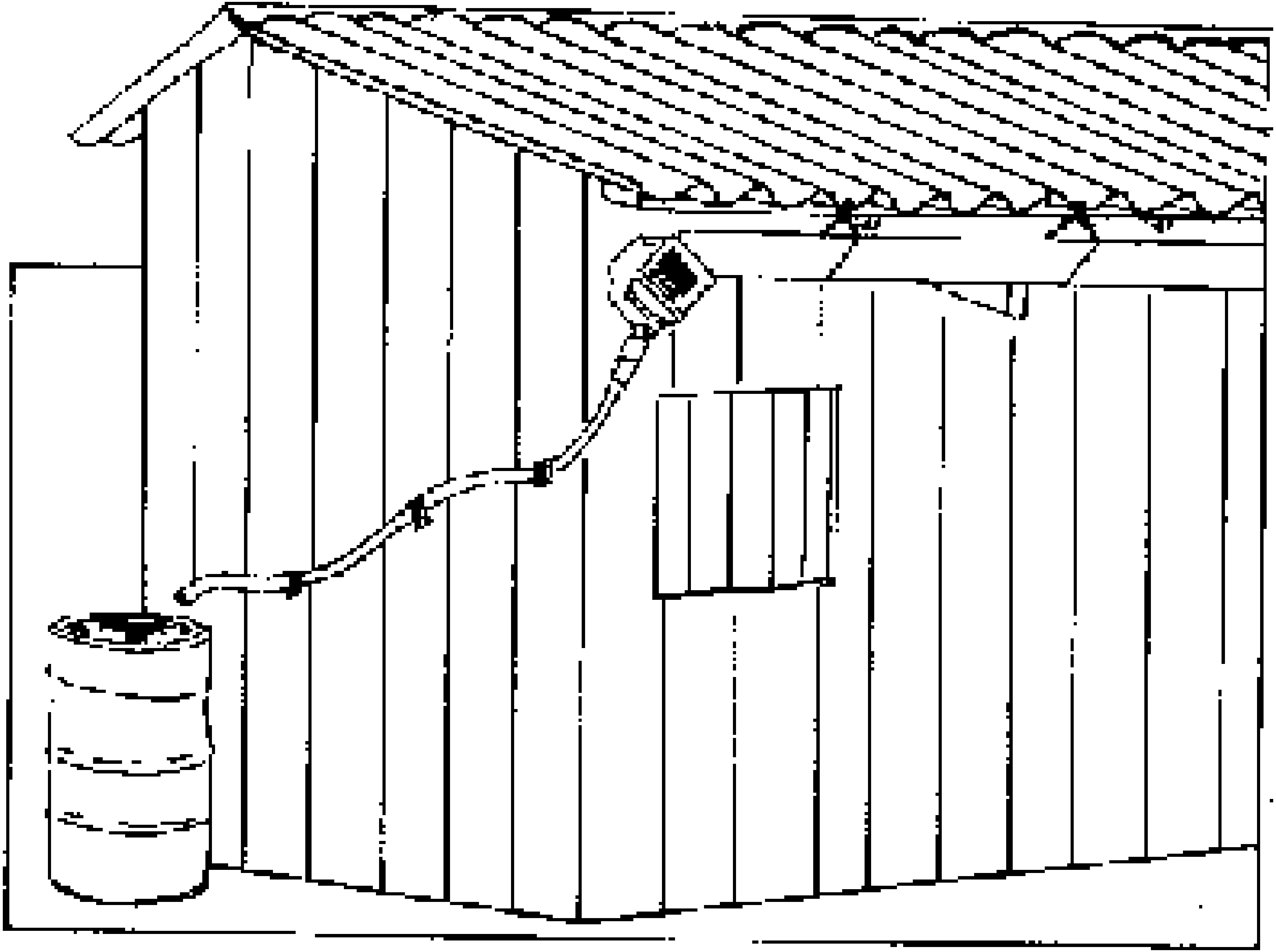
Ring moulds and reinforcement

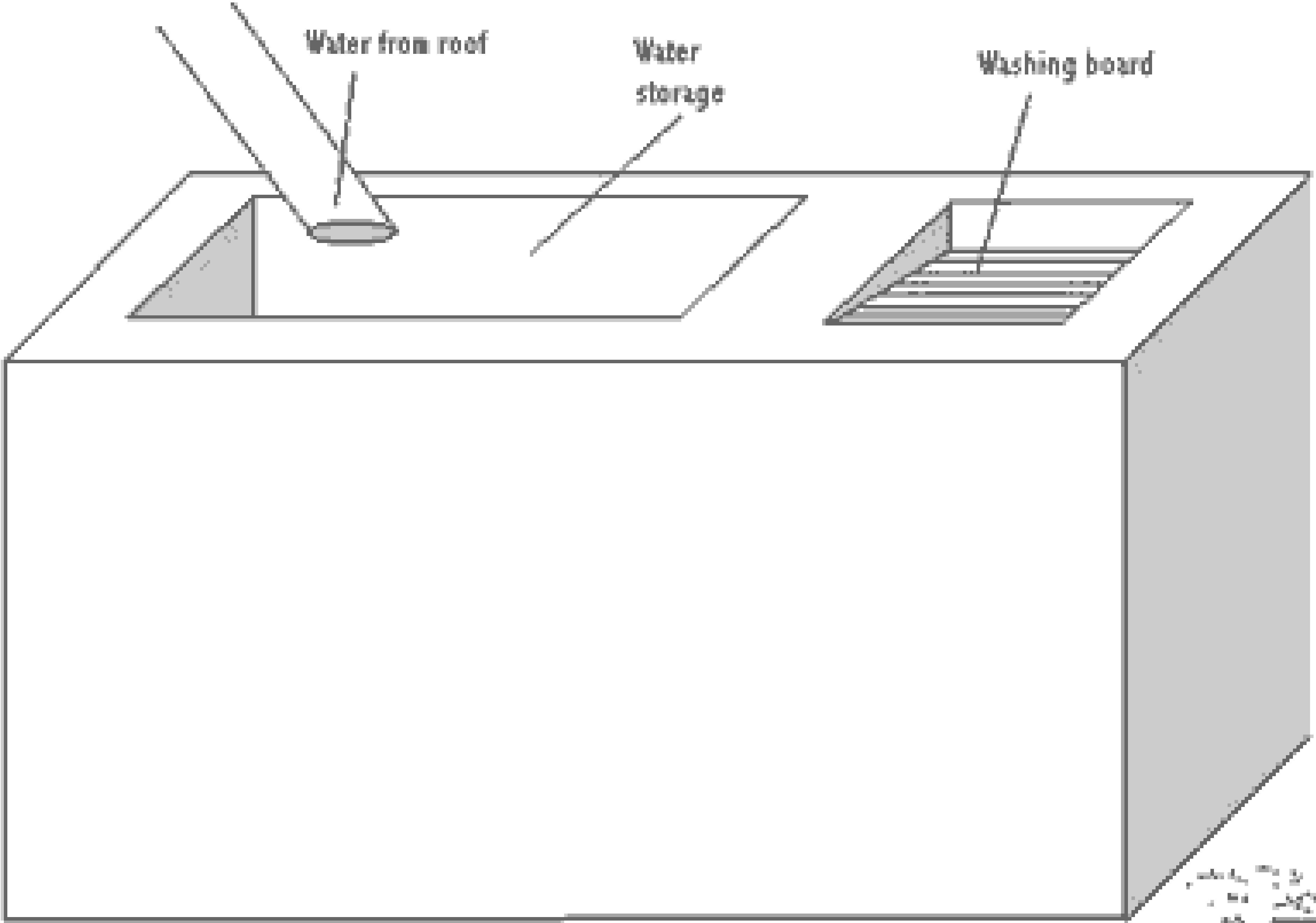
Poured floor

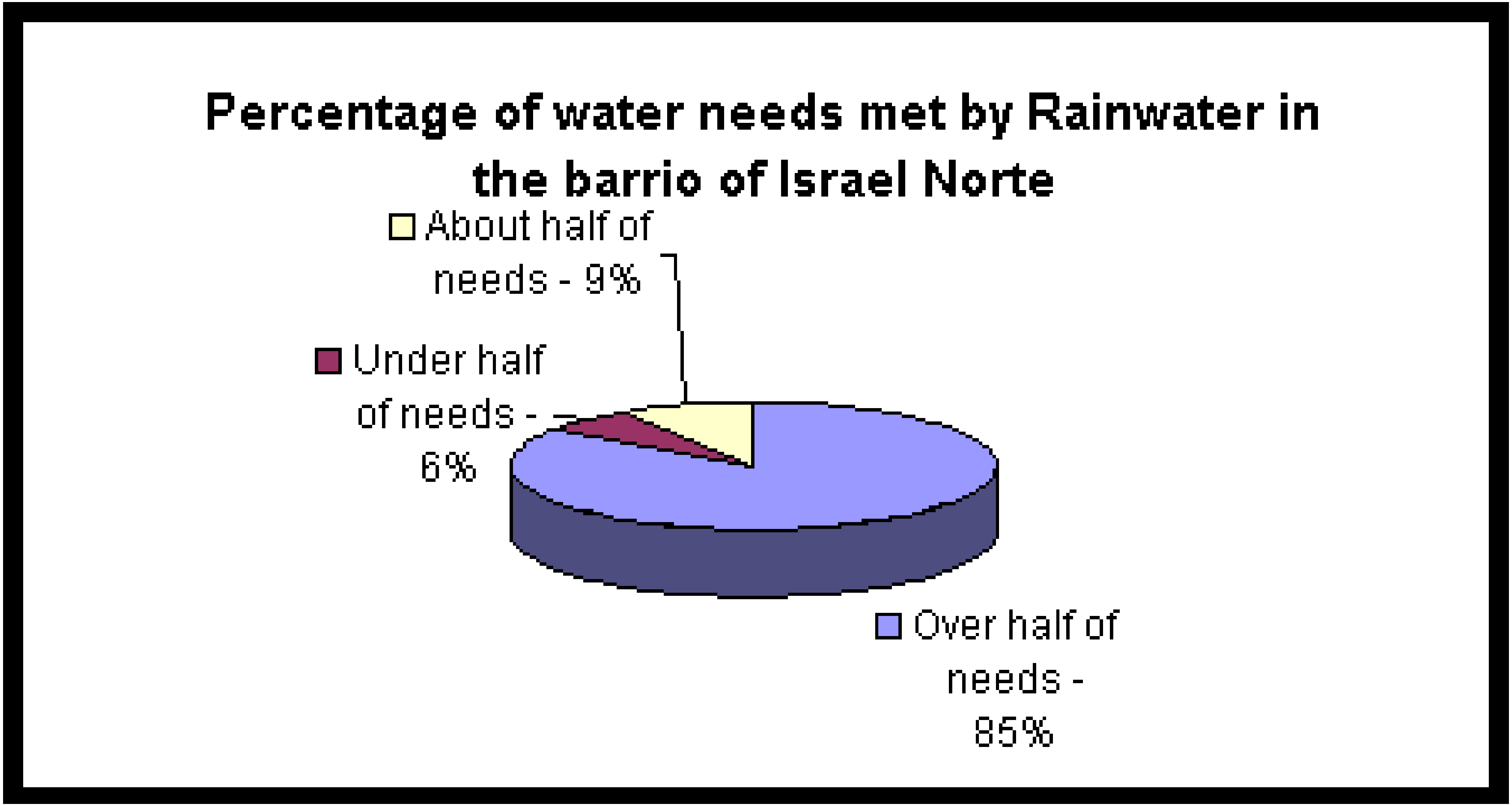


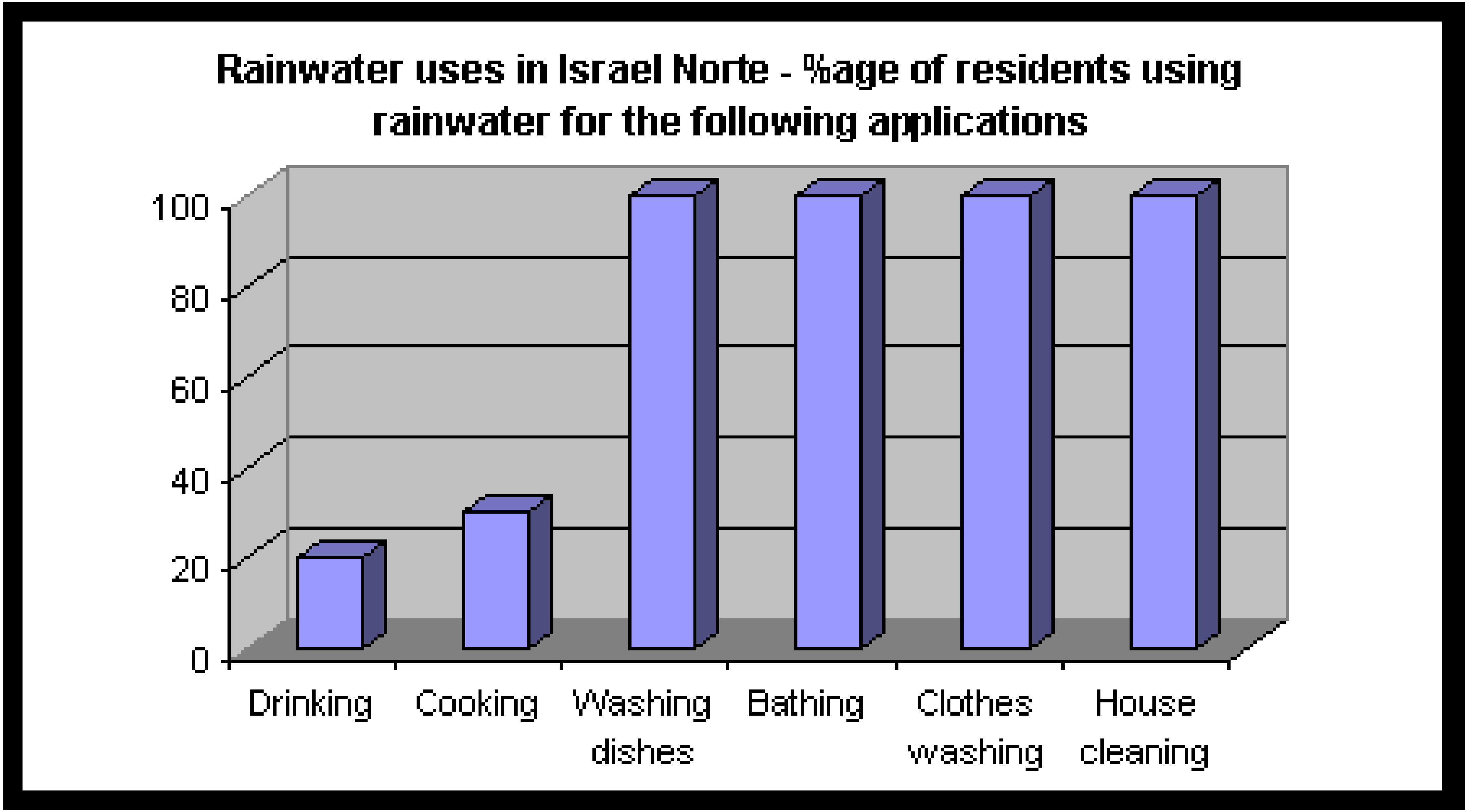
Cylindrical form
Hoop wire reinforcement
Mesh
mesh into
wall

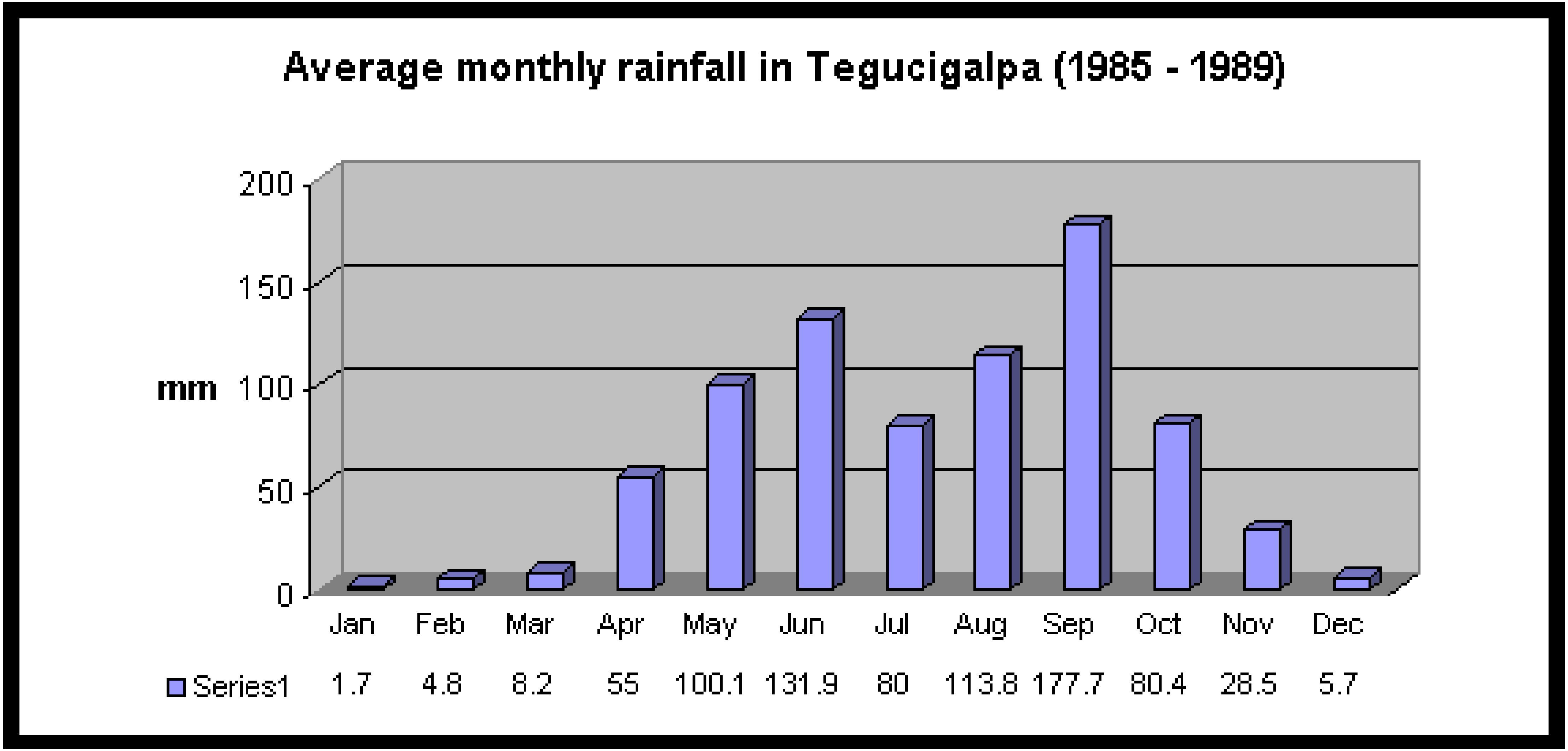

Trowel on mortar
Mesh
Plywood forms
Props
Form for roof



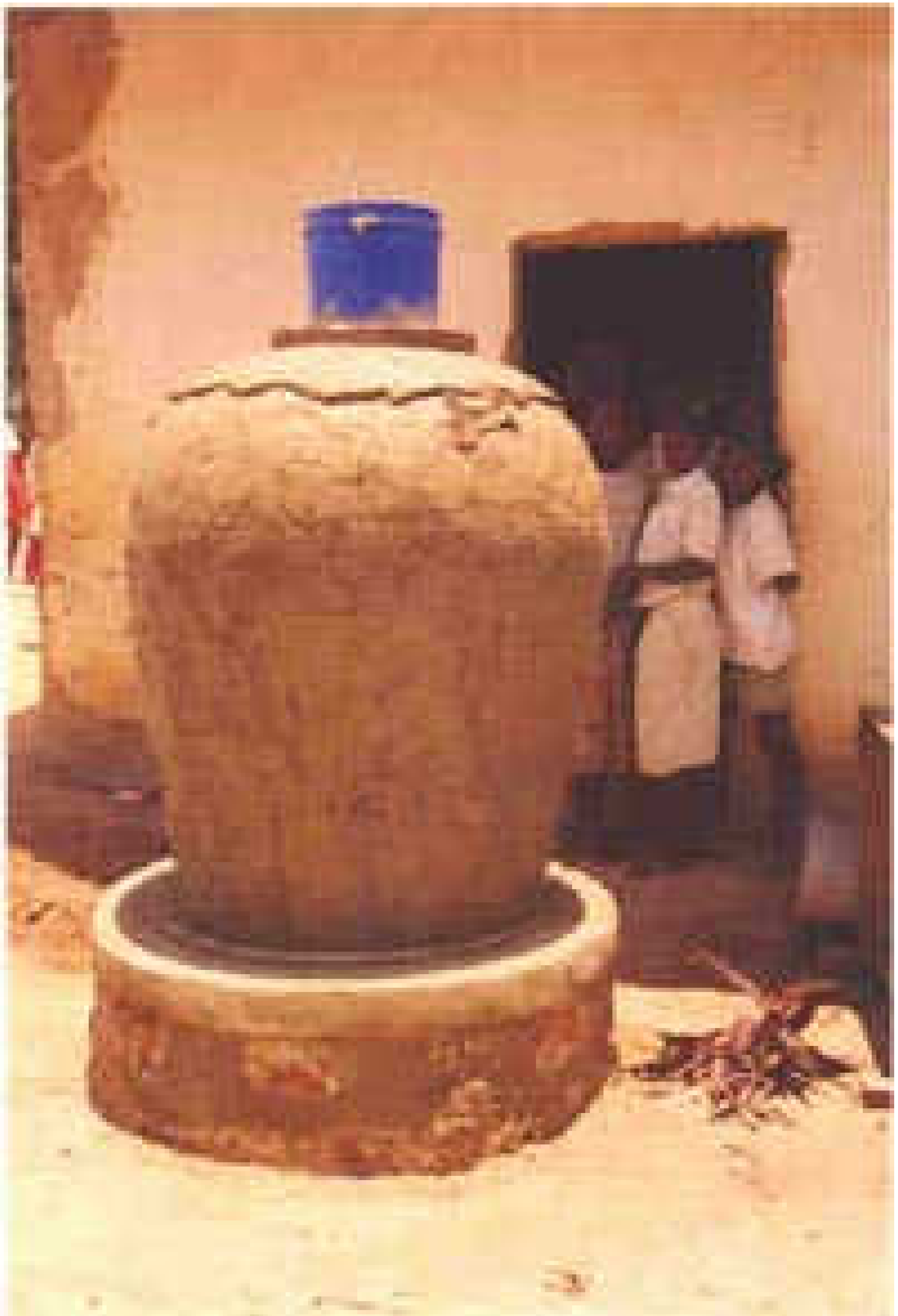

Water from roof
Water storage
Washing board

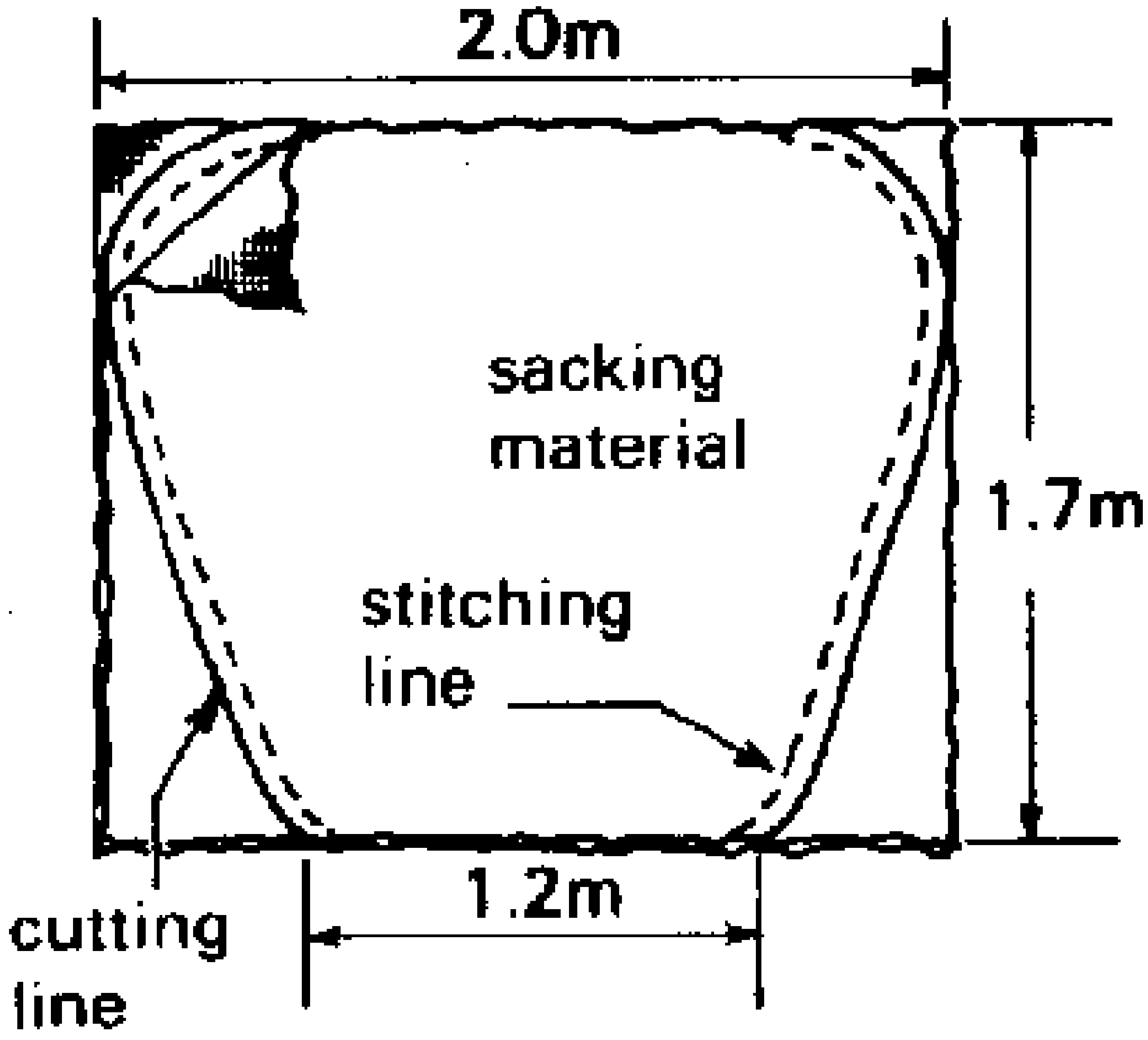

Percentage of water needs met by Rainwater in the barrio of Israel Norte
About half of needs - 9%
Under half of needs - 6%
Over half of needs - 85%


Rainwater uses in Israel Norte - %age of residents using rainwater for the following applications
100
80
60
40
20
0
Drinking
Cooking
Washing dishes
Bathing
Clothes washing
House cleaning

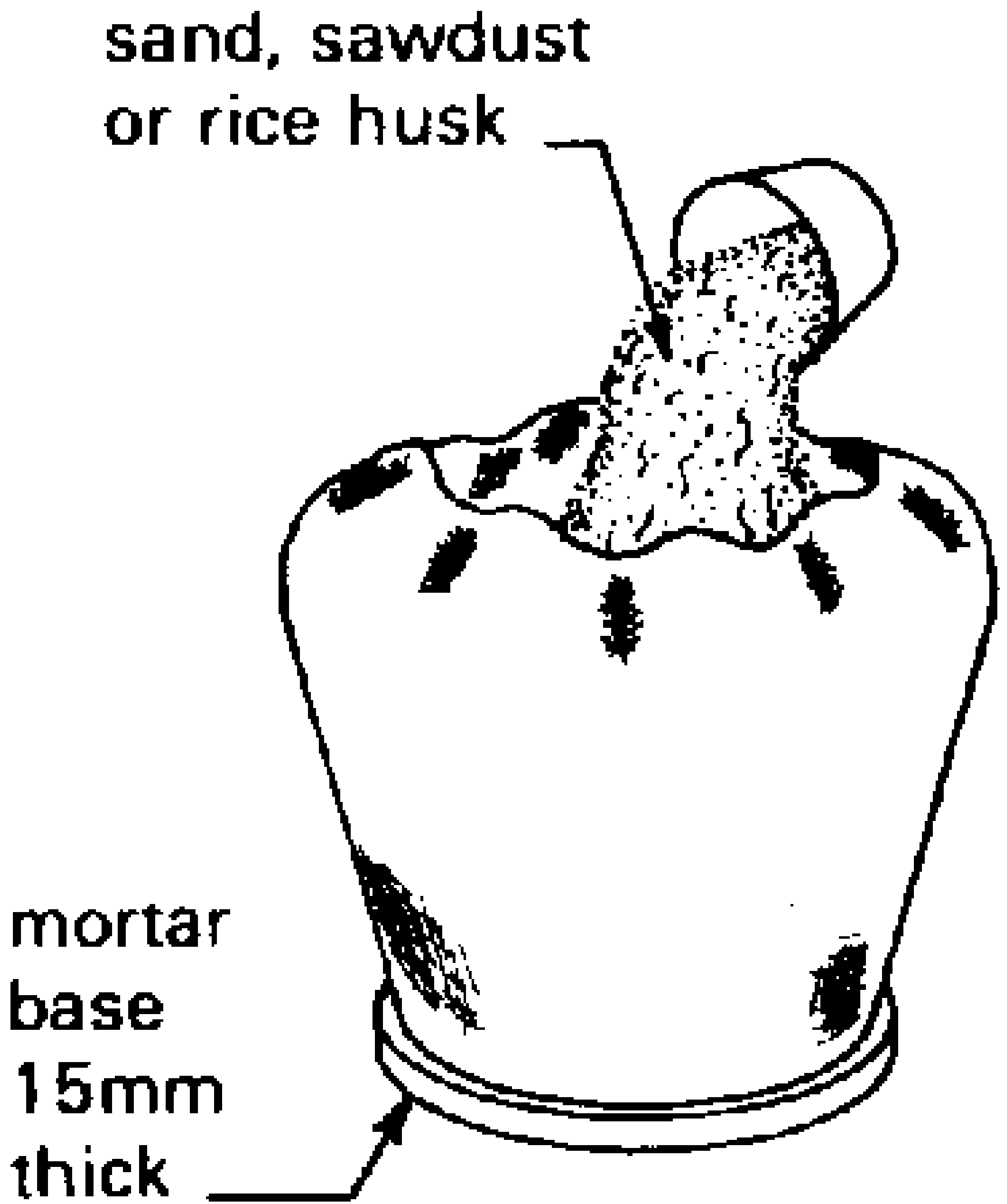


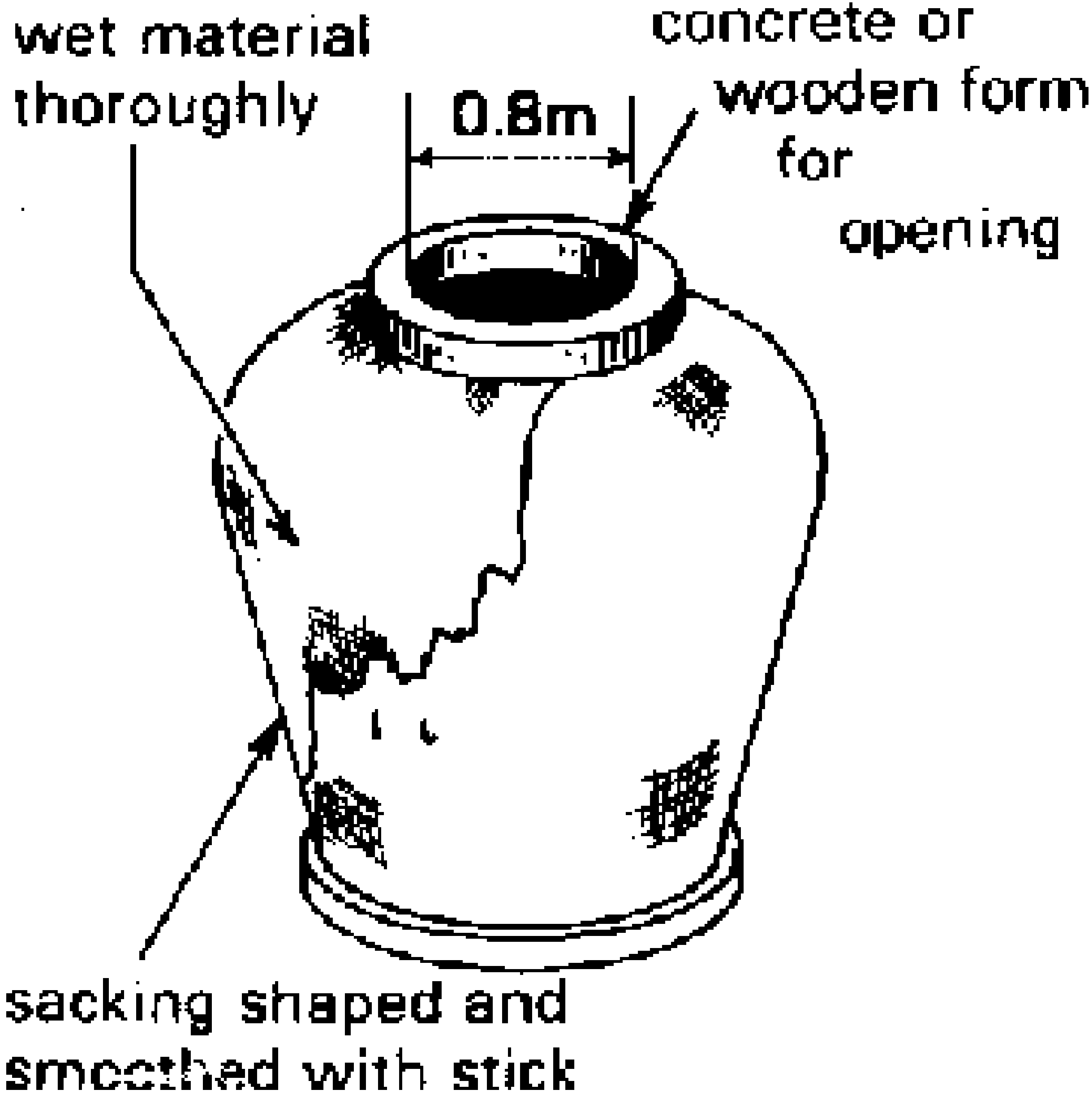
Average monthly rainfall in Tegucigalpa (1985 - 1989)
mm
200
150
100
50
0
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Series1
1.7
4.8
8.2
55
100.1
131.9
80
113.8
177.7
80.4
28.5
5.7

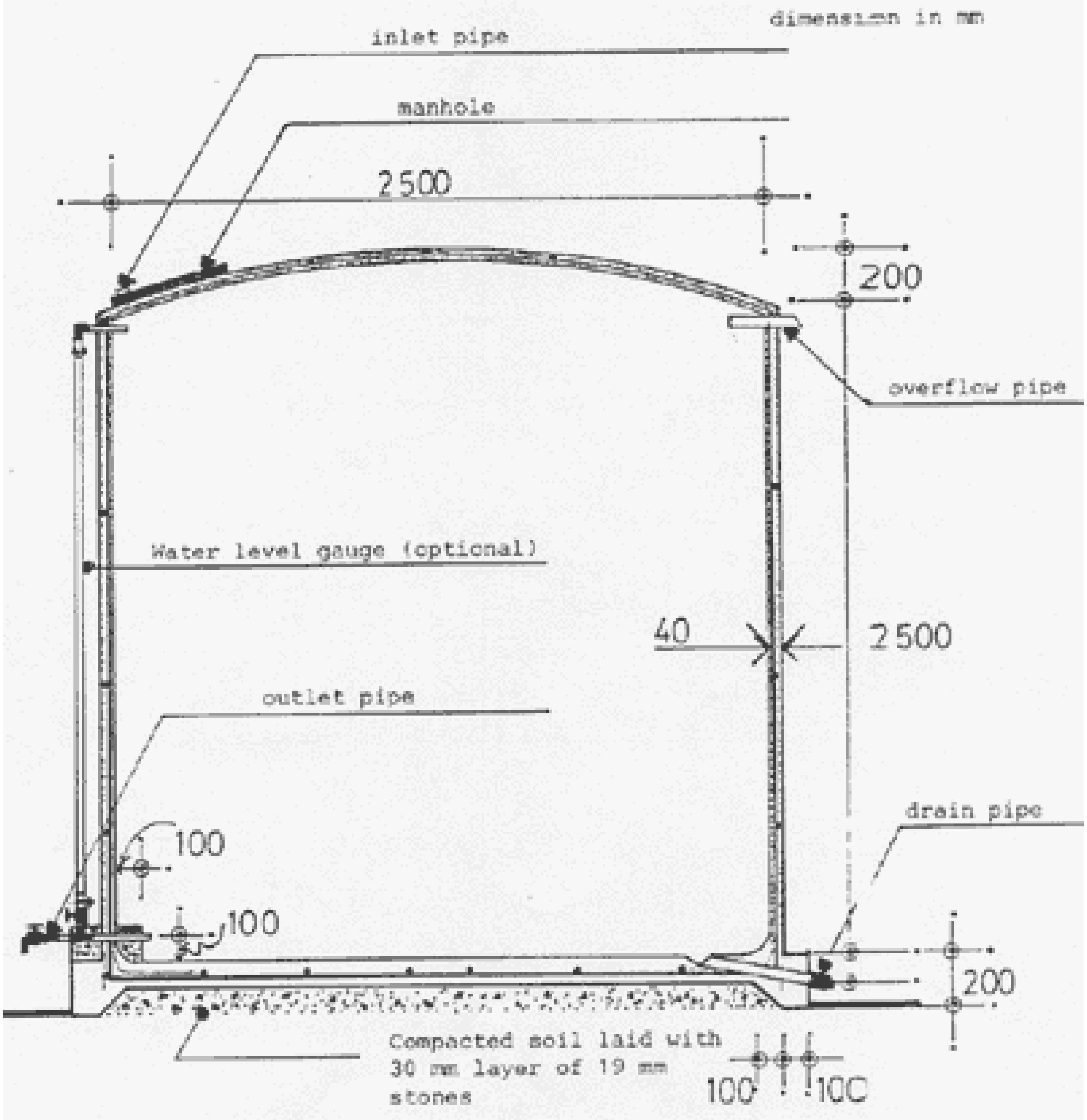


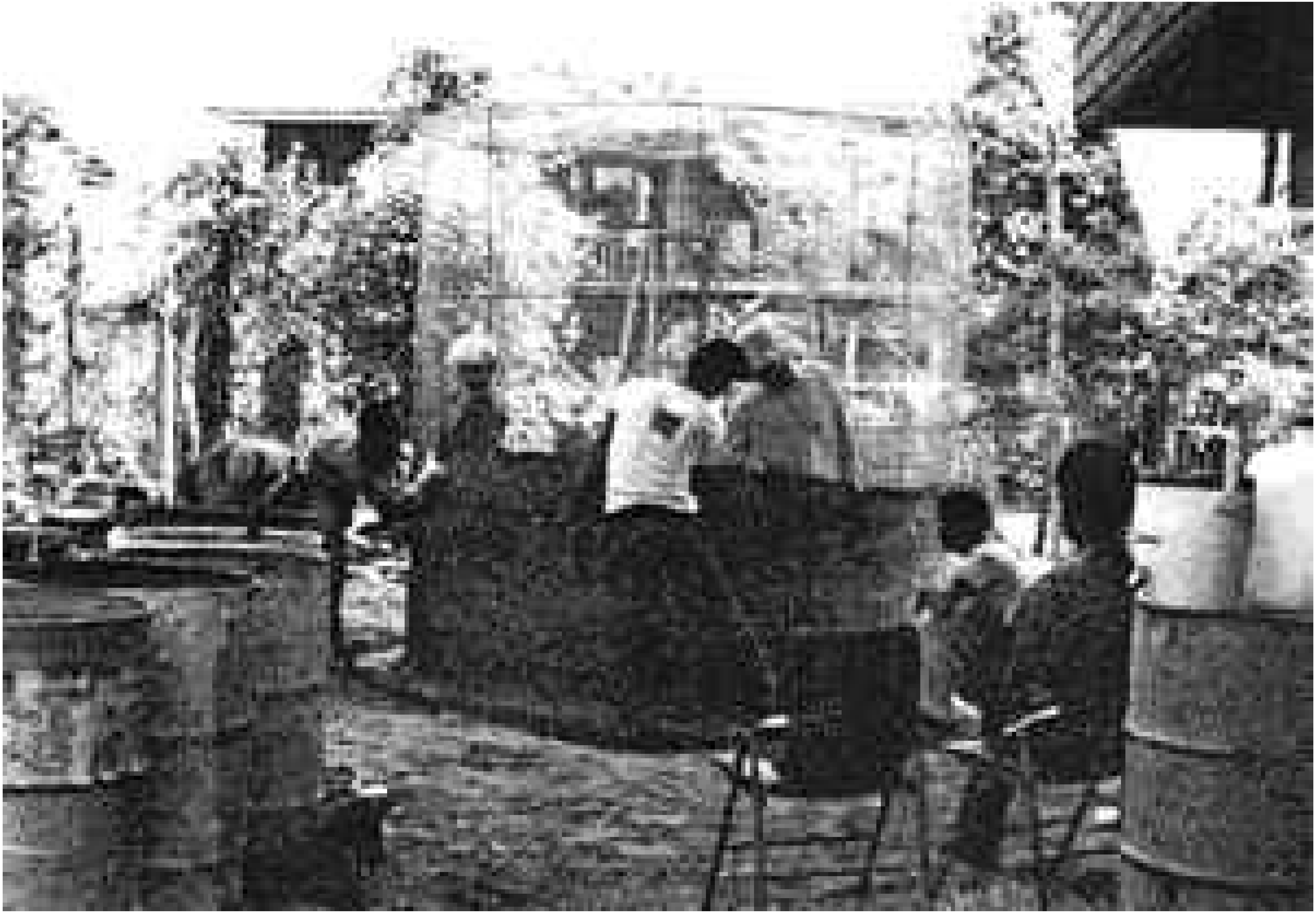


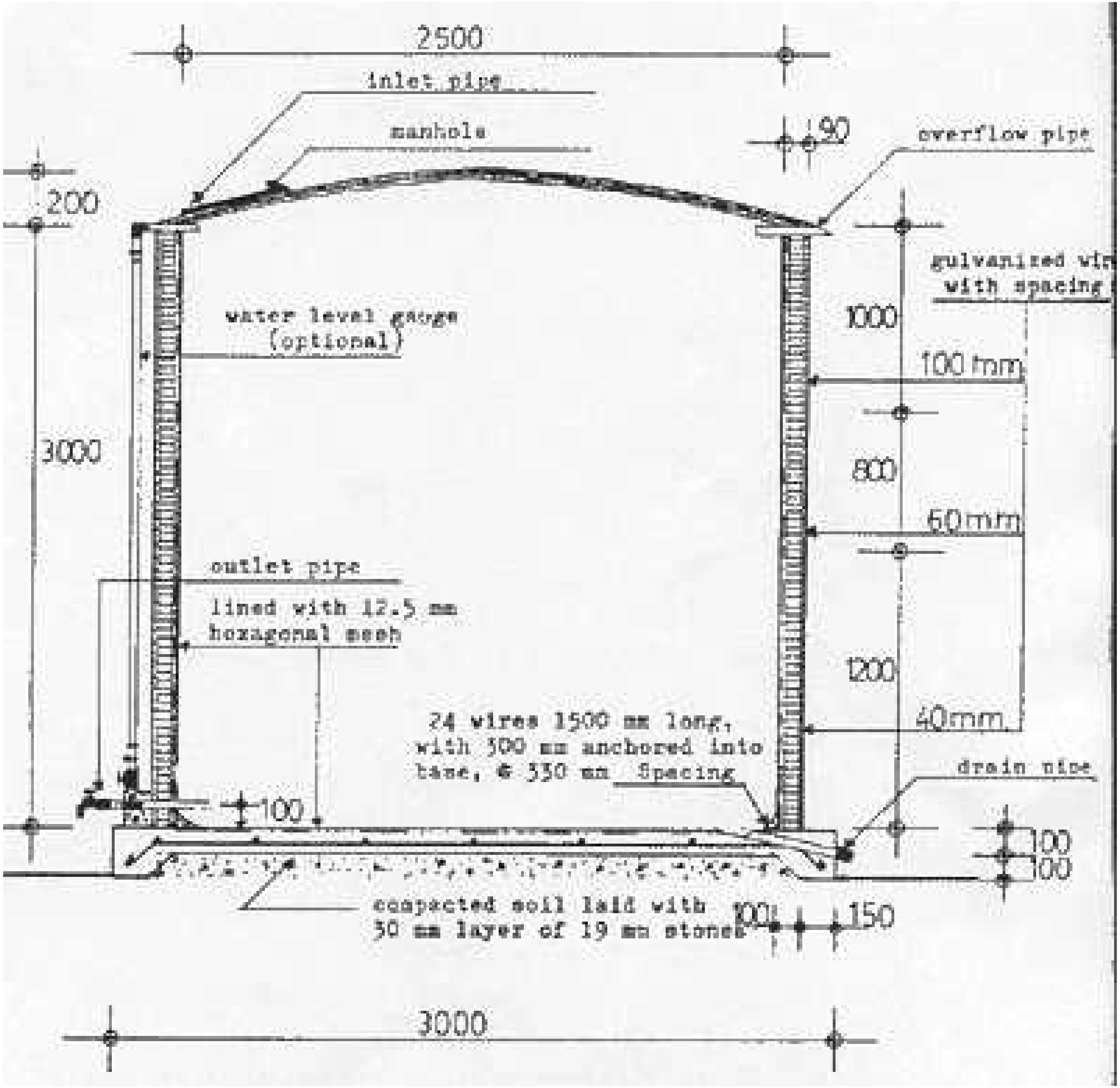
2.0m
sacking
material
1.7m
stitching
line
1.2m
cutting
line


sand, sawdust
or rice husk
mortar
base
15mm
thick

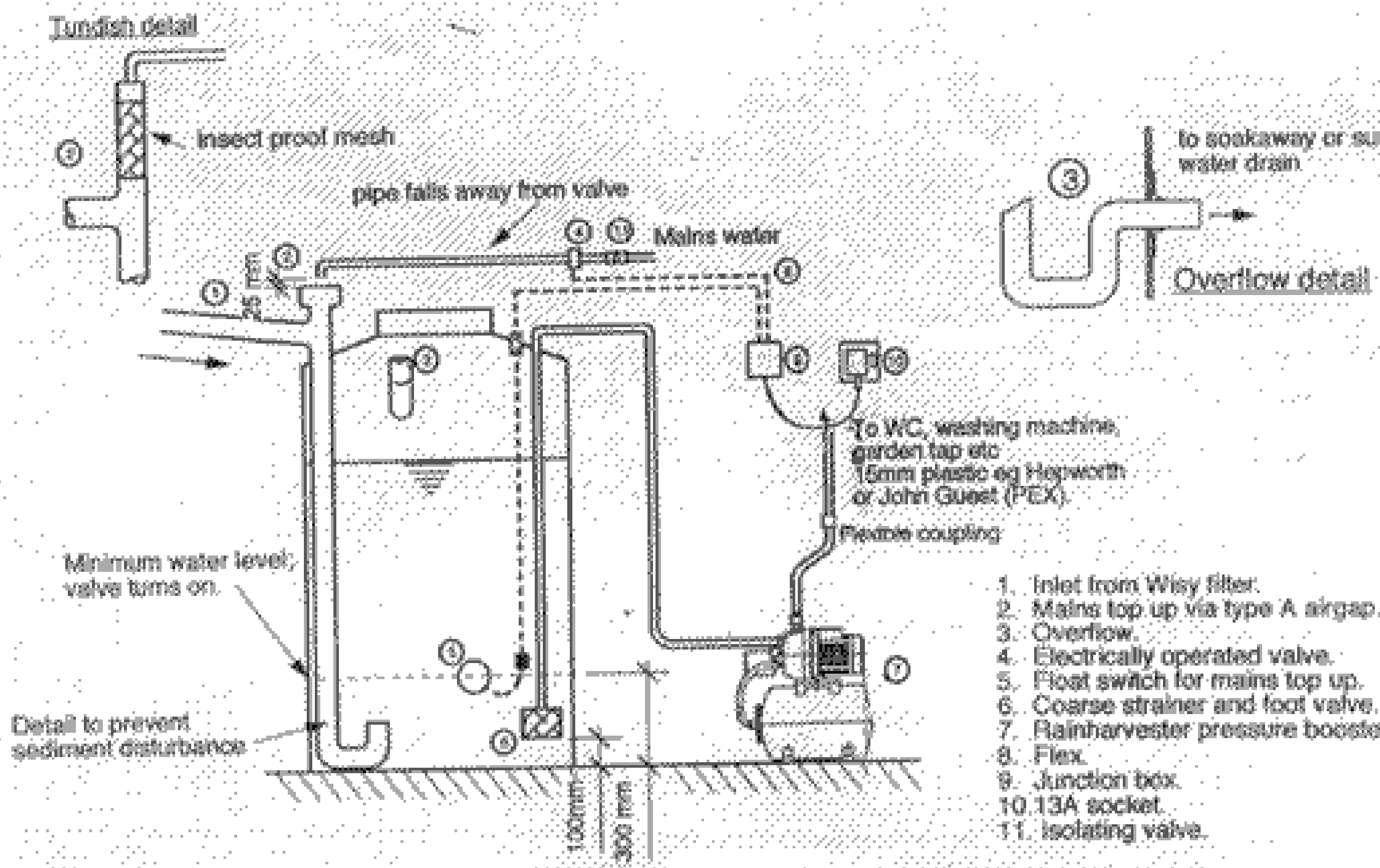

wet material
thoroughly
0.8m
concrete or
wooden form
for
opening
sacking shaped and
smoothed with stick


dimension in mm
inlet pipe
manhole
2500
200
overflow pipe
Water level gauge (optional)
40
2500
outlet pipe
drain pipe
100
100
200
Compacted soil laid with
30 mm layer of 19 mm
stones
100
100



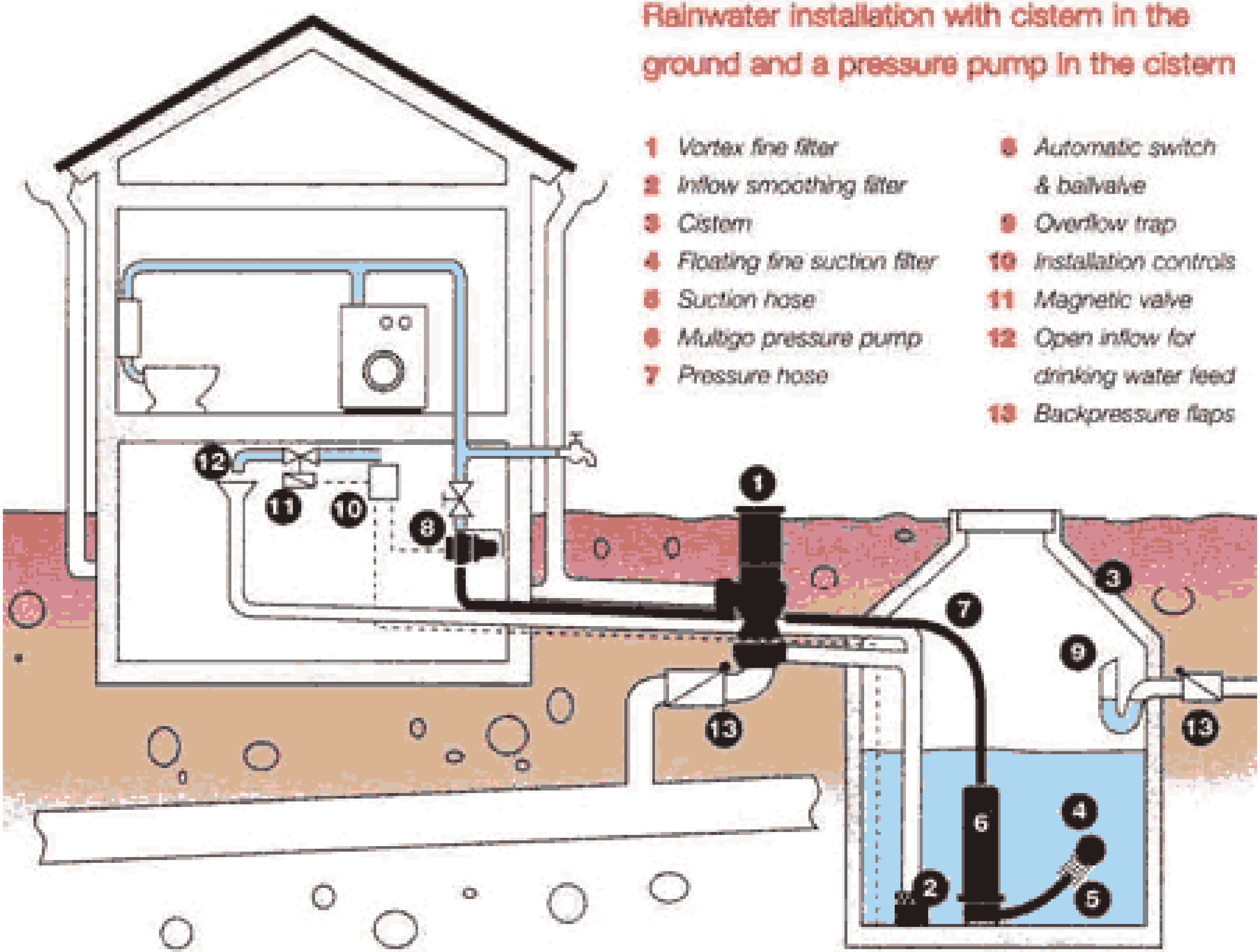


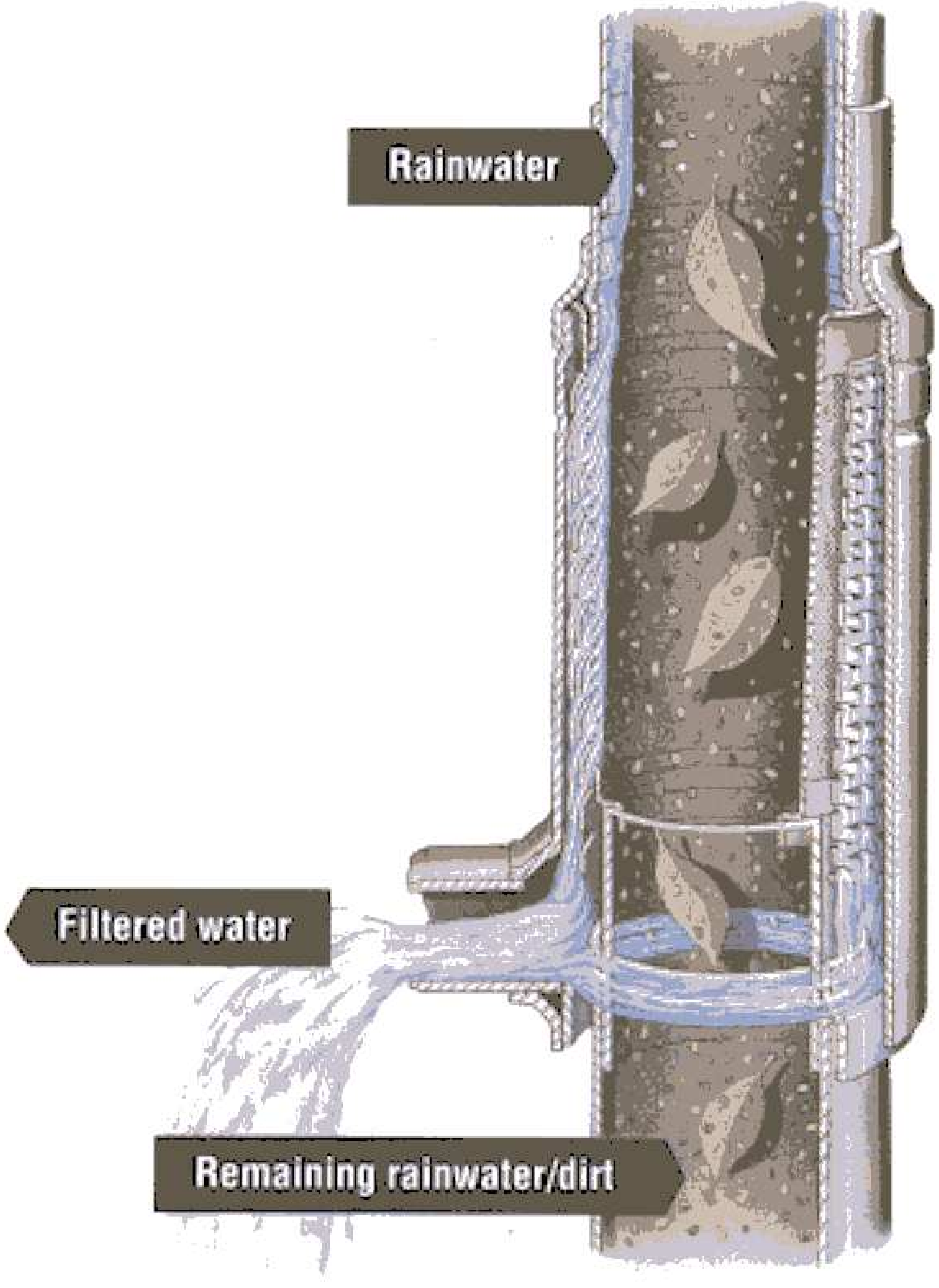
2500
inlet pipe
manhole
90
overflow pipe
200
gulvanized wir
with spacing
water level gauge
(optional)
1000
100 mm
3000
800
60 mm
outlet pipe
lined with 12.5 mm
hexagonal mesh
1200
40mm
24 wires 1500 mm long,
with 300 mm anchored into
base, @ 330 mm Spacing
drain pipe
100
100
100
compacted soil laid with
30 mm layer of 19 mm stones
100
150
3000

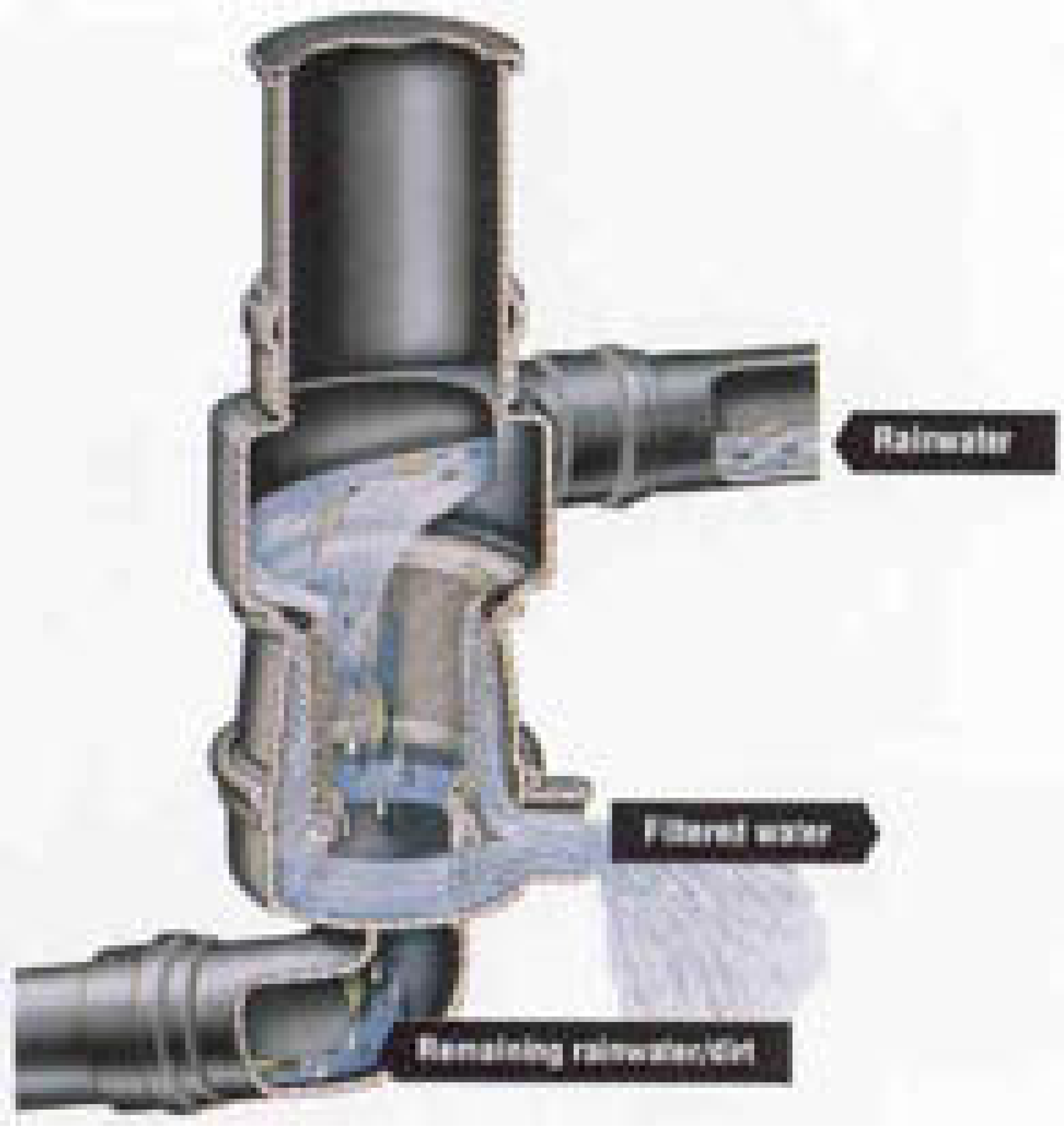


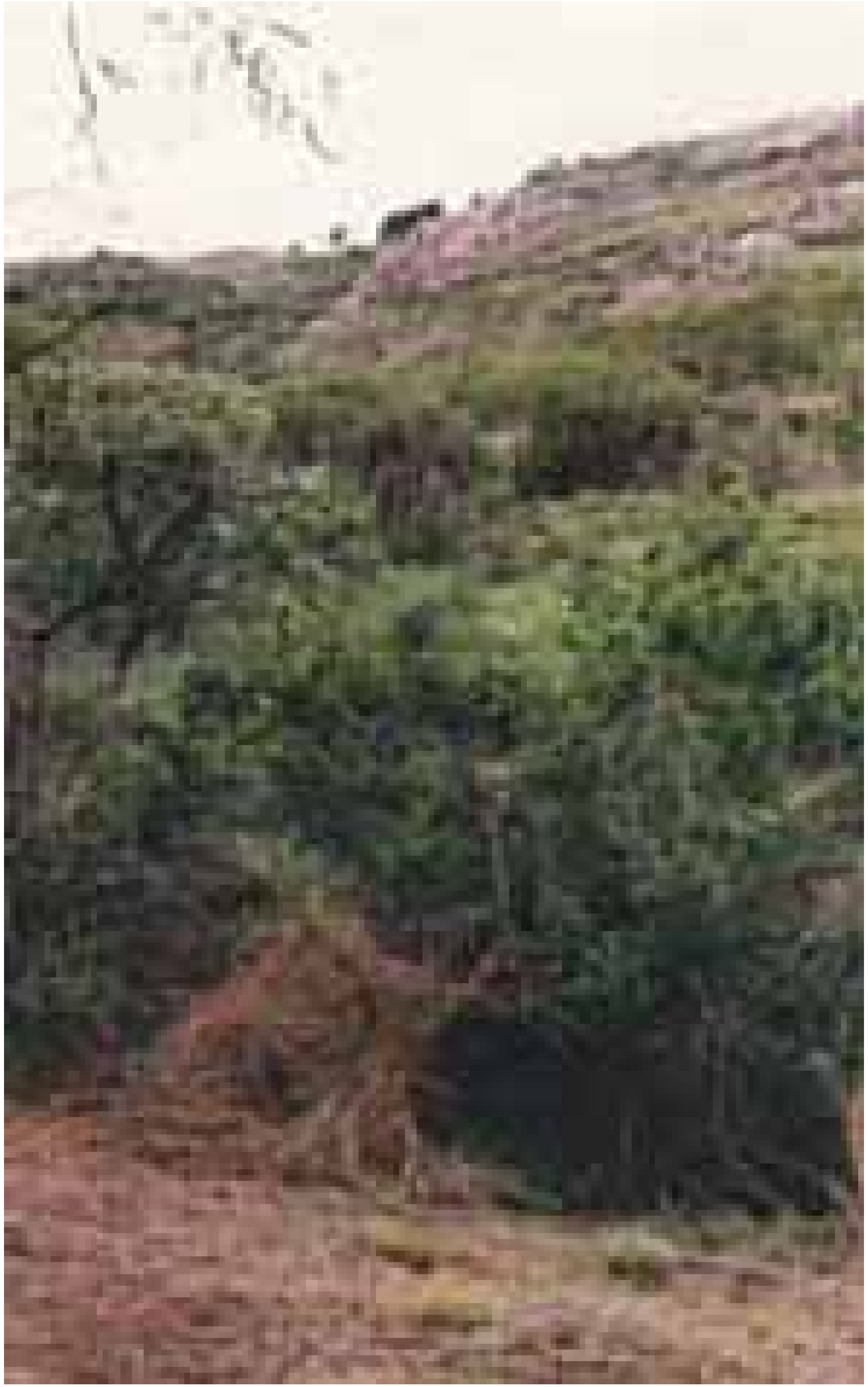
1. Inlet from Wisy filter.
2. Mains top up via type A airgap.
3. Overflow.
4. Electrically operated valve.
5. Float switch for mains top up.
6. Coarse strainer and foot valve.
7. Rainharvester pressure booster pump.
8. Flex.
9. Junction box.
10. 13A socket.
11. Isolating valve.

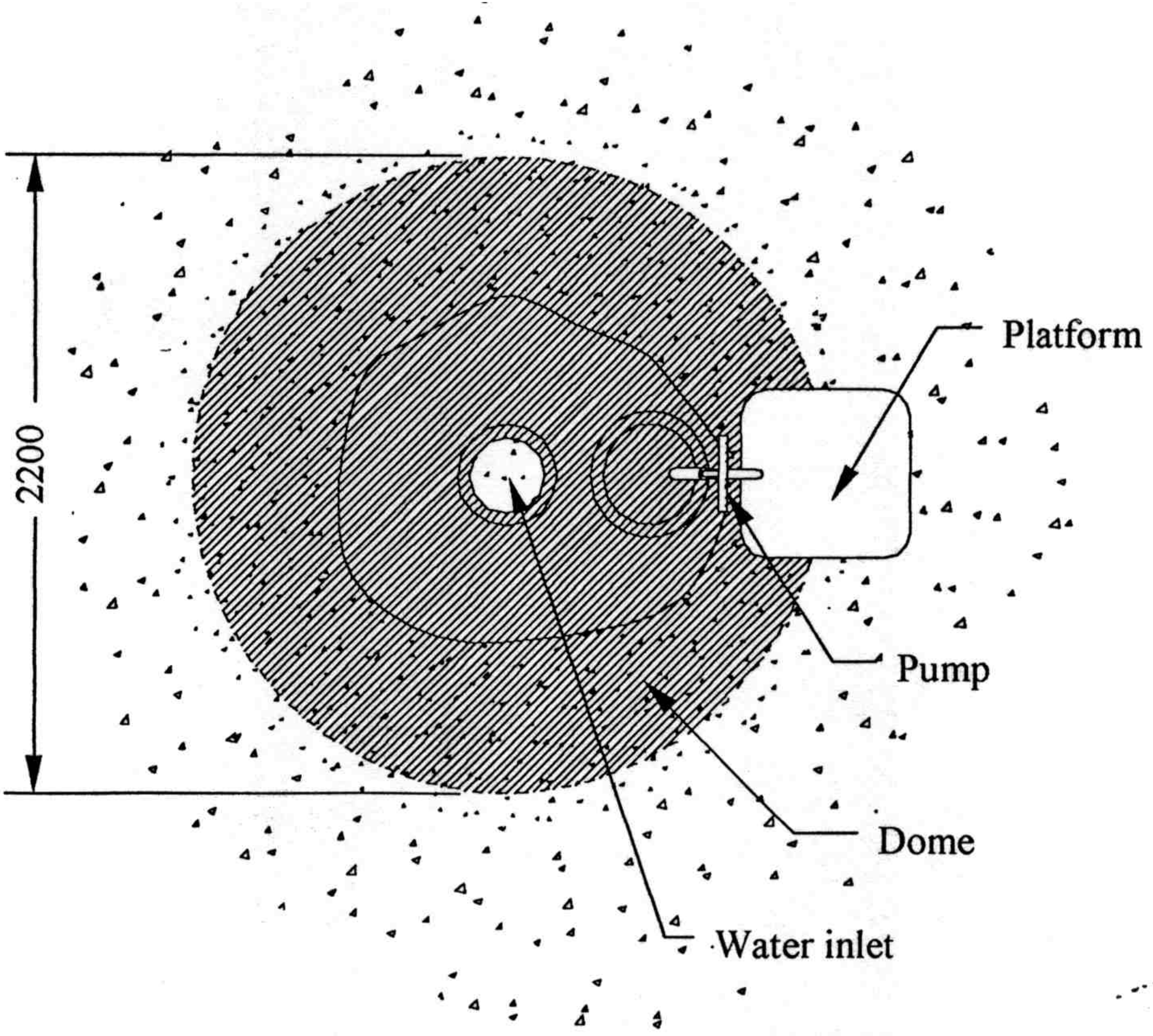
Rainwater installation with cistern in the ground and a pressure pump in the cistern



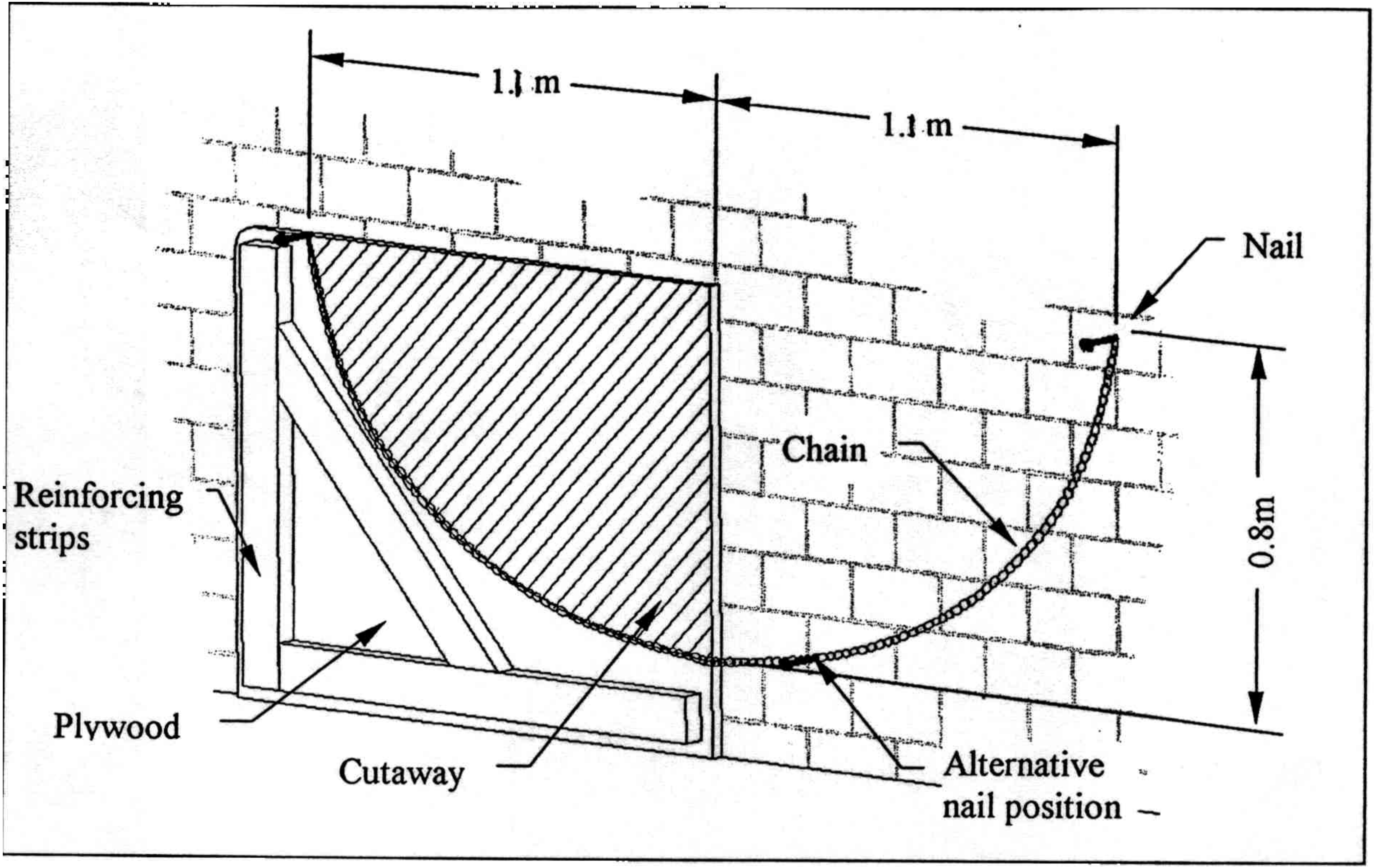

Rainwater
Filtered water
Remaining rainwater/dirt


Rainwater
Filtered water
Remaining rainwater/dirt

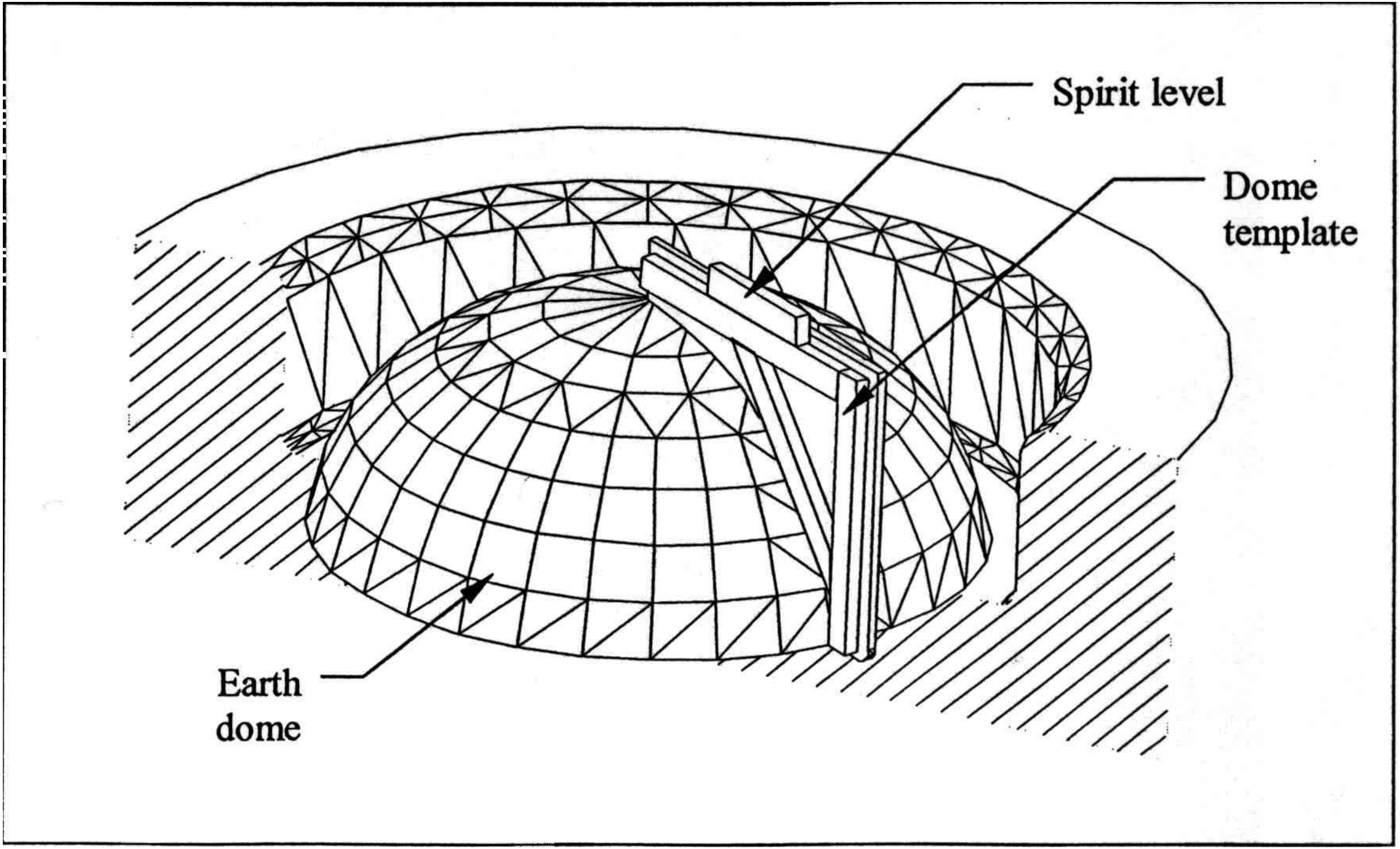


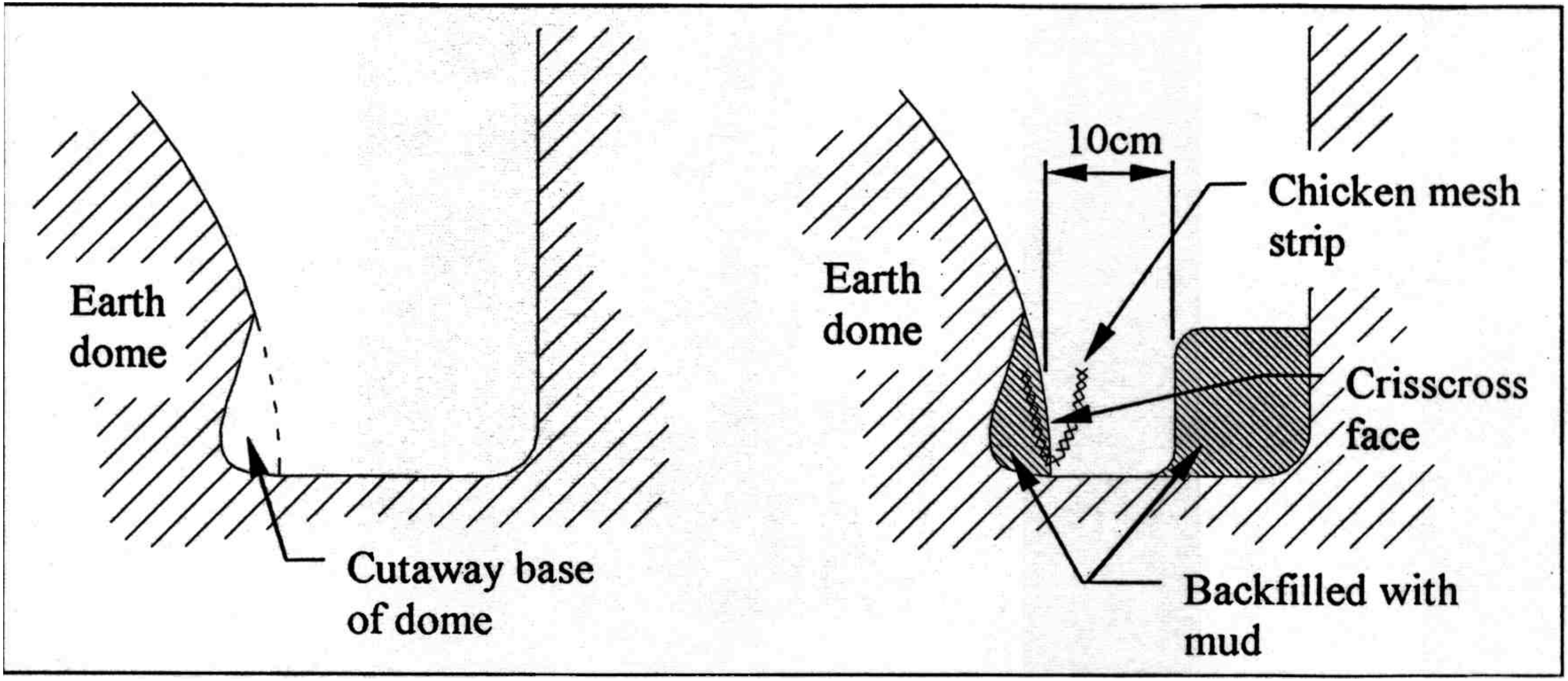


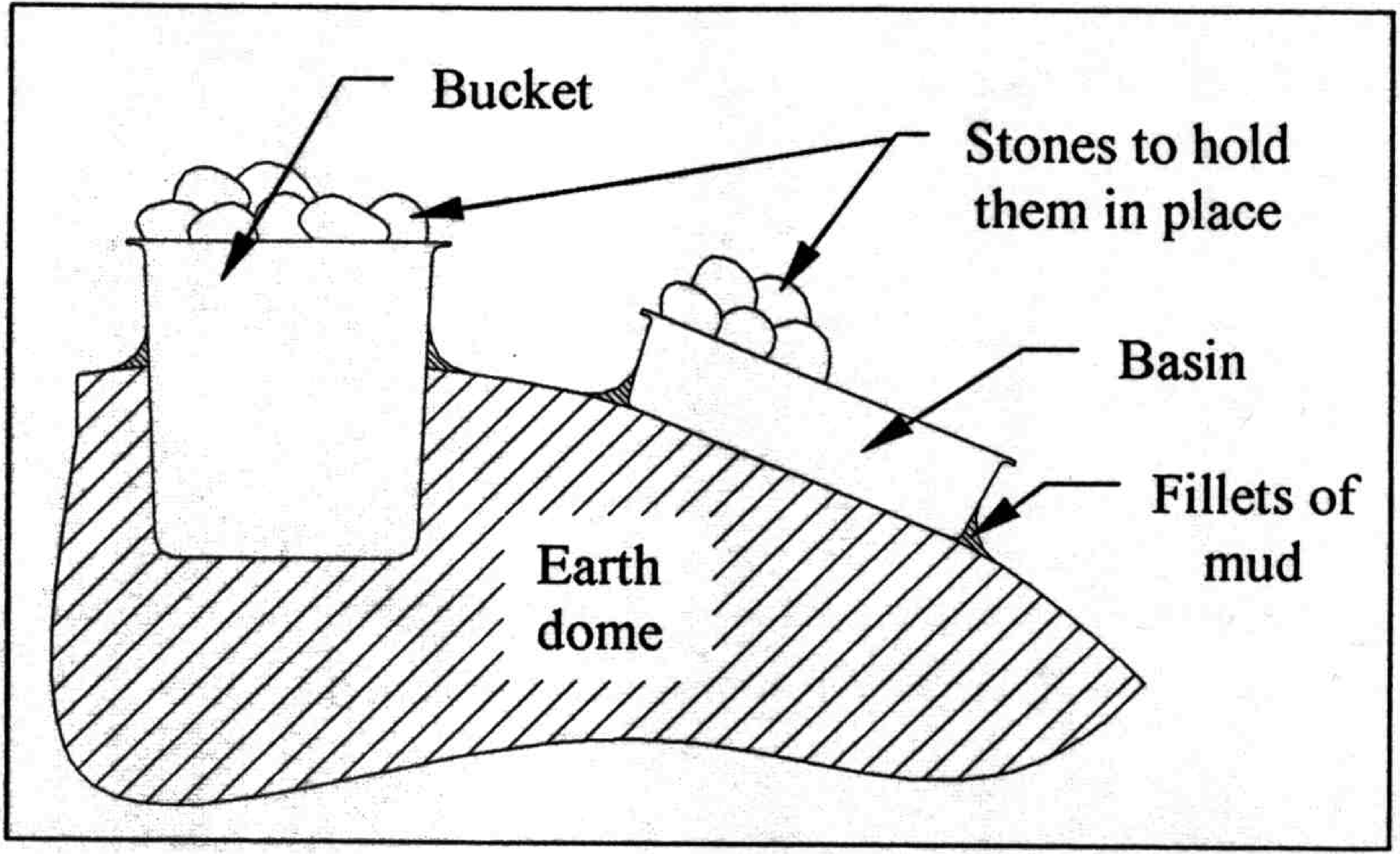
2200
Platform
Pump
Dome
Water inlet

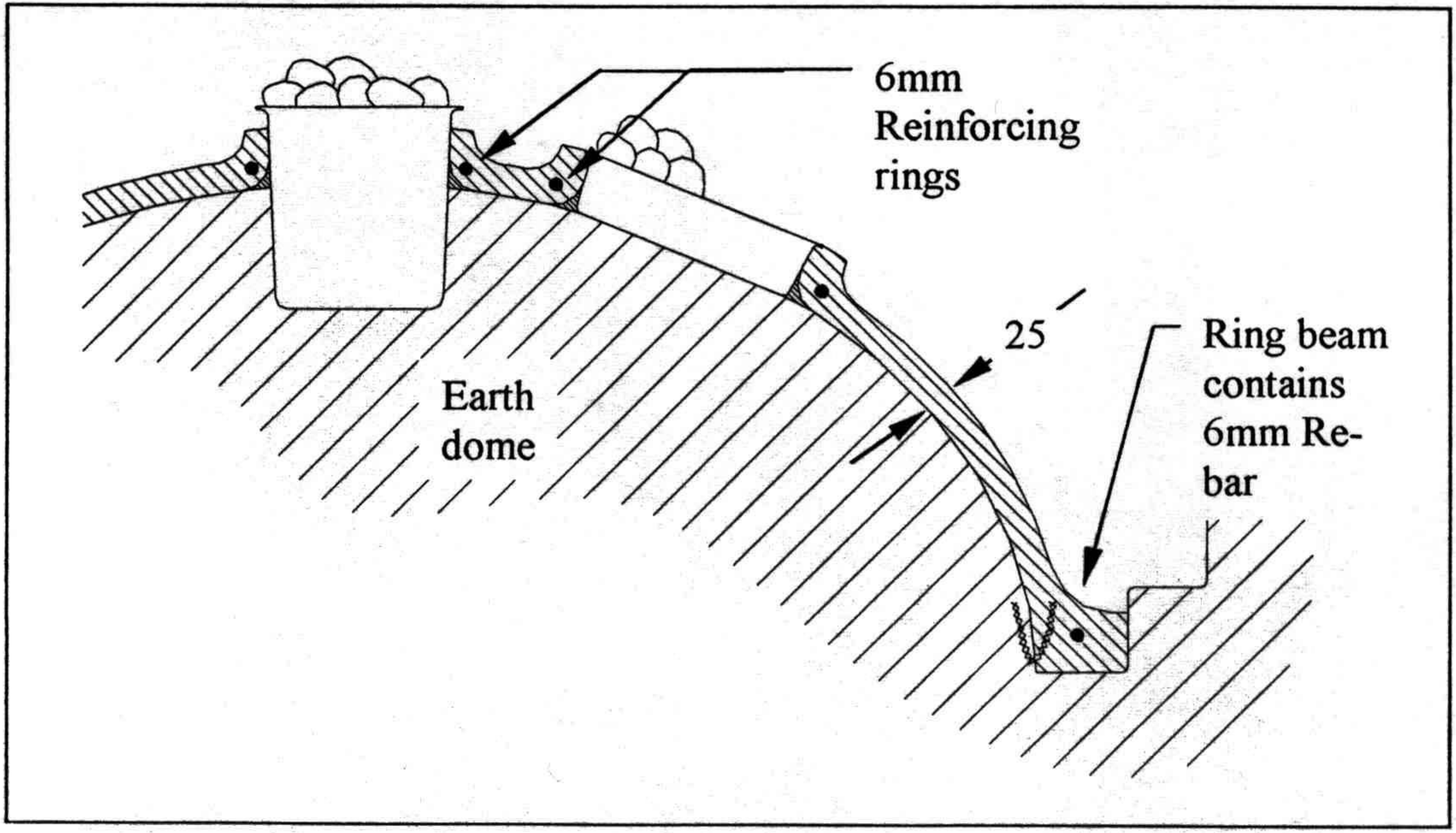


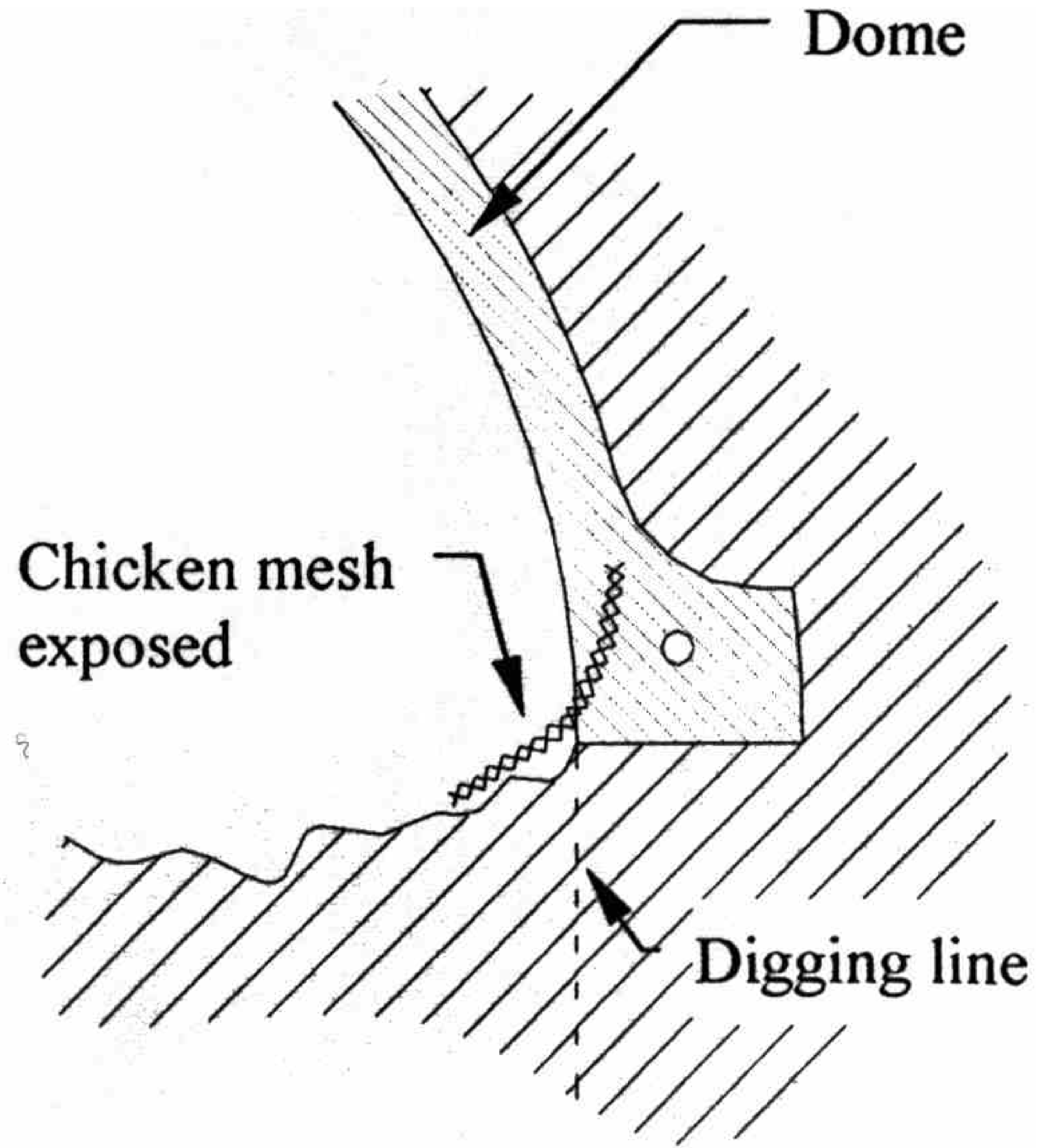
1.1 m
1.1 m
Nail
Chain
0.8m
Reinforcing strips
Plywood
Cutaway
Alternative nail position

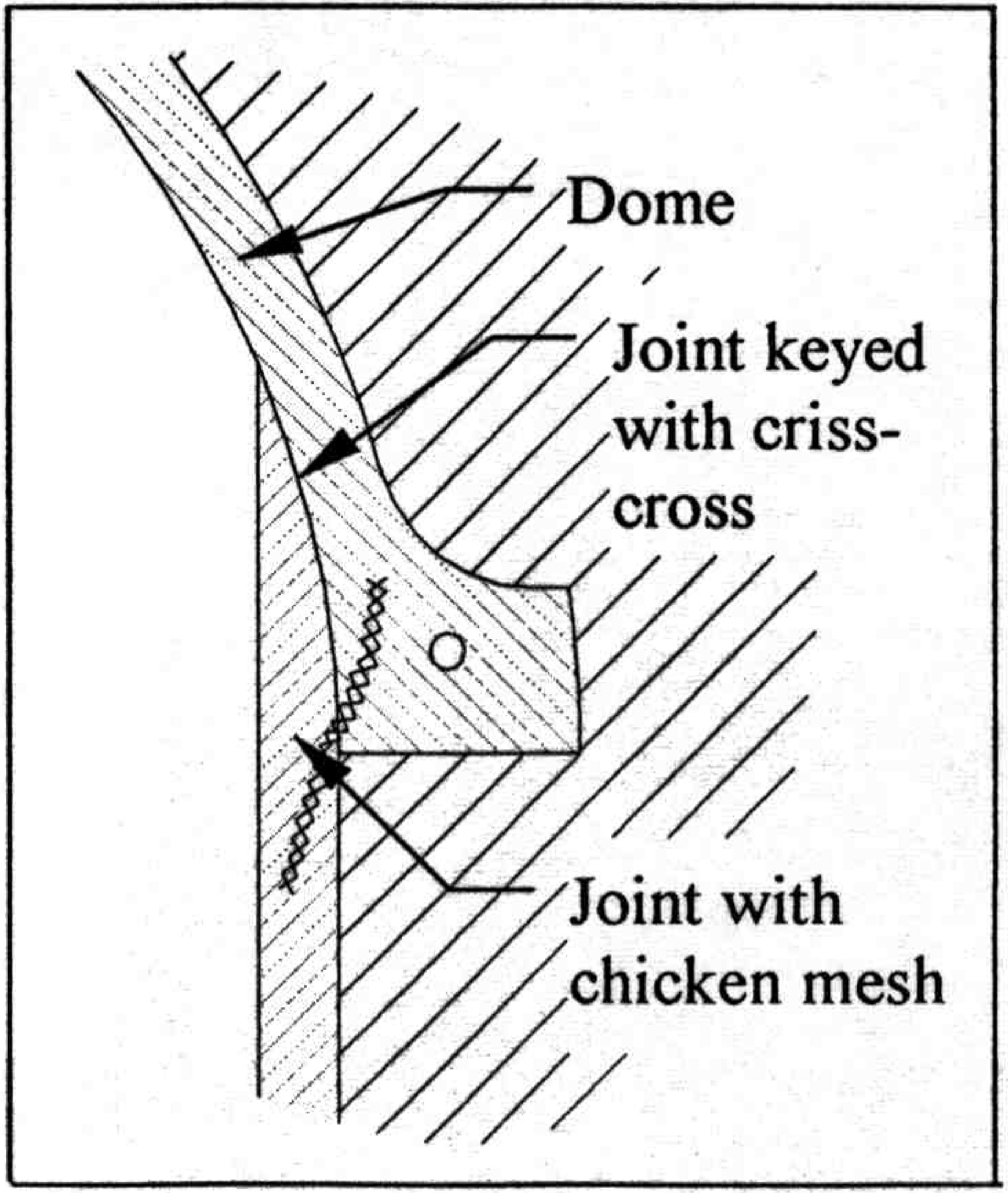

Spirit level
Dome template
Earth dome

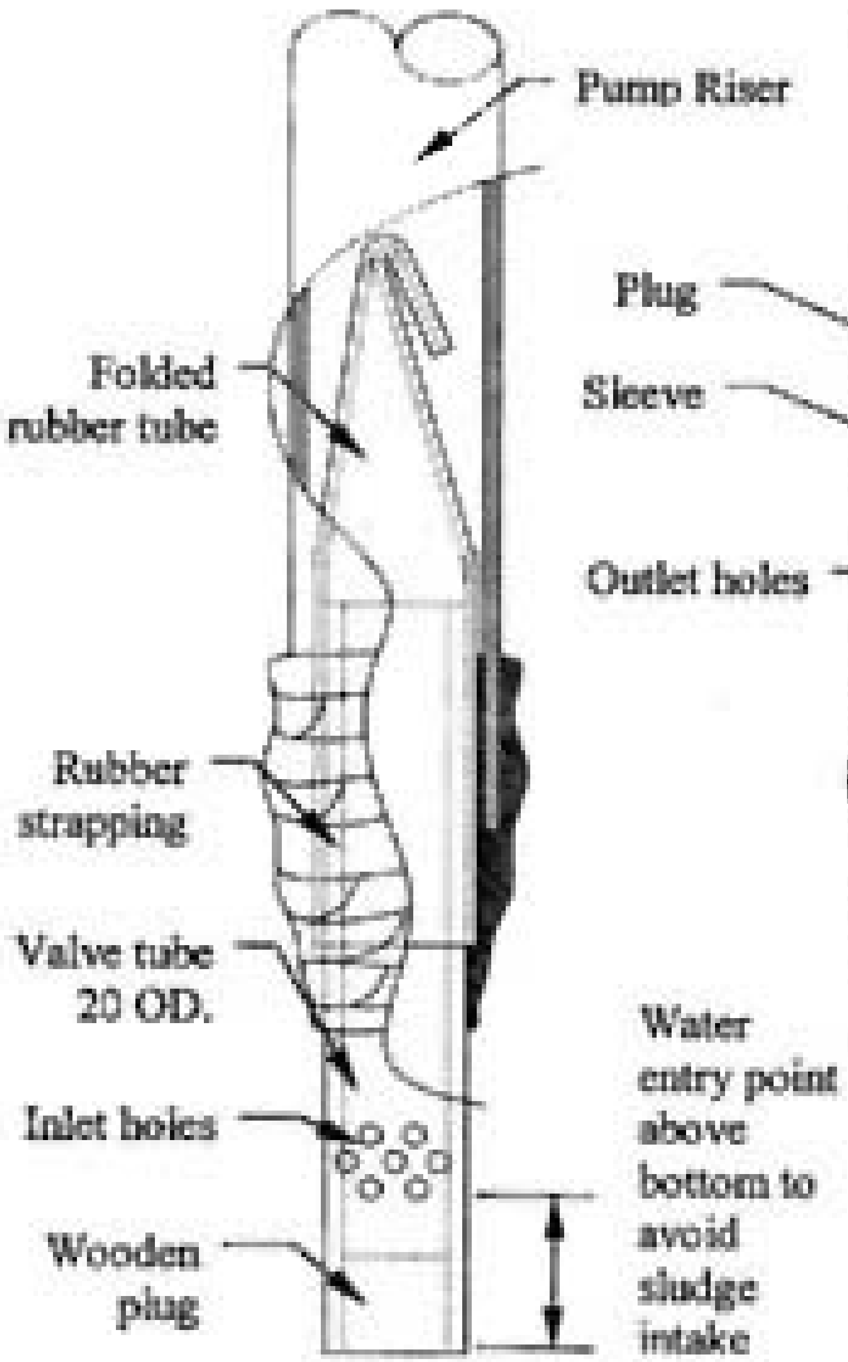

Earth
dome
Cutaway base
of dome
10cm
Chicken mesh
strip
Earth
dome
Crisscross
face
Backfilled with
mud


Bucket
Stones to hold them in place
Basin
Fillets of mud
Earth dome

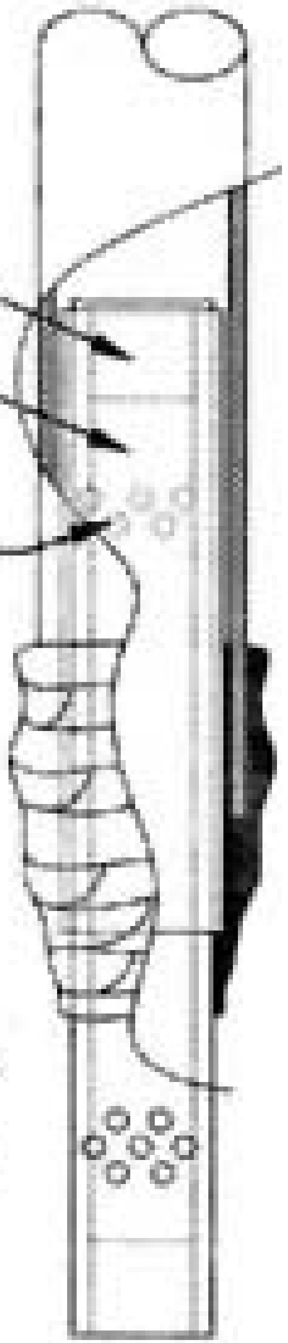

6mm
Reinforcing
rings
25
Ring beam
contains
6mm Re-
bar
Earth
dome


Dome
Chicken mesh
exposed
Digging line

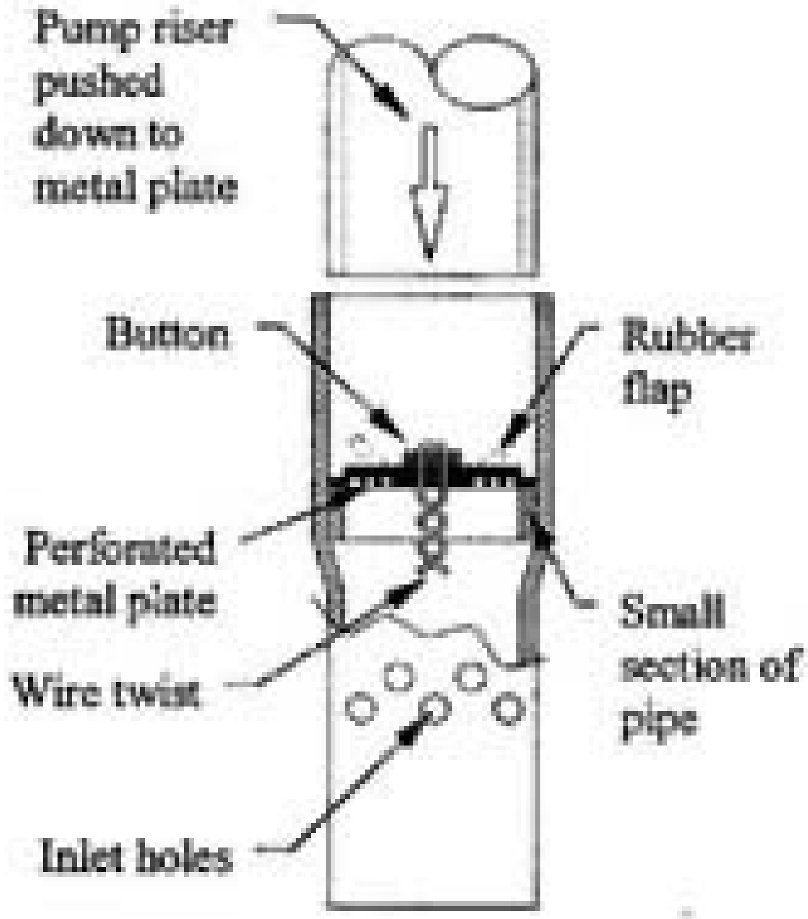

Dome
Joint keyed
with criss-
cross
Joint with
chicken mesh



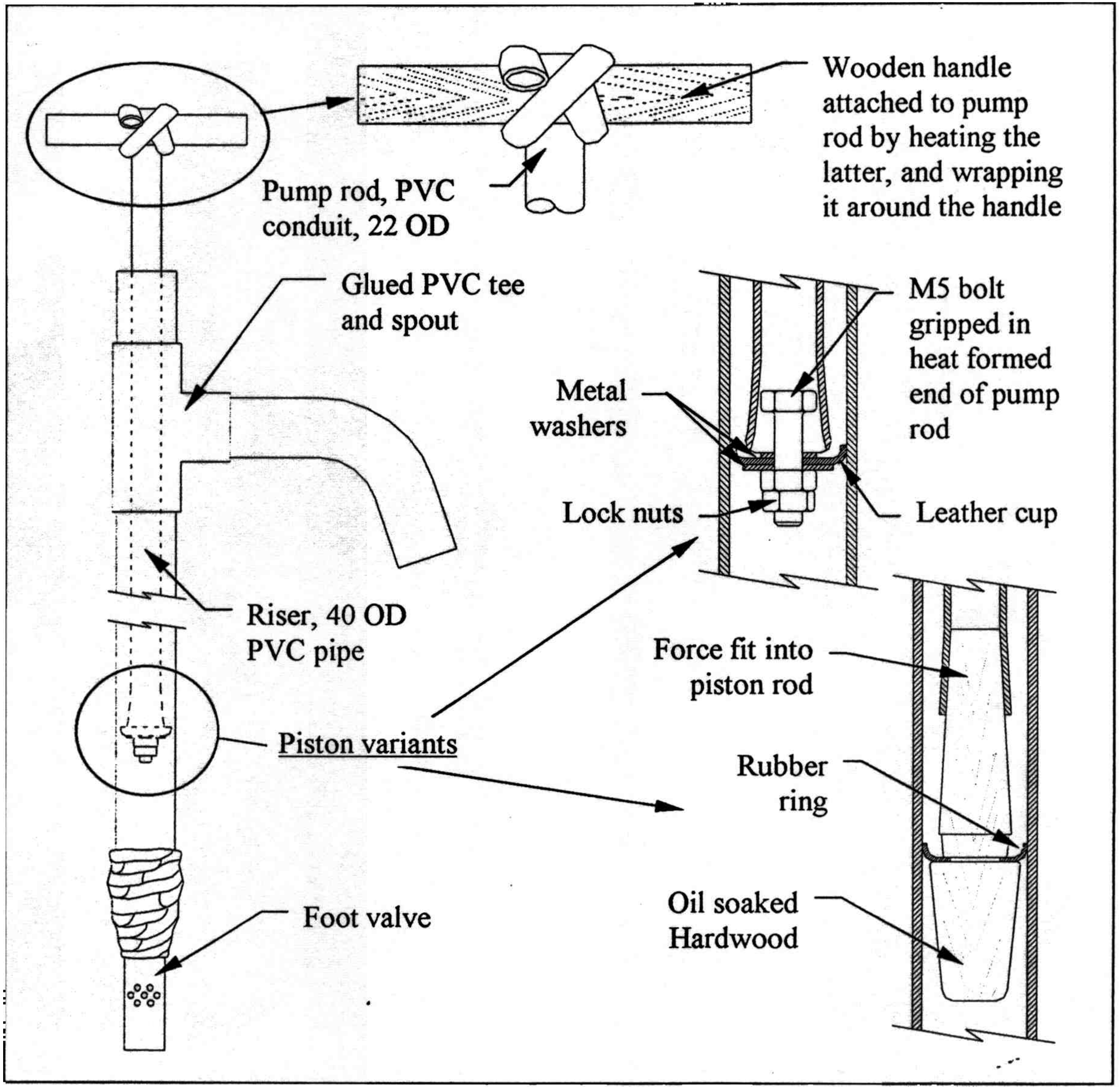
Collapsible rubber tube valve



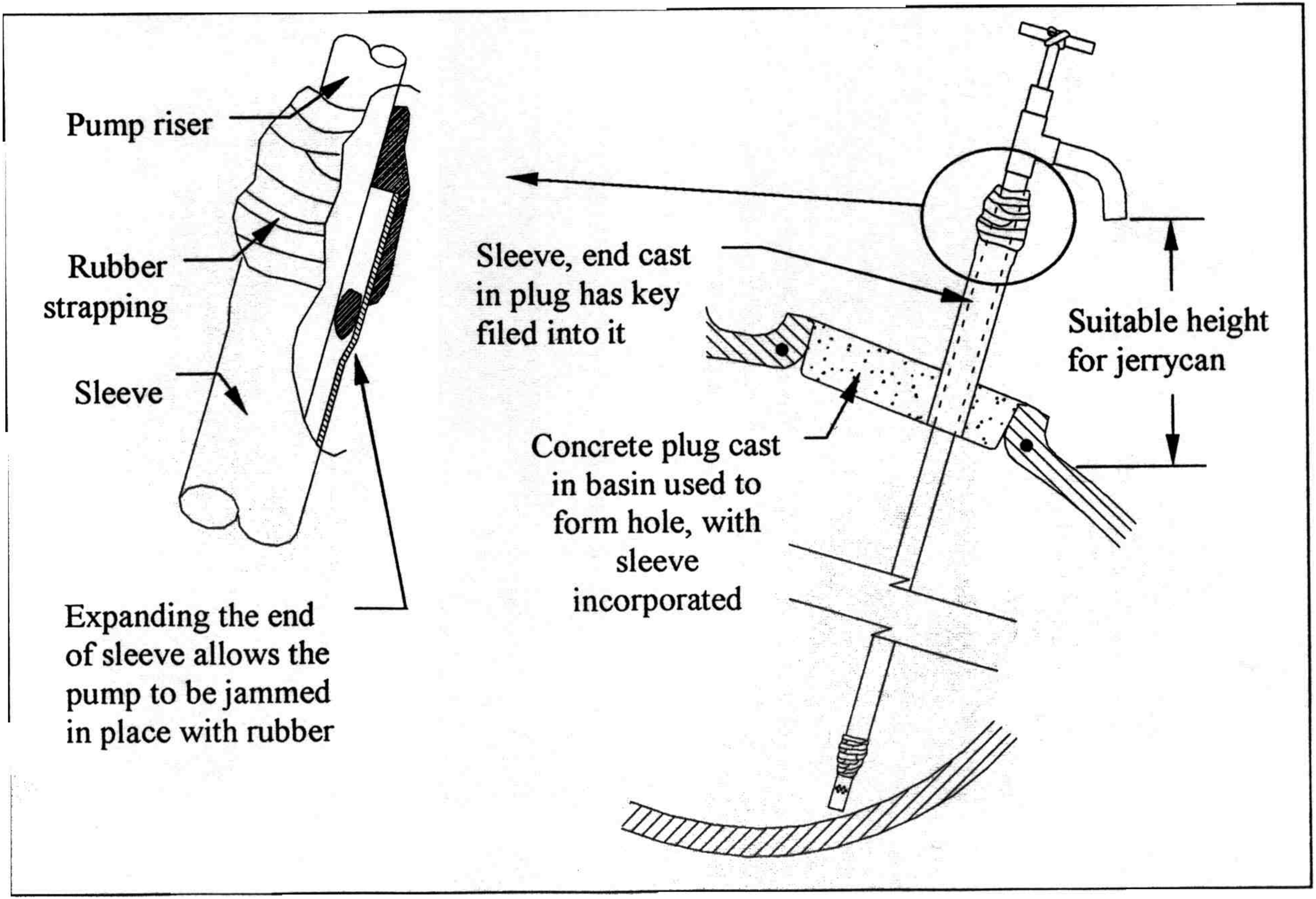
Sleeve valve

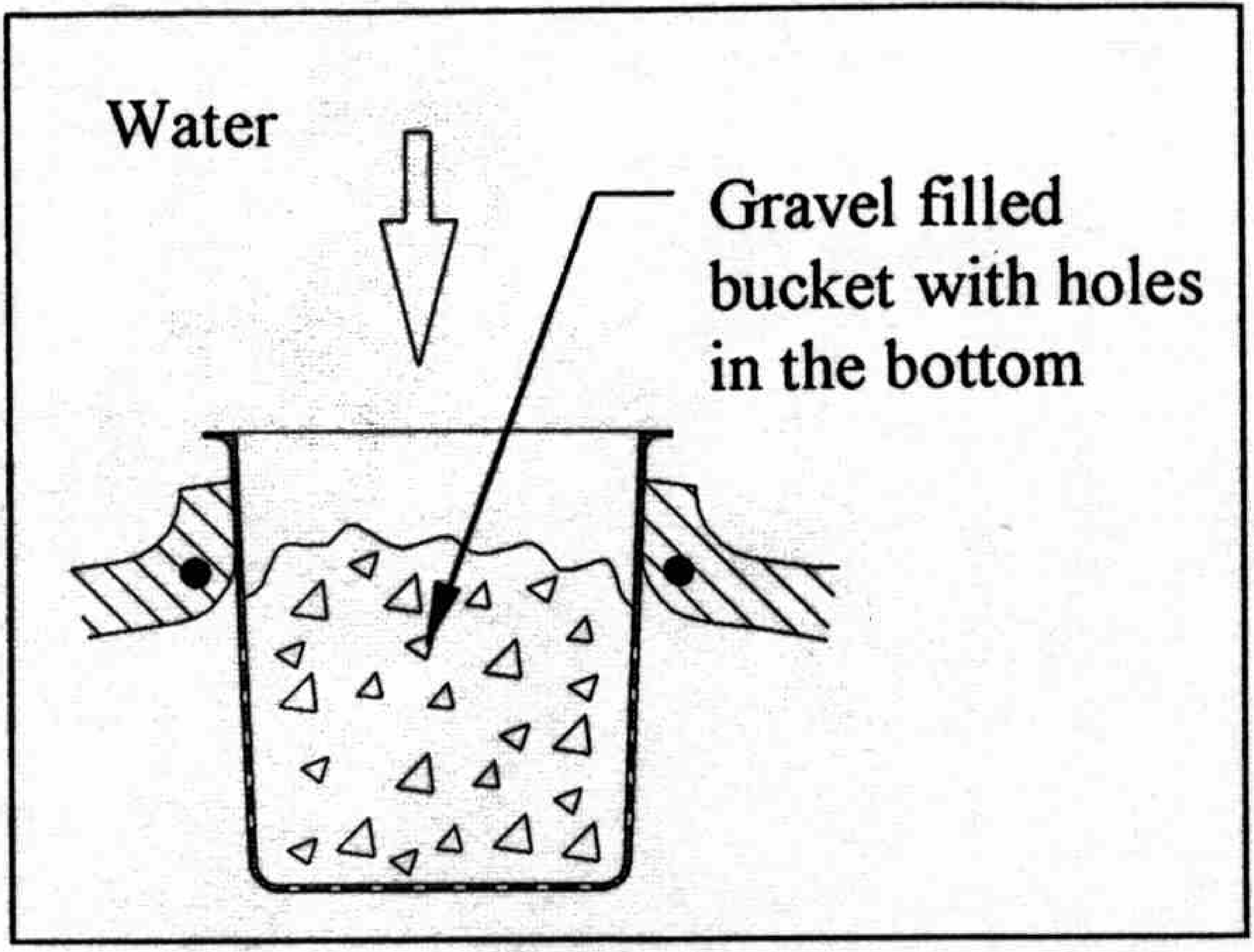


Butterfly valve

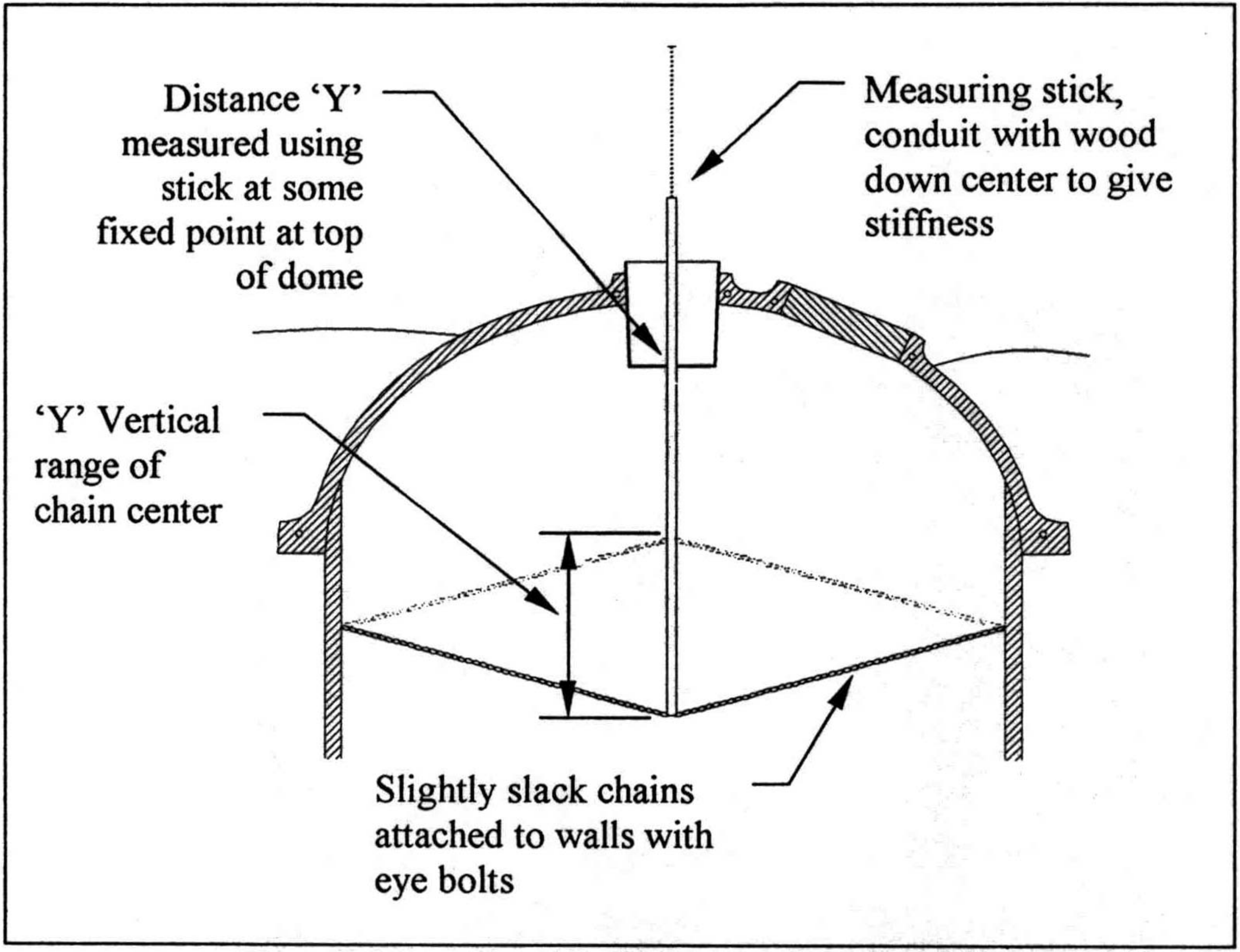


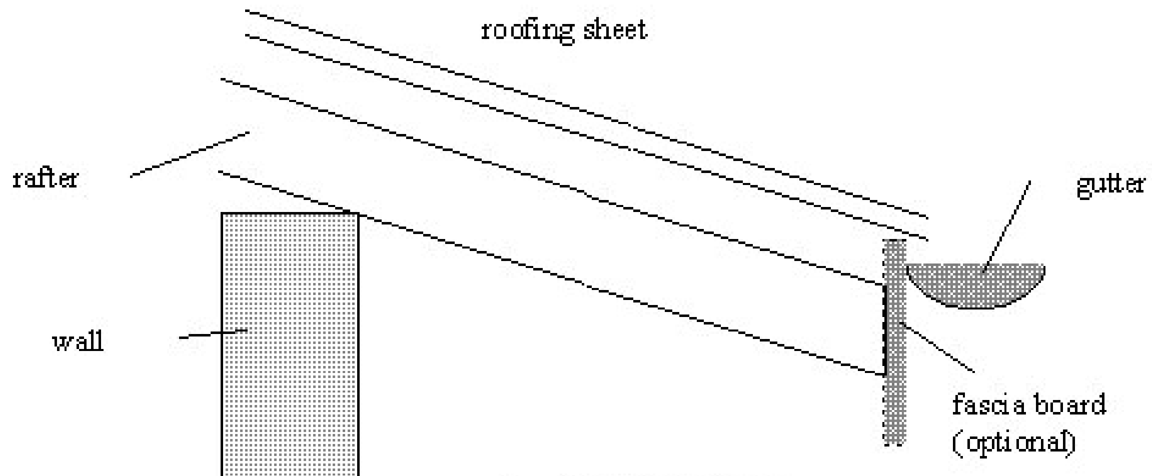
Wooden handle
attached to pump
rod by heating the
latter, and wrapping
it around the handle
Pump rod, PVC
conduit, 22 OD
Glued PVC tee
and spout
M5 bolt
gripped in
heat formed
end of pump
rod
Metal
washers
Lock nuts
Leather cup
Riser, 40 OD
PVC pipe
Force fit into
piston rod
Piston variants
Rubber
ring
Oil soaked
Hardwood
Foot valve


Pump riser
Rubber strapping
Sleeve
Expanding the end of sleeve allows the pump to be jammed in place with rubber
Sleeve, end cast in plug has key filed into it
Concrete plug cast in basin used to form hole, with sleeve incorporated
Suitable height for jerrycan

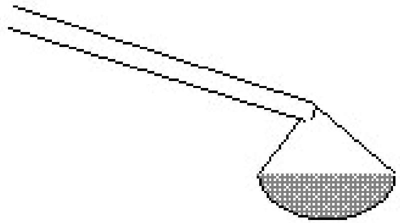


Water
Gravel filled
bucket with holes
in the bottom

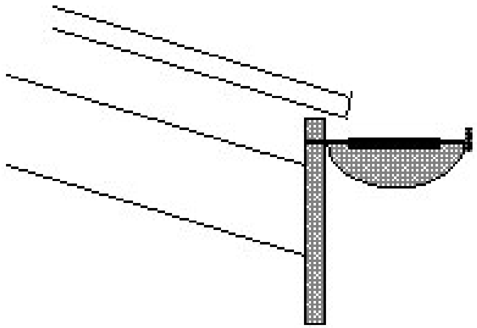

Distance 'Y' measured using stick at some fixed point at top of dome
Measuring stick, conduit with wood down center to give stiffness
'Y' Vertical range of chain center
Slightly slack chains attached to walls with eye bolts



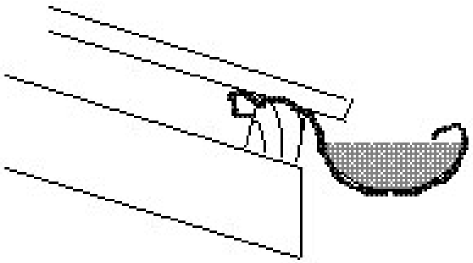
4a - Typical roof edge



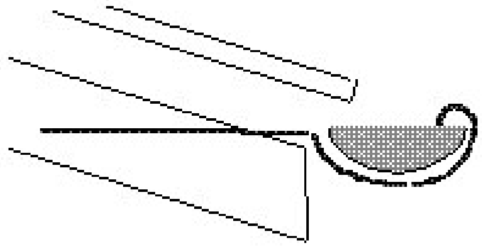
4b - Wiring to roof edge or spar



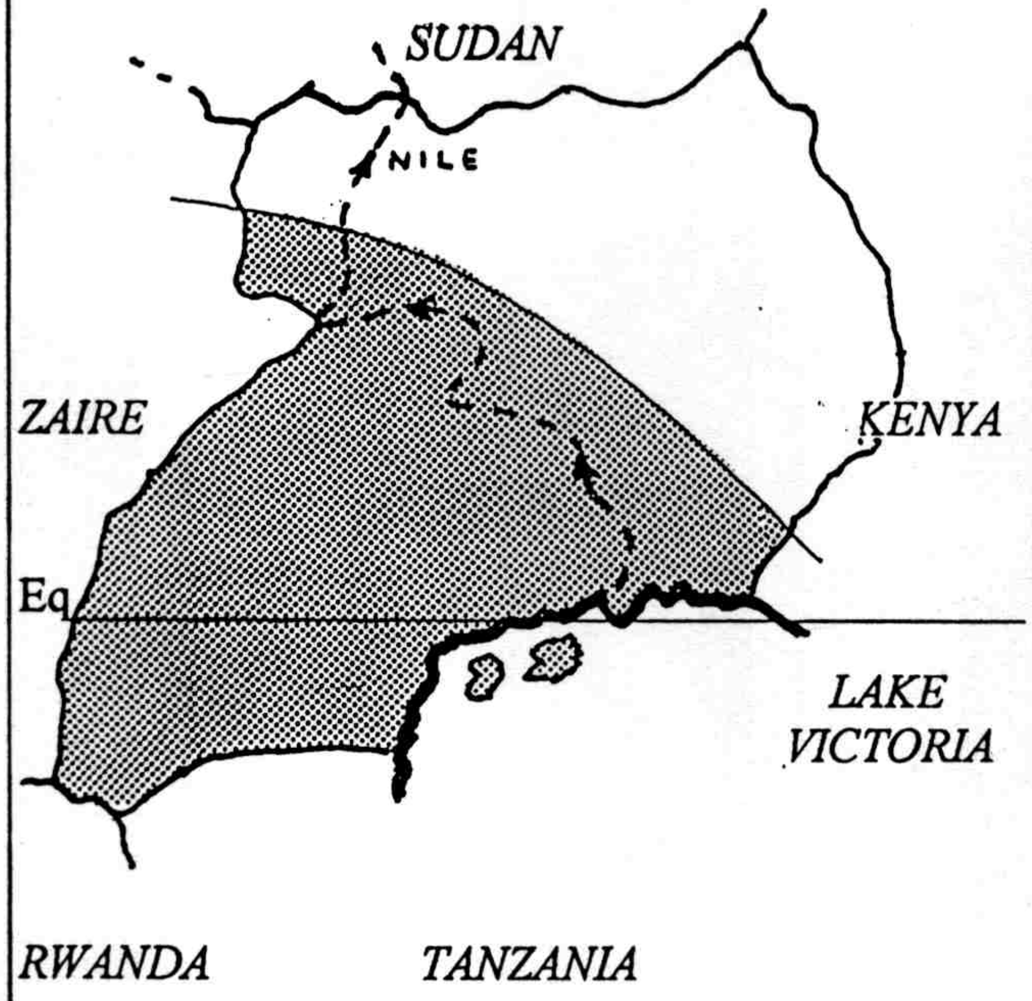
4c - Nailing to fascia board - plastic or metal tube inside gutter prevents the nail from collapsing it



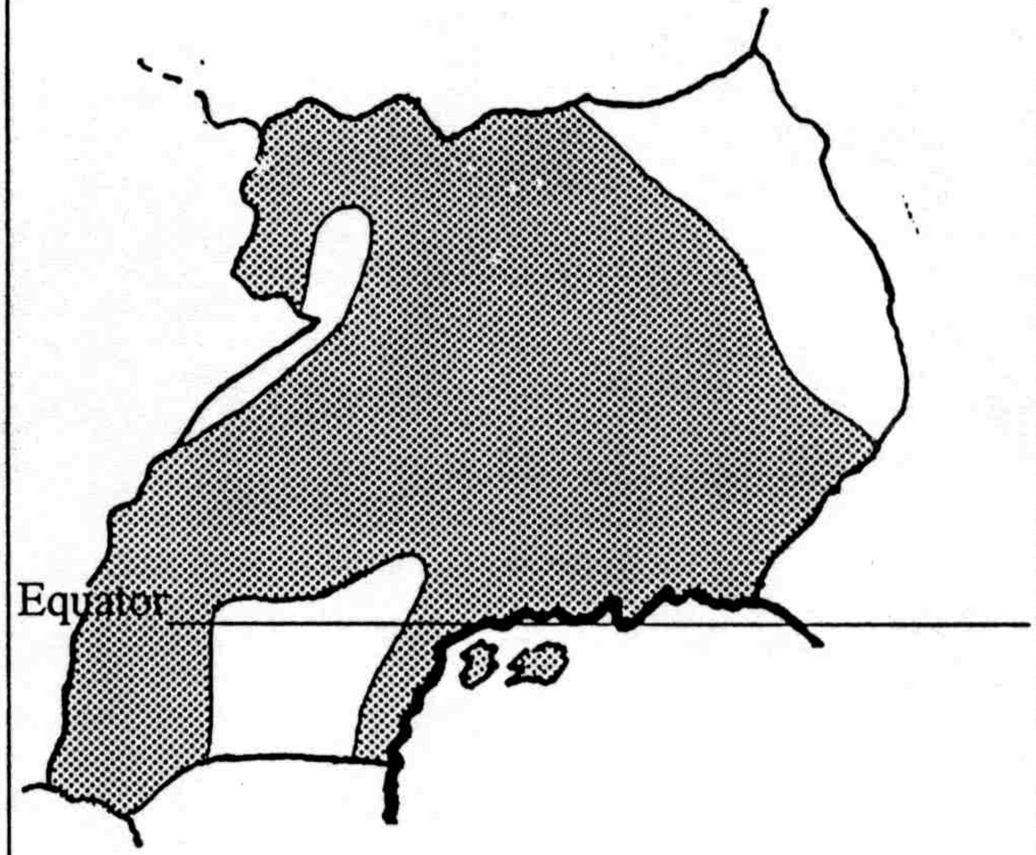
4d - Shaped bar connected to purlin



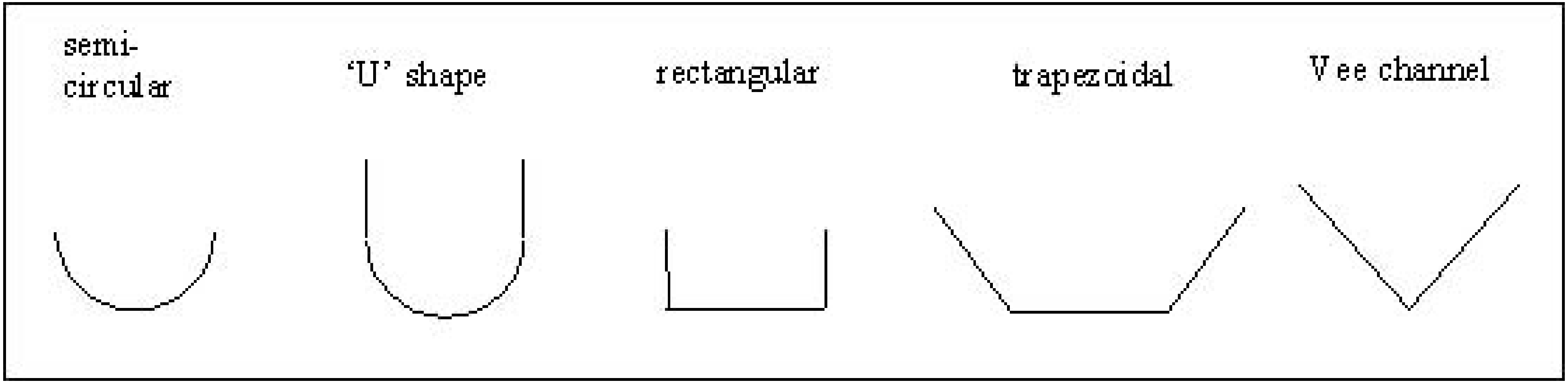
4e - Strip connected to end of rafter

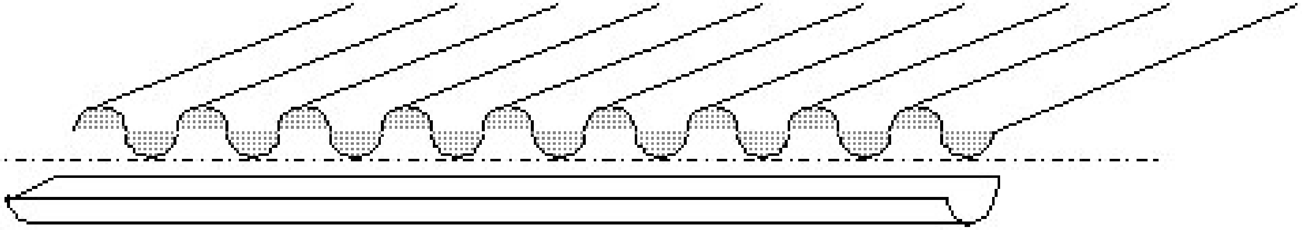


Bimodal Rainfall Area

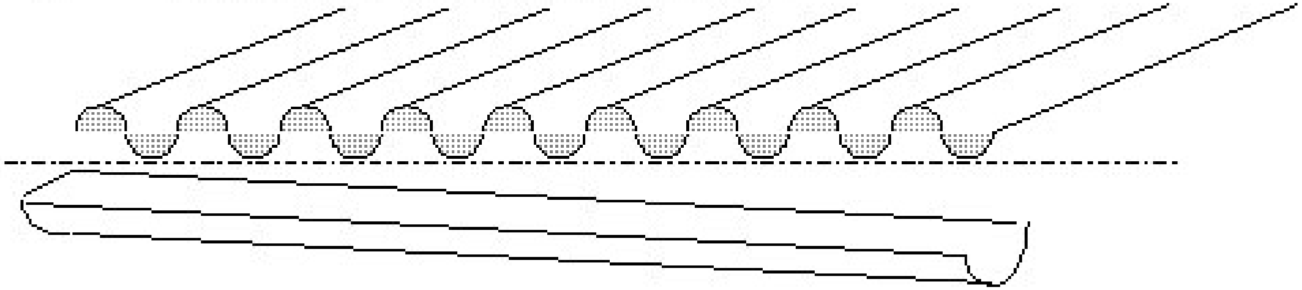


Area of Rainfall over 1000 mm

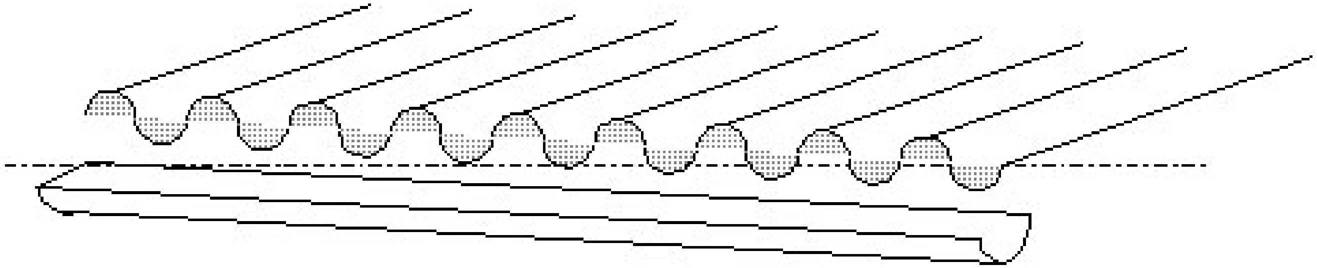

semi-circular
'U' shape
rectangular
trapezoidal
Vee channel



2a Flat gutter (no slope) under flat roof edge

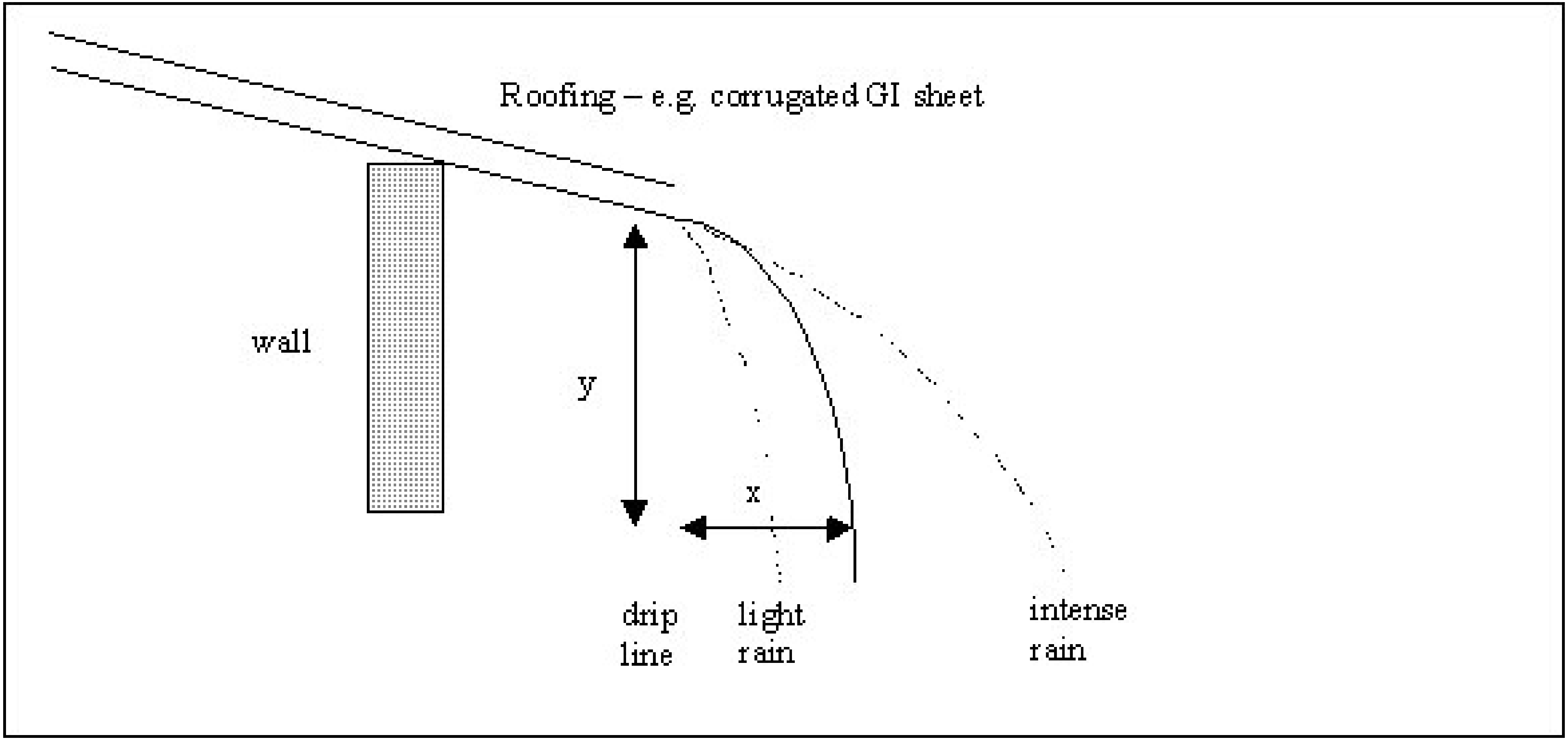


2b Uniformly sloping gutter under flat roof edge



2c Both gutter and roof edge sloping uniformly

2d Gutter slope increases towards the discharge end


Roofing – e.g. corrugated GI sheet
wall
y
x
drip line
light rain
intense rain

Report A1 –Current Technology for Storing Domestic Rainwater (Part 1)

The work in this report forms the basis for the current DTU roofwater harvesting Web Site. The site can be found at:

http://www.eng.warwick.ac.uk/DTU/rainwaterharvesting/index.html

The work in this document is on-going and will be added to as the programme progresses. The aim is to collect examples of DRWH practice from around the world and to provide a useful resource for practitioners of DRWH. Many of the graphics shown in the Web Site are not shown in this document due to electronic storage requirement limitations. The report is in two parts so that the document remains manageable. This is Part 1.

Contents of report (Part 1)

Current Technology for Storing Domestic Rainwater

1. An Introduction to Domestic Roofwater Harvesting

A sufficient, clean drinking water supply is essential to life. Millions of people throughout the world still do not have access to this basic necessity. After decades of work by governments and organisations to bring potable water to the poorer people of the world, the situation is still dire. The reasons are many and varied. The poor of the world cannot afford the capital intensive and technically complex traditional water supply systems which are widely promoted by governments and agencies throughout the world.

Roof-water or rainwater harvesting (RWH) is an option which has been adopted in many areas of the world where conventional water supply systems have failed to meet the needs of the people. It is a technique which has been used since antiquity. Examples of RWH systems can be found in all the great civilisations throughout history. The technology can be as simple or as complex as required. In many African countries this is often as simple as placing a small container under the eaves of the roof to collect falling water during a storm. One 20 litre container of clean water captured from the roof can save a walk of many kilometres, in some cases, to the nearest clean water source. In the industrialised countries of the world, sophisticated RWH systems have been developed with the aim of reducing water bills or to meet the needs of remote communities or individual households in arid regions. Traditionally, in Uganda rainwater is also collected from trees, using banana leaves or stems as temporary gutters; up to 200 litres may be collected from a large tree in a single storm. Many individuals and groups have taken the initiative and developed a wide variety of different RWH systems throughout the world.

It is worth bearing in mind, however, that Domestic Rainwater Harvesting (DRWH) is not the definitive answer to household water problems. There is a complex set of inter-related circumstances which have to be considered when choosing the appropriate water source. Cost, climate, technology, hydrology, social and political elements all play a role in the eventual choice of water supply scheme which is adopted for a given situation. RWH is only one possible choice, but one which is often overlooked by planners, engineers and builders. The reason that RWH is rarely considered is often due to lack of information – both technical and otherwise. In many areas where RWH has been introduced as part of a wider drinking water supply programme, it was at first unpopular, simply because little was known about the technology by the eneficiaries. In most of these cases the technology has quickly gained popularity as the user realises the benefits of a clean, reliable water source at the home. In many cases RWH has been introduced as part of an integrated water supply system, where the town supply is unreliable or where local water sources dry up for a part of the year, but is also often used as the sole water source for a community or household. It is a technology which is flexible and adaptable to a very wide variety of conditions, being used in the richest and the poorest societies on our planet, and in the wettest and the driest regions of the world.

The aim of this web site is to enable readers to view a wide variety of these systems, with the aim of providing interested parties with a selection of possible technical solutions to their water problems. We try to provide guidelines for the sizing of RWH systems, a brief overview of the components of a RWHS, a critique for examining the systems with a mind to their possible application, and look at the cost of the system (or at least the material requirements). We also look at ways in which water quality can be improved and maintained before, during and after storage.

2. Styles of Roofwater Harvesting

User regimes

Rainwater harvesting is used in many different ways. In some parts of the world it is used merely to capture enough water during a storm to save a trip or two to the main water source. In this case only small storage capacity is required, maybe just a few small pots to store enough water for a day or half a day. At the other end of the spectrum we see, in arid areas of the world, systems which have sufficient collection surface area and storage capacity to provide enough water to meet the full needs of the user. Between these two extremes exists a wide variety of different user patterns or regimes. There are many variables that determine these patterns of usage for RWH. Some of these are listed below:

- Rainfall quantity (mm/year) – the total amount of water available to the consumer is a product of the total available rainfall and the collection surface area. There is usually a loss coefficient included to allow for evaporation and other losses. Mean annual rainfall data will tell us how much rain falls in an average year.
- Rainfall pattern - climatic conditions vary widely throughout the world. The type of rainfall pattern, as well as the total rainfall, which prevails will often determine the feasibility of a RWHS. A climate where rain falls regularly throughout the year will mean that the storage requirement is low and hence the system cost will be correspondingly low and vice versa. More detailed rainfall data is required to ascertain the rainfall pattern. The more detailed the data available, the more accurately the system parameters can be defined.
- Collection surface area (m2) - this, where rooftop catchment systems are used, is restricted by the size of the roof of the dwelling. Sometimes other surfaces are used to supplement the rooftop catchment area.
- Storage capacity (m3) - the storage tank is usually the most expensive component of the RWHS and so a careful analysis of storage requirement against cost has to be carried out.
- Daily consumption rate (litres/capita /day or lpcd) - this varies enormously – from 10 – 15 lpcd a day in some parts of Africa to several hundred lpcd in some industrialised countries. This will have obvious impacts on system specification.
- Number of users - again this will greatly influence the requirements.
- Cost – a major factor in any scheme.
- Alternative water sources – where alternative water sources are available, this can make a significant difference to the usage pattern. If there is a groundwater source within walking distance of the dwelling (say within a kilometre or so), then a RWHS that can provide a reliable supply of water at the homestead for the majority of the year, will have a significant impact to lifestyle of the user. Agreed, the user will still have to cart water for the remainder of the year, but for the months when water is available at the dwelling there is a great saving in time and energy. Another possible scenario is where rainwater is stored and used only for drinking and cooking, the higher quality water demands, and a poorer quality water source, which may be near the dwelling, is used for other activities.
- Water management strategy – whatever the conditions, a careful water management strategy is always a prudent measure. In situations where there is a strong reliance on stored rainwater, there is a need to control or manage the amount of water being used so that it does not dry up before expected.

Ideally, we would like to be able to classify the various common user regimes that are adopted. This can help us to develop a nomenclature for dealing with the systems we will look at later. We can simply classify most systems by the amount of 'water security' or 'reliability' afforded by the system. There are four types of user regimes listed below:

Occasional (or opportunist) - water is collected occasionally with a small storage capacity, which allows the user to store enough water for a maximum of, say, one or two days. During the wet season this means that the user will benefit considerably from having such a system and most, if not all, of the user needs will be met during this time. After a day or two of dry weather the user will have to return to using an alternative water source. This type of system is ideally suited to a climate where there is a uniform, or bimodal, rainfall pattern with very few dry days during the year and where an alternative water sources is close at hand.

Intermittent – this type of pattern is one where the requirements of the user are met for a part of the year. A typical scenario is where there is a single long rainy season and, during this time, most or all of the user needs are met. During the dry season an alternative water source has to be used or, as we see in the Sri Lankan case, water is carted/ bowsered in from a nearby river and stored in the RWH tank. Usually, a small or medium size storage vessel is required to bridge the days when there is no rain.

Partial – this type of pattern provides for partial coverage of the water requirements of the user, during the whole of the year. An example of this type of system would be where a family gather rainwater to meet only the high-quality needs, such as drinking or cooking, while other needs, such as bathing and clothes washing, are met by a water source with a lower quality. This could be achieved either in an area with a uniform rainfall pattern and with a small to medium storage capacity or in an area with a single (or two short) wet season(s) and a larger storage capacity to cover the needs during the dry season.

Full – with this type of system the total water demand of the user is met for the whole of the year by rainwater only. This is sometimes the only option available in areas where other sources are unavailable. Sufficient a/ rainfall, b/ collection area, c/ storage capacity is required to meet the needs of the user and a careful feasibility study must be carried out before hand to ensure that conditions are suitable. In areas where there is a bimodal rainfall pattern (i.e. two rainy seasons) this type of system is far more attractive, as the tank will be recharged during both wet seasons. Where there is a single (unimodal) wet season the storage capacity will normally be very large – and therefore expensive. A strict water management strategy is required when such a system is used to ensure that the water is used carefully and will last until the following wet season.

3. Components of a DRWH system

Introduction

Technically, DRWH systems vary in complexity. Some of the traditional Sri Lankan systems are no more that a pot situated under a piece of cloth or plastic sheet tied at its corners to four poles. The cloth captures the water and diverts it through a hole in its centre into the pot. On the other hand, some sophisticated systems used in the industrialised nations, incorporate clever computer management systems, submersible pumps, and links into the grey water and mains domestic plumbing systems.
Somewhere between these two extremes we find the typical DRWH system that is used in a typical developing country scenario. Such a system will usually comprise a collection surface, a roof, a storage tank, and guttering to transport the water from the roof to the storage tank. Other peripheral equipment is sometimes incorporated: first flush systems to divert the dirty water which contains roof debris after prolonged dry periods; filtration equipment and settling chambers to remove debris and contaminants before water enters the storage tank or cistern.

In this section we will look at the various components commonly found in typical DRWH systems. In the Case Studies section we will look at actual systems. In the Case Studies section, where possible, we have tried to look at full systems, showing how the various components interact. In some cases, however, we have been able to show only certain components of the system – usually the tank or cistern as this is the most costly and critical component of the DRWH system, and the area that has attracts most design attention.

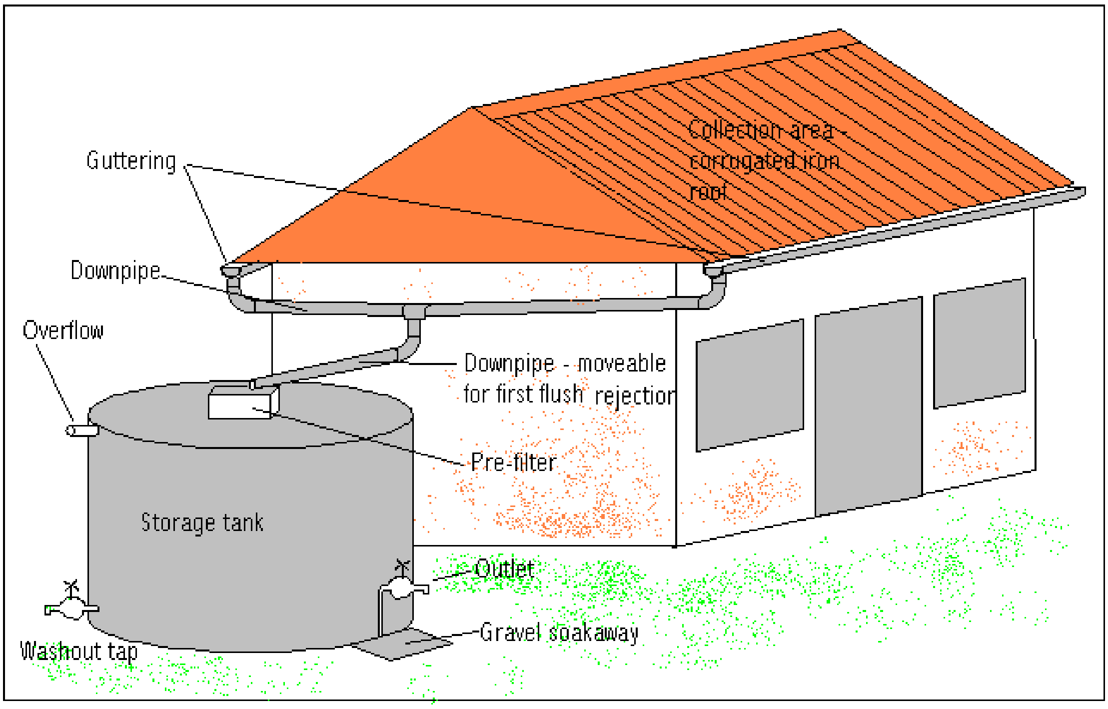


Figure 1 – Typical domestic roofwater harvesting system – showing the main components of the system

Storage tanks and cisterns

The water storage tank usually represents the biggest capital investment element of a DRWH system. It therefore usually requires the most careful design – to provide optimal storage capacity while keeping the cost as low as possible. The catchment area is usually the existing rooftop, and guttering can often be obtained relatively cheaply, or can be manufactured locally.

There are an almost unlimited number of options for storing water. Common vessels used for very small-scale water storage in developing countries include such examples as plastic bowls and buckets, jerrycans, clay or ceramic jars, cement jars, old oil drums, empty food containers, etc.

For storing larger quantities of water the system will usually require a tank or a cistern. For the purpose of this document we will classify the tank as an above-ground storage vessel and the cistern as a below-ground or under-ground storage vessel. These can vary in size from a cubic metre or so (1000 litres) up to hundreds of cubic metres for large projects, but typically up to a maximum of 20 or 30 cubic metres for a domestic system. There is a mind-boggling range of options open to the prospective rainwater harvester, with a wide variety of shapes, materials, sizes and prices on offer. The choice will depend on a number of technical and economic considerations. Some of these are listed below:

- Space availability
- Options available locally
- Local traditions for water storage
- Cost – of purchasing new tank
- Cost – of materials and labour for construction
- Materials and skills available locally
- Ground conditions
- Style of RWH (see link to this section)

One of the main choices will be whether to use a tank or a cistern. Both tanks and cisterns have their advantages and disadvantages. Table 1 summarises the pros and cons of each.

	Tank (above ground)	Cistern (below ground)

Pros	Above ground structure allows for easy inspection for cracks or leakage Many existing designs to choose from Can be easily purchased ‘off-the-shelf’ in most market centres Can be manufactured from a wide variety of materials Easy to construct for traditional materials Water extraction can be by gravity in many cases Can be raised above ground level to increase water pressure	generally cheaper more difficult to empty by leaving tap on require little or no space above ground unobtrusive surrounding ground gives support allowing lower wall thickness.
Cons	Require space Generally more expensive More easily damaged Prone to attack from weather Failure can be dangerous	water extraction is more problematic – often requiring a pump leaks or failures are more difficult to detect contamination of the tank from groundwater is more common tree roots can damage the structure there is danger to children and small animals if tank cover is left off flotation of the cistern may occur if groundwater level is high and cistern is empty heavy vehicles driving over a cistern can cause damage

Table 1. Pros and Cons of Tanks and Cisterns

Much work has been carried out on the development of the ideal tank for DRWH. The Case Studies section of this Web Site show a wide variety of tanks that have been built in many countries throughout the world.

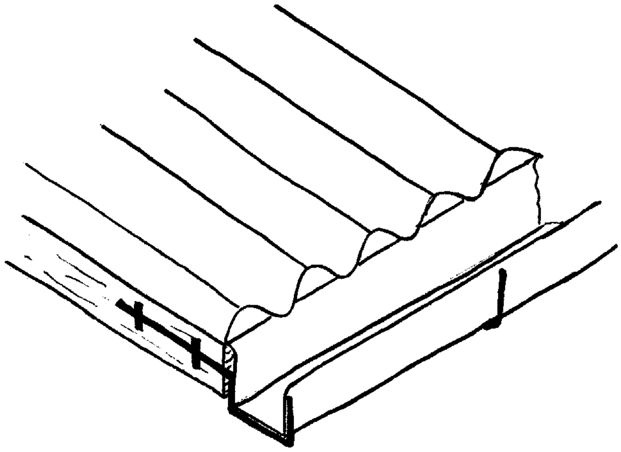
Collection surfaces

For domestic rainwater harvesting the most common surface for collection of water is the roof of the dwelling. Many other surfaces can and are used: courtyards, threshing areas, paved walking areas, plastic sheeting, trees, etc. Most dwellings, however, have a roof. The style, construction and material of the roof affect its suitability as a collection surface for water. Typical materials for roofing include corrugated iron sheet, asbestos sheet; tiles (a wide variety is found), slate, and thatch (from a variety of organic materials).

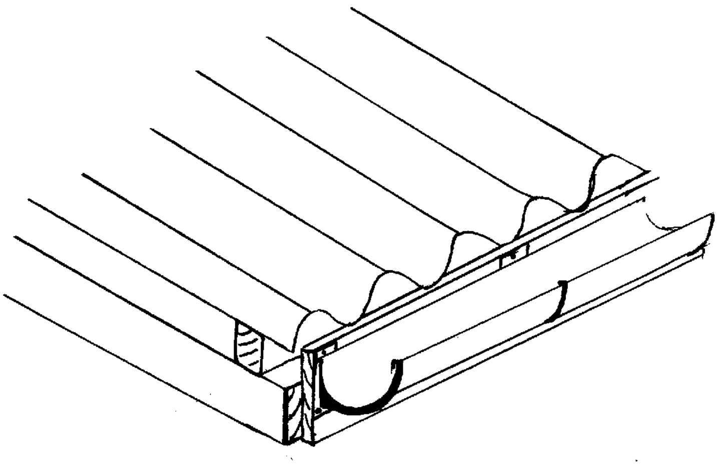
Guttering

Guttering is used to transport rainwater from the roof to the storage vessel. Guttering comes in a wide variety of shapes and forms, ranging from the factory made PVC type to home made guttering using bamboo or folded metal sheet. Guttering is usually fixed to the building just below the roof and catches the water as it falls from the roof. For a detailed analysis of the performance of various types of guttering see the DTU working paper titled ‘Guttering Design for Rainwater Harvesting – with special reference to conditions in Uganda’.

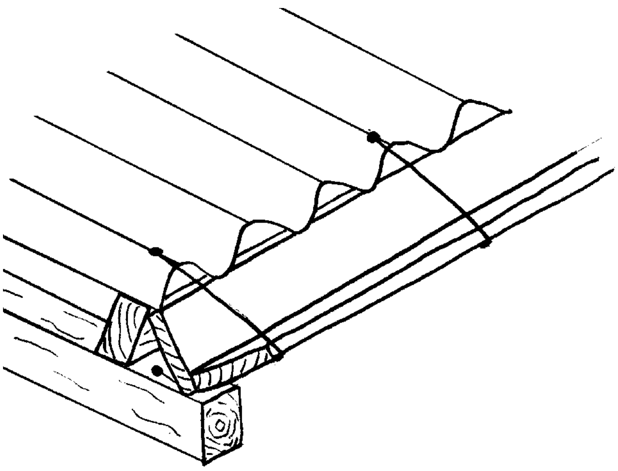
Below are shown some of the common types of guttering and fixings.



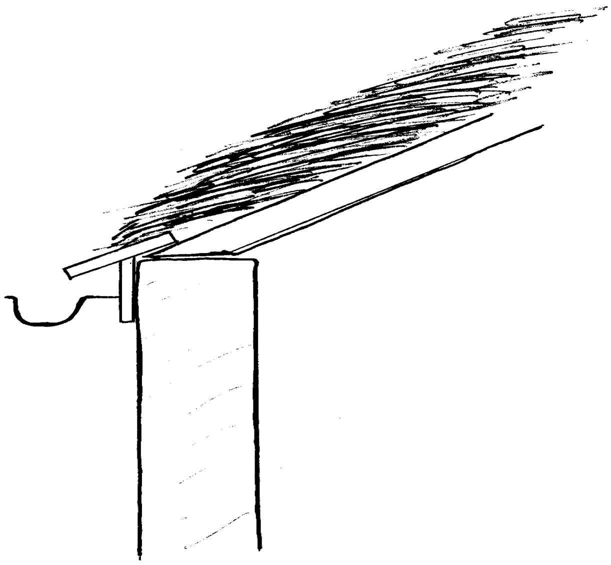
2a - rectangular section guttering fixed to rafter



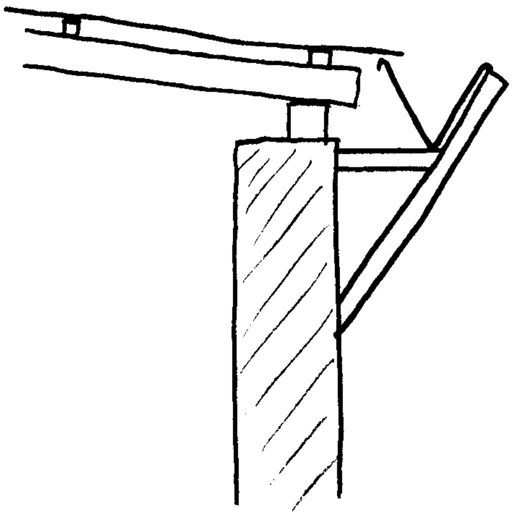
2b - semi circular trough gutter fixed to facia board



2c - timber V-shaped trough fixed with wire to rafter



2d - one configuration for guttering with thatch roof



2e - V-shaped gutter with wall mount

Figure 2 – a variety of guttering types showing possible fixings

Manufacturing low-cost gutters. Factory made gutters are usually expensive and beyond the reach of the poor of developing countries. They are seldom used for very low-cost systems. The alternative is usually to manufacture gutters from materials that can be found cheaply in the locality. There are a number of techniques that have been developed to help meet this demand; one such technique is described below.

V- shaped gutters from galvanised steel sheet can be simply made by cutting and folding flat galvanised steel sheet. Such sheet is readily available in most market centres and can be worked with tools that are commonly found in a modestly equipped workshop. One simple technique is to clamp the cut sheet between two lengths of straight timber and then to fold the sheet along the edge of the wood. A strengthening edge can be made be folding the sheet through 90o and then completing the edge
with a hammer on a hard flat surface. The better the grade of steel sheet that is used, the more durable and hard-wearing the product. Fitting a downpipe to V-shaped guttering can be problematic and the V-shaped guttering will often be continued to the tank rather than changing to the customary circular pipe section downpipe. Methods for fixing gutters are shown in figure 4.

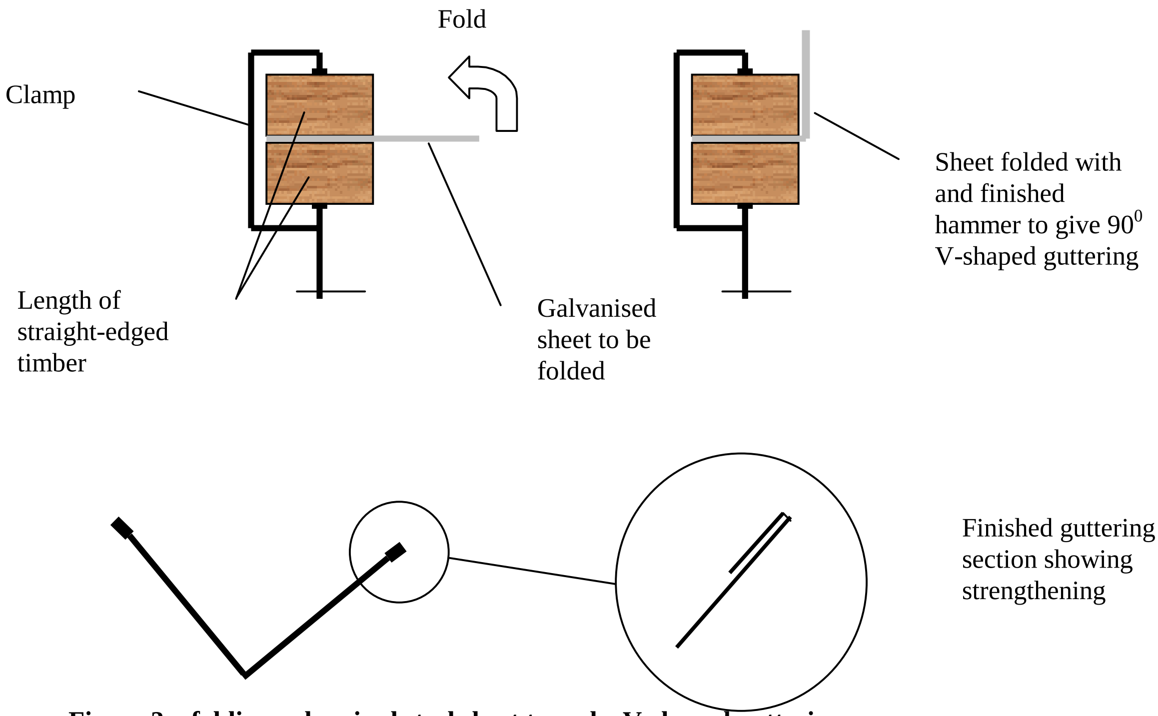


Figure 3 – folding galvanised steel sheet to make V-shaped guttering

Rectangular section gutters can be made in a similar way. It is somewhat easier to fit downpipes to rectangular section guttering.

First flush systems

Variations in Rainwater Quality from Roof Catchment

The quality of rainwater from a tile and a galvanised-iron type roof catchment were analysed over a period of 5 months. Examination of staggered 1 litre samples collected during a rainfall event showed that the concentrations of various pollutants were high in the first litre but decreased in subsequent samples with few exceptions. Faecal coliform and total coliform counts ranged from 8-13 (tile roof) and 4-8 (iron roof) to 41-75 (tile roof) and 25-63 (iron roof) colonies per 100 ml, respectively. However, no faecal coliforms were detected in the fourth and fifth litre samples from both roofs. The pH of rainwater collected from the open was acidic but increased slightly after falling on the roofs. The average zinc concentrations in the run-off from the galvanised-iron roof was about 5-fold higher compared to the tile roof, indicating leaching action but was well below the WHO limits for drinking water quality. Lead concentrations remained consistently high in all samples collected and exceeded the WHO guidelines by a factor of 3.5. For the roof area studied, a ' foul flush ' volume of 5l. would be the minimum to safeguard against microbiological contamination but the high metals content in the water indicate the need for some form of treatment. Rainfall intensity and the number of dry days preceding a rainfall event significantly affect the quality of run-off water from the catchment systems.

Source: Yaziz, M.I. Gunting, H. Sapari, N.,Ghazali, A.W.
Pertanian Malaysia Univ. Serdang, Dept. of Environmental Sciences.
Citation: Water Research WATRAG Vol. 23, No. 6, p 761-765, June 1989. 1 fig, 5 tab, 3 ref.

Debris, dirt and dust will collect on the roof of a building or other collection area. When the first rains arrive, this unwanted matter will be washed into the tank. This will cause contamination of the water and the quality will be reduced. Many RWH systems therefore incorporate a system for diverting this ‘first flush’ water so that it doesn’t enter the tank.

There are a number of simple systems which are commonly used and also a number other, slightly more complex, arrangements. The simpler ideas are based on a simple manually operated arrangement whereby the inlet pipe is moved away from the tank inlet and then replaced again once the initial first flush has been diverted.

This method has obvious drawbacks in that there has to be a person present who will remember to move the pipe. Slightly more sophisticated methods include arrangements such as those shown in Figure 4 below, where the stopper in the inlet chamber can be removed to allow the first flush to be diverted.

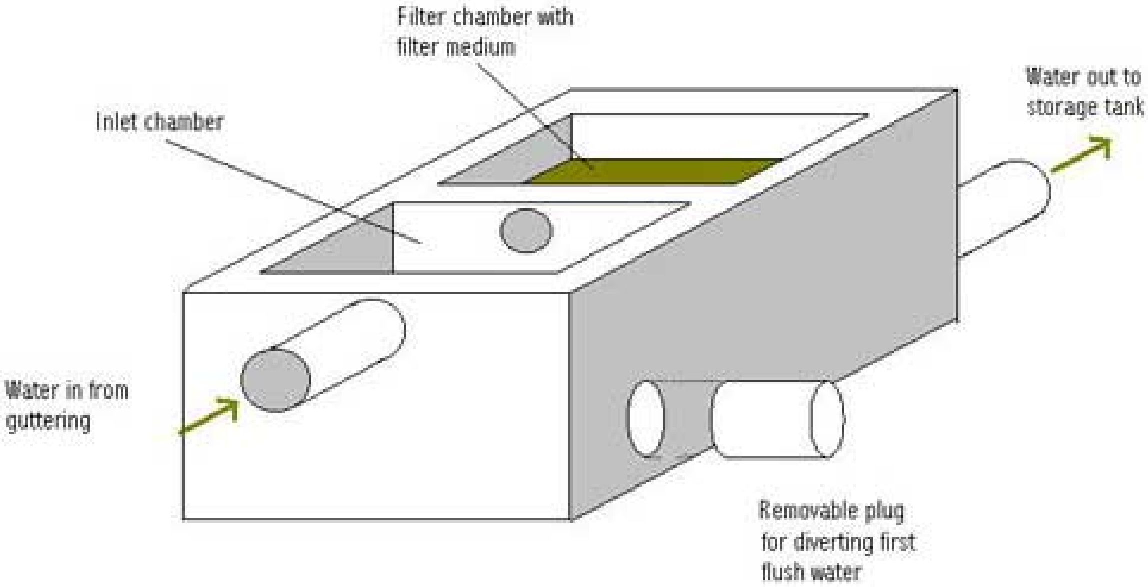


Figure 4 – First flush device using removable stopper in bottom of inlet chamber (above) and using diversion pipe (below)

Other systems use tipping gutters to achieve the same purpose. The most common system (as shown in Figure 5 below) uses a bucket which accepts the first flush and the weight of this water off-balances a tipping gutter which then diverts the water back into the tank. The bucket then empties slowly through a small-bore pipe and automatically resets. The process will repeat itself from time to time if the rain continues to fall, which can be a problem where water is really at a premium. In this case a tap can be fitted to the bucket and will be operated manually. The quantity of water that is flushed is dependent on the force required to lift the guttering. This can be adjusted to suit the needs of the user.

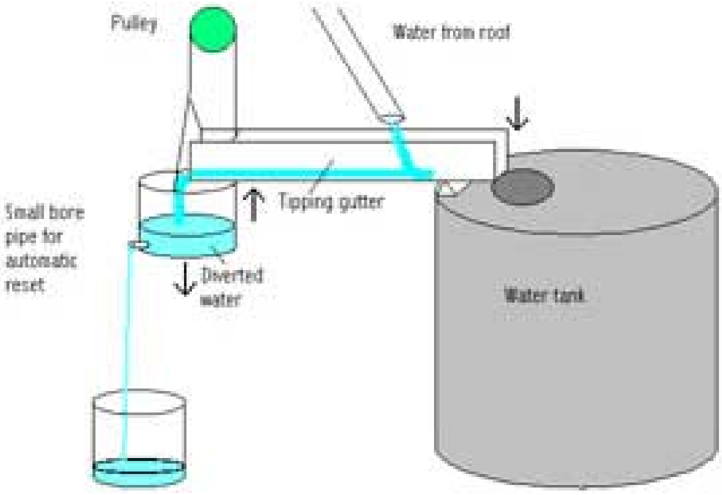


Figure 5 – the tipping gutter first flush system

Another system that is used relies on a floating ball that forms a seal once sufficient water has been diverted (see Figure 6 below). The seal is usually made as the balls rises into the apex of an inverted cone. The ball seals the top of the 'waste' water chamber and the diverted water is slowly released, as with the bucket system above, through a small bore pipe. Again the alternative is to use a tap. In some systems (notably one factory manufactured system from Australia) the top receiving chamber is designed such that a vortex is formed and any particles in the water are held in suspension in the vortex while only clean water passes into the storage tank. The 'waste' water can be used for irrigating garden plants or other suitable application. The debris has to be removed from the lower chamber occasionally.

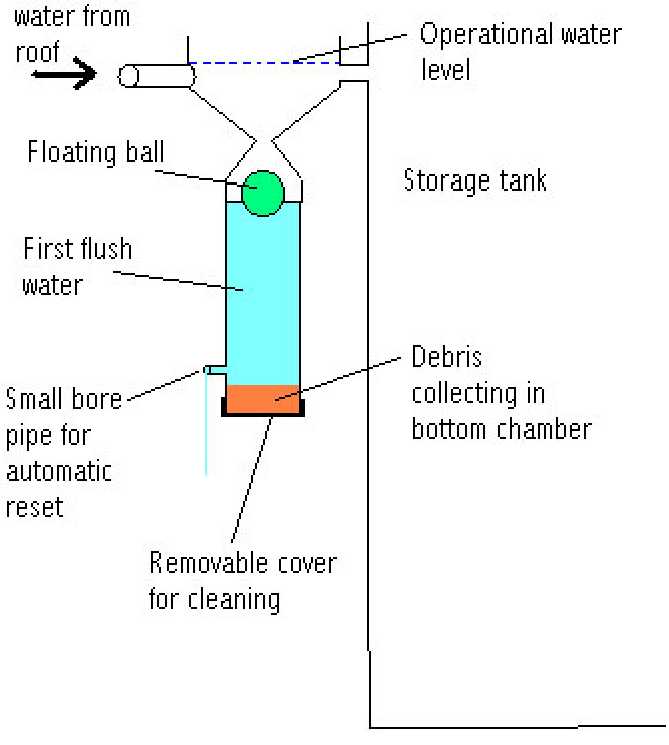


Figure 6 – the floating ball first flush system

Although the more sophisticated methods provide a much more elegant means of rejecting the first flush water, practitioners often recommend that very simple, easily maintained systems be used, as these are more likely to be repaired if failure occurs.

Filtration systems and settling tanks

Again, there are a wide variety of systems available for treating water before, during and after storage. The level of sophistication also varies, from extremely high-tech to very rudimentary. A German company, WISY, have developed an ingenious filter which fits into a vertical downpipe and acts as both filter and first-flush system. The filter cleverly takes in water through a very fine (0.17mm) mesh while allowing silt and debris to continue down the pipe. The efficiency of the filter is over 90%. This filter is commonly used in European systems.

The simple trash rack has been used in some systems but this type of filter has a number of problems attached: firstly it only removes large debris; and secondly the rack can become clogged easily and requires regular cleaning.

The sand-charcoal-stone filter is often used for filtering rainwater entering a tank. This type of filter is only suitable, however, where the inflow is slow, and will soon overflow if the inflow exceeds the rate at which the water can percolate through the sand.

Settling tanks and partitions can be used to remove silt and other suspended solids from the water. These are effective where used but add significant additional cost if elaborate techniques are used.

Post storage filtration include such systems as the upflow sand filter shown in Figure 7. Many other systems exist and can be found in the appropriate water literature.

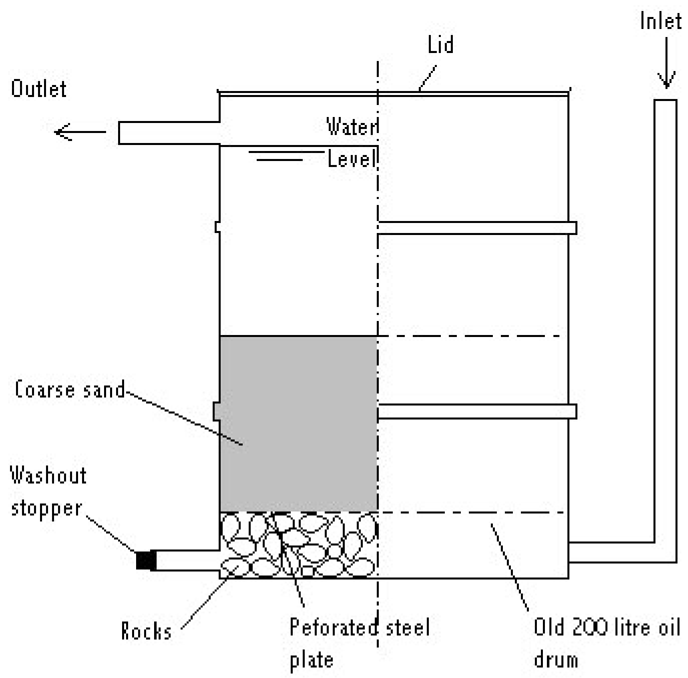
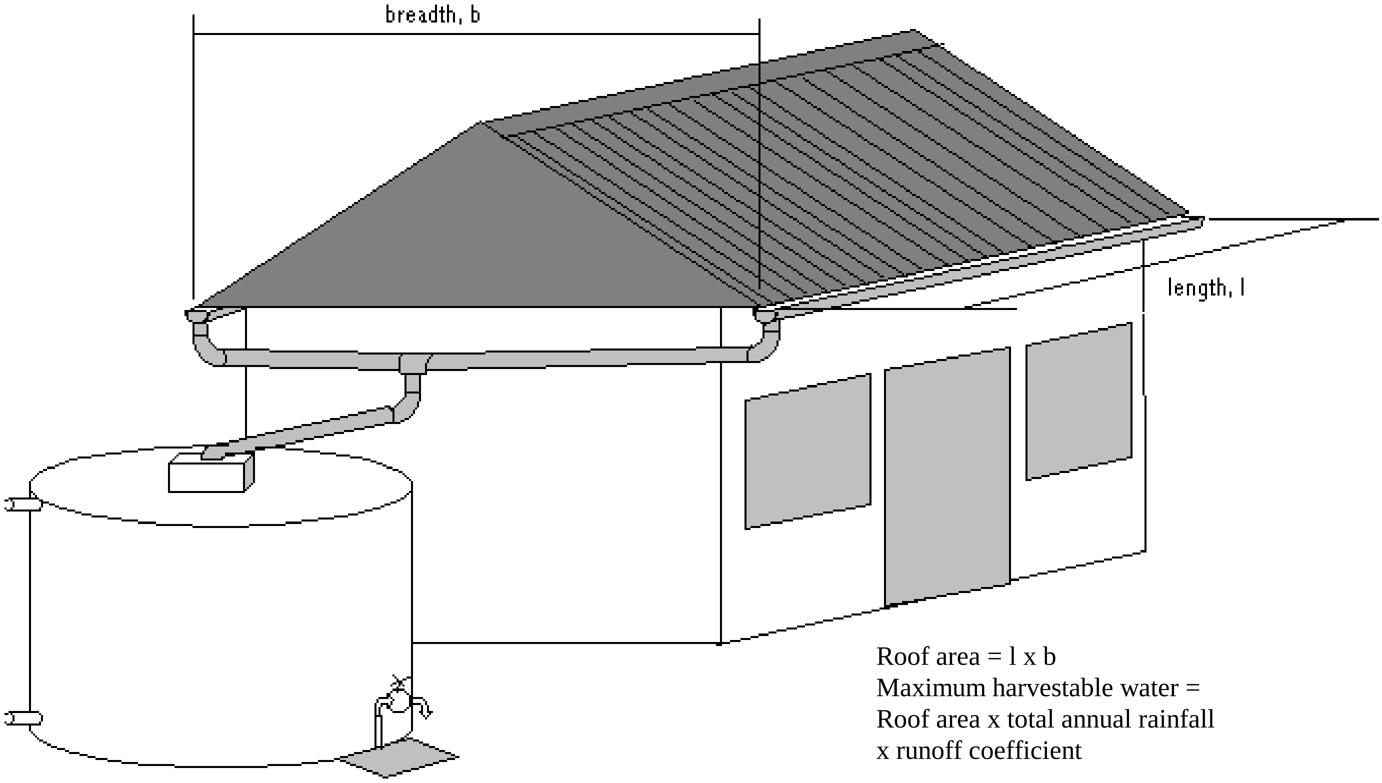


Figure 7 – Upflow sand filter for post treatment of stored water.

4. Sizing the DRWH system

Usually, the main calculation when designing a DRWH system will be to size the water tank correctly to give adequate storage capacity. The storage requirement will be determined by a number of interrelated factors. They include:

- local rainfall data and weather patterns
- roof (or other) collection area
- runoff coefficient (this varies between 0.5 and 0.9 depending on roof material and slope)
- user numbers and consumption rates



Roof area = l x b
Maximum harvestable water =
Roof area x total annual rainfall
x runoff coefficient

The style of rainwater harvesting (see Rainwater Harvesting Styles) will also play a part in determining the system components.

There are a number of different methods for sizing system components. These methods vary in complexity and sophistication. Some are readily carried out by relatively inexperienced first-time practitioners; others require computer software and trained engineers who understand how to use this software. The choice of method used to design system components will depend largely on the following factors:

- the size and sophistication of the system and its components
- the availability of the tools required for using a particular method (e.g. computers)
- the skill and education levels of the practitioner / designer

Below we will outline 3 different methods for sizing RWH system components.

Method 1 – demand side approach

A very simple method is to calculate the largest storage requirement based on the consumption rates and occupancy of the building.

As a simple example we can use the following typical data:

Consumption per capita per day, C – 20 litres
Number of people per household, n – 6
Longest average dry period – 25 days

Annual consumption = C x n x 365 = 43,800 litres

Storage requirement, T = $\frac{43{,}800 \times 25}{365}$ = 3,000 litres

This simple method assumes sufficient catchment area and rainfall and catchment area which is adequate, and is therefore only applicable in areas where this is the situation. It is a method for acquiring rough estimates of tank size.

Method 2 – supply side approach

In low rainfall areas or areas where the rainfall is of uneven distribution, more care has to be taken to size the storage properly. During some months of the year there may be an excess of water, while at other times there will be a deficit (see figure 1 below). If there is sufficient water throughout the year to meet the demand, then sufficient storage will be required to bridge the periods of scarcity. As storage is expensive, this should be done carefully to avoid unnecessary expense.

The example given here is a simple spreadsheet calculation for a site in North Western Tanzania. The rainfall statistics were gleaned from a nurse at the local hospital who had been keeping records for the previous 12 years. Average figures for the rainfall data were used to simplify the calculation, and no reliability calculation is done. This is a typical field approach to RWH storage sizing.

Example

Site: Medical dispensary, Ruganzu, Biharamulo District, Kagera, Tanzania (1997)
Demand:
Number of staff: 7
Staff consumption: 45 litres per day x 7 = 315 litres per day
Patients: 40
Patient consumption : 10 litres per day x 40 = 400 litres per day
Total demand: 715 litres per day or 260.97m^3 per month

Roof area: 190m^2
Runoff coefficient (for new corrugated GI roof): 0.9
Average annual rainfall: 1056mm per year
Annual available water (assuming all is collected) = 190 x 1.056 x 0.9 = 180.58m^3
Daily available water = 180.58 / 365 = 0.4947 m^3 / day or 494.7 litres per day or 150.48m^3 per month

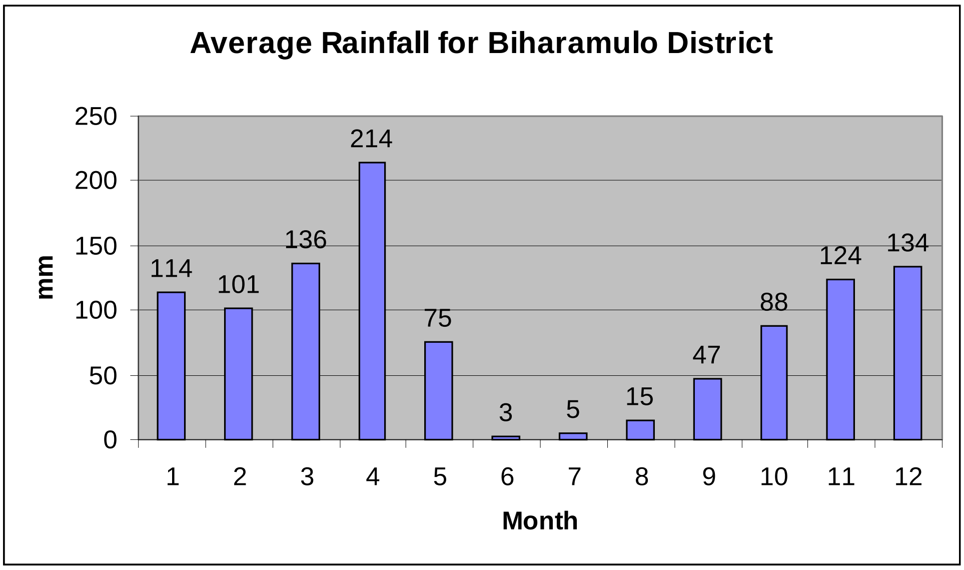


Figure 8 – average annual rainfall for the District of Biharamulo.

So, if we want to supply water all the year to meet the needs of the dispensary, the demand cannot exceed 494.7 litres per day. The expected demand cannot be met by the available harvested water. Careful water management will therefore be required.

Figure 9 below shows the comparison of water harvested and the amount that can be supplied to the dispensary using all the water which is harvested. It can be noted that there is a single rainy season. The first month that the rainfall on the roof meets the demand is October. If we therefore assume that the tank is empty at the end of September we can form a graph of cumulative harvested water and cumulative demand and from this we can calculate the maximum storage requirement for the dispensary.

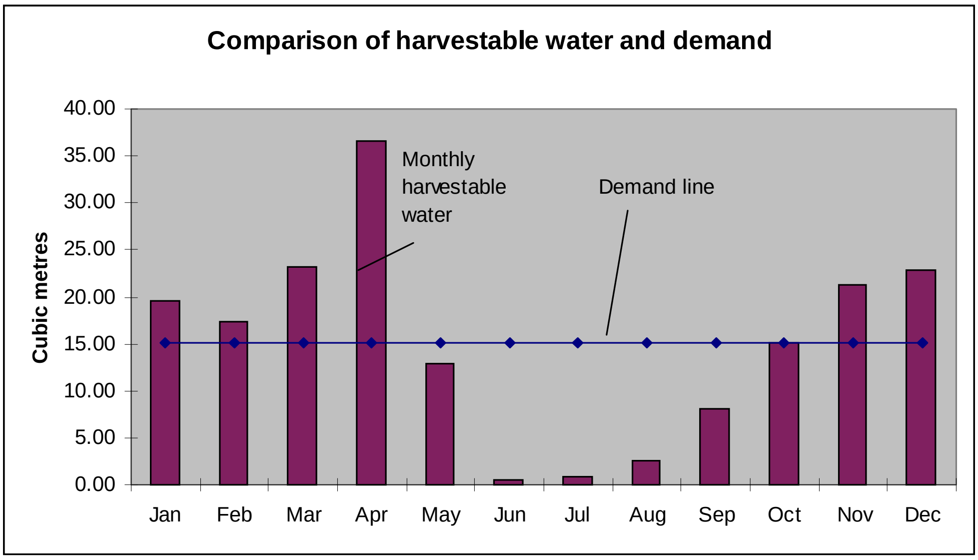


Figure 9 – comparison of the harvestable water and the demand for each month.

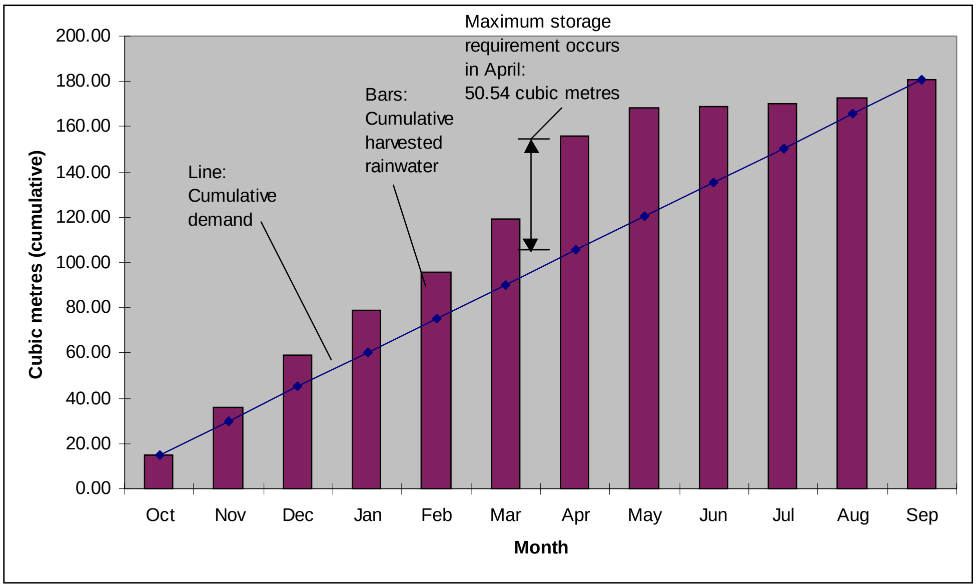


Figure 10 – showing the predicted cumulative inflow and outflow from the tank. The maximum storage requirement occurs in April.

Month	Rainfall (mm)	Rainfall harvested (cubic metres)	Cumulative rainfall harvested (cubic metres)	Demand (based on total utilisation)	Cumulative demand (cubic metres)	Difference between column 4 and 6
Oct	88	15.05	15.05	15.05	15.05	0.00
Nov	124	21.20	36.25	15.05	30.10	6.16
Dec	134	22.91	59.17	15.05	45.14	14.02

Jan	114	19.49	78.66	15.05	60.19	18.47
Feb	101	17.27	95.93	15.05	75.24	20.69
Mar	136	23.26	119.19	15.05	90.29	28.90
Apr	214	36.59	155.78	15.05	105.34	**50.45**
May	75	12.83	168.61	15.05	120.38	48.22
Jun	3	0.51	169.12	15.05	135.43	33.69
Jul	5	0.86	169.97	15.05	150.48	19.49
Aug	15	2.57	172.54	15.05	165.53	7.01
Sep	47	8.04	180.58	15.05	180.58	0.00
Totals		180.58		180.58		

Table 1 - shows the spreadsheet calculation for sizing the storage tank. It takes into consideration the accumulated inflow and outflow from the tank and the capacity of the tank is calculated as the greatest excess of water over and above consumption. This occurs in April with a storage requirement of 50.45 cubic metres. All this water will have to be stored to cover the shortfall during the dry period.

Method 3 – computer model

There are several computer-based programmes for calculating tank size quite accurately. One such programme, known as SimTanka, has been written by an Indian organisation and is available free of charge on the World Wide Web. The Ajit Foundation is a registered non-profit voluntary organisation with its main office in Jaipur, India and its community resource centre in Bikaner, India.

SimTanka is a software programme for simulating performance of rainwater harvesting systems with covered water storage tank. Such systems are called Tanka in western parts of the state of Rajasthan in India.

The idea of a computer simulation is to predict the performance of a rainwater harvesting system based on the mathematical model of the actual system. In particular SimTanka simulates the fluctuating rainfall on which the rainwater harvesting system is dependent.

Rainwater harvesting systems are often designed using some statistical indicator of the rainfall for a given place, like the average rainfall. When the rainfall is meagre and shows large fluctuations then a design based on any single statistical indicator can be
misleading. SimTanka takes into account the fluctuations in the rainfall, giving each fluctuation its right importance for determining the size of the rainwater harvesting system. The result of the simulation allows you to design a rainwater harvesting
system that will meet demands reliably, that is, it allows you to find the minimum catchment area and the smallest possible storage tank that will meet your demand with probability of up to 95% in spite of the fluctuations in the rainfall. Or you can use SimTanka to find out what fraction of your total demand can be met reliably.

SimTanka requires at least 15 years of monthly rainfall records for the place at which the rainwater harvesting system is located. If you do not have the rainfall record for the place then the rainfall record from the nearest place which has the same PATTERN of rainfall can be used.

The included utility, RainRecorder, is used for entering the rainfall data. Daily consumption per person is also entered and then the software will calculate optimum storage size or catchment size depending on the requirements of the user. SimTanka also calculates the reliability of the system based on the rainfall data of the previous 15 years.

SimTanka is free and is and was developed by the Ajit Foundation in the spirit that it might be useful for meeting the water needs of small communities in a sustainable and reliable manner. But no guaranties of any kind are implied.

For more information or to download the software see their website at
http://www.geocities.com/Rainforest/Canopy/4805/

(Source: the information given here is taken from this website).

Further comments

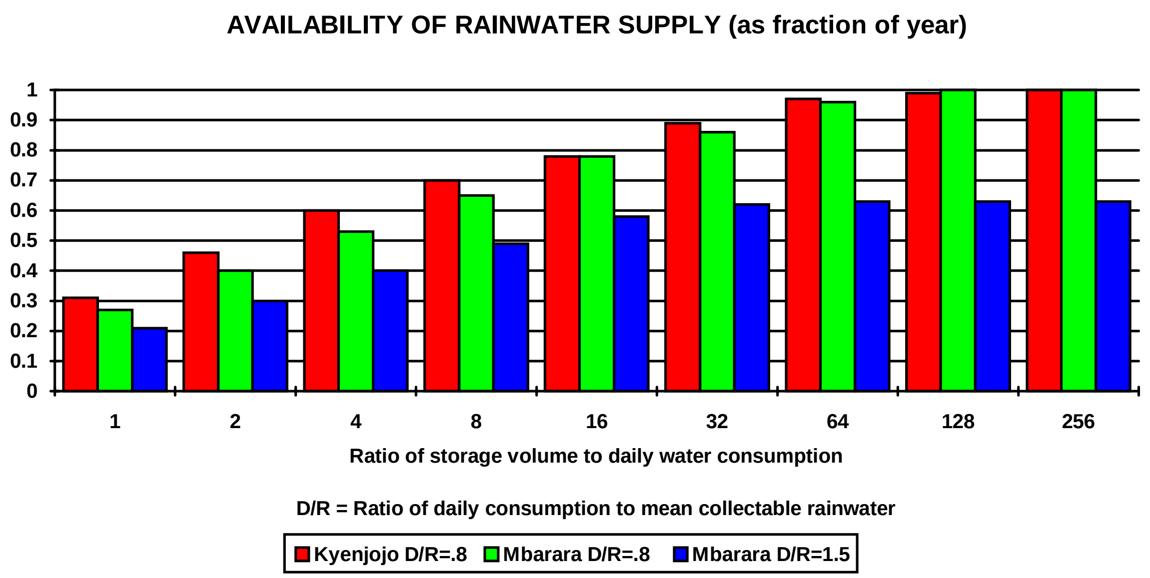
These methods outlined above can be further refined where necessary to use daily rainfall data. This is particularly important in areas where rainfall is more evenly distributed and more sensitive calculations are necessary.

Rainfall data can be obtained from a variety of sources. The first point of call should be the national meteorological organisation for the country in question. In some developing countries, however, statistics are limited due to lack of resources and other sources are often worth seeking. Local Water Departments or organisations, local hospitals or schools are all possible sources of information.

In reality the cost of the tank materials will often govern the choice of tank size. In other cases, such as large RWH programmes, standard sizes of tank are used regardless of consumption patterns, roof size or number of individual users.

Tank efficiency and the case for diminishing returns

On days when rainfall is heavy, the flow into a tank is higher than the outflow drawn by water users. A small tank will soon become full and then start to overflow. An *inefficient* system is one where, taken over say a year, that overflow constitutes a significant fraction of the water flowing into the tank. Insufficient storage volume is however not the only cause of inefficiency: poor guttering will fail to catch water during intense rain, leaking tanks will lose water, and an 'oversize' roof will intercept more rainfall than is needed.
Storage efficiency (%) =100 x (1 - *overflow / inflow*) provided that *inflow<demand*
System efficiency (%) = 100 x *water used / water falling on the roof*
In the dry season, a small tank may run dry, forcing users to seek water from alternative sources. *Unreliability* might be expressed as either the fraction of time (e.g. of days) when the tank is dry or the fraction of annual water use that has to be drawn from elsewhere. A RWH system may show unreliability not only because storage is small, but because the roof area is insufficient. The figure below shows how *reliability*, expressed as a fraction of year, varies with storage volume (expressed as a multiple of daily consumption) for two locations close to the Equator and therefore both with double rainy seasons.



From this graph one can see that increasing storage size, and therefore cost, gives diminishing returns. For example look at the left hand column of each triplet (Kyenjojo with roof sized such that average annual water demand is only 80% of average annual roof runoff). Assuming a say 100 litres per day demand, shows that increasing storage from 1 day (100 l) to 16 days (1600 l) raises the reliability from 31% to 78%, but storage has to be increased as high as 128 days (12,800 l) to achieve 99% reliability. Such high reliability is so expensive that it is an unrealistic design objective for a DRW system in a poor country. In any case, as we shall see below, users may change behaviour so as to reduce the effective unreliability of their systems.

System features (that affect tank sizing)

- An oversize roof slightly extent compensates for an undersize tank.
- If users are able and willing to adjust their consumption downwards during dry seasons, or when they find water levels in their tank lower than average, tanks can be sized smaller.
- ‘Partial’ RWH systems, either where it is accepted that RW will not meet needs throughout the year or where rainwater is only used to meet specific water needs like cooking/drinking, can be built with surprisingly small tanks.
- The reliability level appropriate to the design of a RWH system rises with the cost (in money, effort or even ill-health) of the alternative source that is used when the tank runs dry.

Rainfall data

Rainfall is very variable, especially where annual precipitation is less than 500mm. It also varies with location, so that data from a rain gauging station 20km away may be misleading when applied to the site of the RWH system. From the lowest to the highest quality of rainfall data we can think of at least 6 categories:

1. No numerical data available, but of course local people know quite well the seasonality of precipitation and which crops will grow (with what sort of water-stress failure rate).
2. There is no numerical data, but RWH has been practised for long enough locally for people to have a feel for what is an adequate tank size.
3. Only annual average rainfall is available, probably at a somewhat distant recording point, plus local knowledge of seasonality.
4. Monthly rainfall, averaged over at least 4 years, can be obtained.
5. Actual monthly rainfall records for at least 4 years, and preferably 7 years, are available for the site or for a location sufficiently nearby to give confidence or allow some systematic correction to be applied.
6. Daily rainfall data for a relevant location and lasting at least 4 years is available.

Daily data is adequate for all design methods except perhaps the optimisation of gutters for which *rainfall intensity* data is useful (e.g. the fraction of annual precipitation falling at a rate faster than say 1mm per minute). Rainfall data can be expensive to purchase and is often hard to locate even where it exists. Obviously methods of sizing tanks that require as an input rainfall data of a sort that is not locally available should not be used.

Possible methods for estimating likely future RWH system performance with a tank of given size

1. Ask local people with existing RWH systems and a comparable roof size what tank size they used and what has been their experience with it.
2. Mean dry season duration x daily dry season consumption. This may define an ‘ideal’ below which the designer chooses the ‘biggest affordable’.
3. Model *reliability* or *efficiency* using mean monthly rainfall data. (One variant of this is to compare the cumulative supply and usage loci to determine the maximum deviation of the former below the latter)
4. Test different tank sizes against actual *monthly* rainfall data over several recent years (or for tanks equivalent to less than 2 months water demand actual *daily* data) and thereby deciding the best reliability v cost trade-off.

5. Reasons for the current interest in DRWH

In the last decade there has been a large increase in the amount of interest in, and application of, DRWH in both developed and developing countries. There are a number of reasons for this growth. In developing countries these include: disillusionment with traditional technologies, which have failed to meet the needs of the people; a growing number of problems with groundwater sources; a realisation that DRWH is a viable alternative which gives the user autonomy and independence. In the industrialised countries the motivation has been: to reduce water bills, which have grown in some countries at alarming rates; to meet the ever growing needs (particularly for non-potable water) of ever more demanding societies; to offset the huge demand on depleted or polluted groundwater resources.

The following is a small collection of articles, reports and comments which illustrate these issues:

Rainwater Cistern Systems as an Alternative Drinking Water Source in Regions of Inadequate or Unsuitable Groundwater

The Canadian Province of Nova Scotia includes a population of approximately 850,000 persons in an area of 52,840 sq km. Approximately 320,000 persons are served by private well supplies. The quality of groundwater in Nova Scotia is generally good. However, there are areas of the province in which adequate reliable supplies of groundwater of acceptable quality are not available to individual dwellings. In these situations rainwater cistern systems have been used or proposed for use. An estimated total of up to 500 dwellings in Nova Scotia are now served by rainwater cistern systems. The rainwater system consists of a roof , which serves as a collection surface, and gutters and down-spouts that are connected to a storage reservoir located in a basement or underground. Treatment devices for disinfection and/or filtration have been installed by some owners on the household side of the cistern. The largest single reason for use of these systems is their location in areas where groundwater is completely unacceptable for domestic purposes because of local gypsum deposits. Other reasons include unsatisfactory well water caused by salt water intrusion, iron and manganese, or inadequate groundwater yield. Studies have indicated that a rainwater cistern can provide a reliable and effective method for the supply of water to an individual dwelling, provided that the size of the roof surface is adequate to meet the long-term demands of the occupants, and that the owner is prepared: (1) to install a cistern of adequate size; to provide a method for disinfection; and, (2) deal with routine maintenance in the form of cleaning of gutters and the cistern interior.

Author(s): D. H. Waller, R. S. Scott.
Citation: IN: International Groundwater
Symposium on Hydrogeology of Cold and Temperate Climates and Hydrogeology of Mineralized Zones. Proceedings of the Symposium held May 1-5, 1988, Halifax, Nova Scotia, Canada. (1989). pg 247-255, 1 fig, 7 tab, 14 ref.

The town of Avadi in Tamil Nadu, India, is using rainwater harvesting to augment its scarce groundwater resources. Not only were groundwater levels dropping but also groundwater quality has deteriorated, limiting water availability to only 4 litres per capita per day (lpcd). A consultant calculated that supply can be at least be doubled by channelling rainwater collected from roof tops and vacant areas around houses via percolation pits and a filter into household wells. Total costs for a system are INR 5,000 (US$ 118) of which INR 3,000 (US$ 71) for piping and the remainder is for sinking percolation pits. So far 10 houses have employed a rainwater harvesting system. The recharging method can be modified to tap water from storm-water drains, canals and boreholes.

Source: The Hindu, 21 September 1998

A survey of 3014 people across South Australia was undertaken to determine the sources of water for drinking and cooking. Overall, rainwater was found to be the main source of water for drinking – 42.2% of households used tank water for drinking, followed by mains supply at 40.3%. For cooking, mains supply was used by 64.3% of households, followed by 29.9% of households using rainwater from tanks. In the Metropolitan area, on average 25.6% of households used rainwater for drinking, whereas in rural areas, on average 81.5% of households use rainwater with use in some areas approaching 100%.

Source: Water – Journal of the Australian Water and Wastewater Association, January/ February 1998

The Tokyo Metropolitan Government is also promoting the use of RWH in the city, and have included for RWH in the Water Policy Guidelines. This is in response to an inability to meet urban demand because of inadequate sources of water.

In Germany there are increasing concerns about nitrates and pesticides in ground water. The concentrations of both of these is growing notably. Also an increased demand for drinking water (e.g. 1200% increase in consumption between 1975 and 1994 in the Oberfranken area of Northern Bavaria) is causing a major headache for water supply companies and the government.

Arsenic poisoning in Bangladesh and India

An estimated 18 million people in Bangladesh and West Bengal (India) are poisoning themselves by drinking arsenic-contaminated groundwater. Victims suffer a painful death, from one of several forms of cancer, with skin lesions on hands and feet as advanced signs of poisoning. The crisis is a result of extensive UNICEF and government-sponsored well drilling programs for over 25 years which successfully cut mortality rates attributed to diarrhoea but did not carry out water testing. Despite continued warnings by analytical chemist, Dr. Dipankar Chakraborty, as far back as 1988, government officials generally dismissed him as an alarmist. In late August 1998, the World Bank announced a US$ 32.4 million loan for the Bangladesh Arsenic Mitigation Water Supply Project (BAMWSP). The IRC International Water and Sanitation is helping prepare the operations steps and manuals for BAMWSP.
New York Times, 10 November 1998

An estimated 100 people in Bangladesh have died of arsenic-related diseases so far in 1998 and 1,000 have become ill. Most deaths are not known, however, because the problem was not officially recognised until 1993 [1]. In one of several articles on the arsenic crisis, journalist Sylvia Mortoza mentions that over 2,500 have died in Bangladesh over a few months [2]. Now major donor help is underway, nowhere near the 70 million people potentially at risk will fall ill. The issue has also raised awareness of arsenic poisoning in the US where environmentalists took the Environmental Protection Agency to court for dragging its feet on the adoption of the World Health Organization (WHO) arsenic guideline value of 10 ppb [3]. An overview of current arsenic research projects can be found at the West Bengal and Bangladesh Arsenic Crisis Information Centre site: http://bicn.com/acic/
http://www.wsscc.org/source/weekly/9839.html#arsenic_poisoning
[1] - Fox news, 16 November 1998,
http://www.foxnews.com/js_index.sml?content=/news/wires2/1116/n_ap_1116_139.sml
[2] - http://bicn.com/acic/infobank/mortoza6.htm
[3] - Chemical & Engineering News, vol. 76, no. 46, November 16, 1998,
http://pubs.acs.org/cgi-bin/bottomframe.cgi?7646inter
Source: SOURCE Weekly Bulletin No. 40, 23rd November 1998

Process and Progress
Where they have been installed, government tubewells and handpumps have been poorly maintained in rural areas and delays of several months before repairs are carried out are not uncommon. The urban context differs in that water infrastructure exists with both intermittent and unreliable supply.

Benefits from government sanitation programmes to rural communities in the project areas visited have been limited, and sanitation and drainage in the urban project area were inadequate before the project intervention.
Source: Water Aid Evaluation Report – South India 1995

Rainwater harvesting-Kibwezi project
This project includes the provision of guttering and the construction of rainwater storage tanks at 46 schools in the area. The purpose of the programme is to provide sufficient water throughout the year for the pupils, both for drinking and for cooking their midday meal. The work is carried out under KWAHO supervision with the assistance of parents who excavate the pit in which the ground tank is cast, and provide sand for its lining. At the time of evaluation, this installation had been completed in 27 schools, of which 10 incorporated ferro-cement tanks above ground with a capacity of 30 cubic metres and 17 are below ground with a capacity of 75 cubic metres. It is possible that low lift pumps installed in certain schools may not be suitable, in that they are mounted at about 45 degrees to the vertical.

Mfangano Island Project
Shallow wells constructed on Mfangano Island were intended to respond to endemic cholera, dysentery, diarrhoea and billharzia found in the Lake water from which inhabitants draw their supplies. To provide sources of safe water for inhabitants who tend to cluster around the lake shore, a trial programme for the construction of ten shallow wells has been carried out but with very limited success. In almost all cases, the water was saline and unacceptable to villagers.

Ngusuria Project
In Ngusuria, villagers have laid 12 km of 50 mm diameter GS pipe to supply three villages from a spring and impoundment high up on the side of the escarpment surrounding the area. Service mains, storage tanks and break pressure tanks have also been installed serving some 25 stand pipes and private connections. The discharge from the spring often reduces to zero at the end of the dry season. In this event, supplies are provided from a deep, copious borehole in which an electro-submersible pump has been installed. This was unserviceable at the time of the visit. Arrangements to lift the pump and motor for repairs were being made.

Source: Water Aid Evaluation report – Kenya 1991

6.Criteria for the analysis of DRWH systems

The possible applications of rainwater harvesting are endless. For each circumstance, be it an affluent South Australian urban dwelling or a poor African rural household, there will be a desirable solution. The requirements of each individual will be different. For some, space will be limited, while for others cost may be the over-riding factor. It is therefore impossible to suggest a suitable DRWH system without knowing the context for which it will be used. We can however carry out a broad analysis for the systems considered here.

To facilitate such an analysis to be carried out we have developed a set of criteria by which the DRWH systems can be assessed by the reader. The list is by no means exhaustive and in most cases is purely subjective. Where actual facts and figures are available, they are given in the Case Study and the reader can judge him/herself the suitability of the system in question. The table shown below is completed for each example and shown at the end of each Case Study.

A cost analysis of DRWH systems is somewhat meaningless when the value of world currencies varies so widely. We have therefore considered not only local costs (and converted this into US Dollars for easy reference, but also considered this cost as a percentage of the average monthly income for the region in question.

Material requirement and costs	
Labour costs	

Unit costs (i.e. cost per m^3)	
Unit cost as a percentage of average monthly income	
Skills required	
Equipment / tools required	
Space requirement	
Suitability for incremental adoption	
Reliability	
Durability	
Water quality, safety and health	
Impact on insect breeding	
Ease of operation	
Suitability	
Stage of maturity or experience	
Other	

(This work has yet to be done on our Web Site)

7.Health and DRWH

The literature on health aspects of RWH is surprisingly sporadic Gould & McPherson 1987, S Australia 1981, even in countries where RWH use is widespread and of long standing. We are interested in the mineral quality of roof run-off, in its bacteriological quality, in the connections between RWH and the breeding of disease vectors (especially mosquitoes), and in the risk of accidents such as children drowning. We would like to know not only how new systems might perform and do actually perform, but also whether that performance declines with the age of a RWH system. On the positive side, we should like to be able to quantify the undoubted health benefits of women spending less time collecting water - benefits such as fewer accidents to unattended infants, better nutrition, less female back injury and of course the hygiene benefits of greater water consumption which introducing RWH sometimes brings.

RWH may not be competing on a 'level playing field' when it comes to health criteria. Because there are few specific health standards defined for harvested run-off, there is the danger that inappropriate norms will be demanded of it. Competently harvested roofwater generally has negligible levels of pollution by minerals and low levels of bacterial pollution. In almost all developing country situations its quality is likely to be superior to that of such alternatives as discontinuously-pressured piped water, shallow well water and even deep well water. Conversely it may not achieve the bacteriological quality of treated water entering mains from a water works, or that of delivered water in rich countries. Roofwater that is incompetently collected or stored may indeed be turbid and a possible source of pathogens.

There have been concerns that rain may pick up unhealthy substances whilst falling through the atmosphere, whilst running down a roof or whilst resting in a store. The danger from the first of these, namely atmospheric pollution, seem slight. Measurements of precipitation even in industrialised areas Thomas & Greene 1993 indicate a fairly low take-up of heavy metals from the air and wholly tolerable levels of acidity; however no doubt it would be unwise to harvest rainwater immediately downwind of say a smelter. The probability of finding truly-airborne ingestible pathogenic viruses or bacteria seems low and of finding larger airborne pathogens negligible. Interest therefore focuses mainly upon contamination of roofs and the performance of water stores in reducing or increasing pathogens.

Roofs and gutters are made of a variety of materials. For most practical purposes we can exclude discussion of 'organic' roofs such as grass, reed and palm because they yield such dirty run-off that they are rarely used for RWH. The common materials of interest are ceramic, cementitious, rock and metallic (plastic roofs being neither cheap nor durable). Contamination of water might arise from the roofing material itself or from substances that have accumulated on a roof or in a gutter.

Metal roofs are normally of treated steel or less commonly of aluminium. Aluminium is very inert unless in contact with very acid water. However the effect on health of ingesting tiny amounts of aluminium are controversial; there has been some debate in Europe about a possible link between such ingestion (from the aluminium saucepans popular up to 1960) and the development of Alzheimer's Disease that causes premature senility. Corrugated steel roofing employs mild steel protected by hot-dip or electrolytic galvanising or by painting, since stainless steel is too expensive to use. Galvanising entails zinc compounds: fortunately zinc has a low toxicity, so that roof run-off water does not exceed WHO-permitted zinc levels. Roof paints including bitumen may entail some risk to health and/or may impart unpleasant taste to roofwater and should probably be avoided for RWH. More seriously although no one can now afford lead sheeting on roofs, localised lead 'flashing' is still used at joints. One study in Malaysia Yaziz 1989 reported lead levels of up to 3.5 times WHO limits in roof runoff but this is not a general finding and seems to have arisen from lead in dust deposition rather than the roofing material since it reduced rapidly with storm duration. Not surprisingly the safety of water harvested from 'asbestos' (= asbestos-reinforced cement mortar) roofs has been queried, but the consensus is that the danger of developing cancer from ingested asbestos is very slight Campbell 1993. The danger from inhaled asbestos dust is however sufficiently high that working with asbestos sheeting, for example sawing it, without special protection is now generally banned in industrial countries. The iron in a rusting roof will also enter the runoff, but in such small quantities that it does not prejudice either health or taste.

Metal roofs are comparatively smooth and are therefore less prone to contamination by dust, leaves, bird-droppings and other debris than rougher tile roofs. They may also get hot enough to sterilise themselves. However contamination may be substantial on all roof types and it has been common for many years to design 'first-flush diverters' into RWH systems. During a dry spell debris builds up on roofs, so that the initial run-off during the first following rainfall event can be full of sediment and highly turbid. Overhanging trees, especially coconut palms, make this sediment problem worse, as well as increasing the likelihood of bird and rodent droppings. A common strategy therefore is to divert to waste the first say 5 litres of runoff at the beginning of each rain event. This can be done automatically using proprietary devices, or where the seasons are well defined it can be done manually by temporarily displacing the pipe connecting gutter to storage tank. If this first flush is excluded, we have a water source with modest levels of turbidity and typically medium levels of bacterial contamination (e.g. <10 FC per 100ml). Modern 'no-maintenance' separators, or more traditional screens, cloth or sand filters will reduce turbidity and contamination further and any good tank design will reduce it further still.

Not all tanks are however designed, made and maintained well. One can commonly see tanks which offer access to insects, lizards and rodents and which permit enough light to enter that algae can grow. Such tanks take longer to lower the contamination level of the entering flow and may even permit new infection for example by pathogens carried on the feet of cockroaches. Water abstraction is occasionally by lowered bucket - with all the opportunities for contamination that offers - and not uncommonly by a tap set too low in the tank so that tank-bottom sediment may be drawn into the outflow. However tests in even poorly designed tanks commonly give levels of bacterial contamination (rarely over 5 FC per 100ml) that compare well with those in competing water sources in developing countries. The technique of filling a RWH tank then sealing it for a month or more produces excellent water quality. It seems it is possible even without such steps to meet the highest international standards for bacteria and dissolved substances with well-made RWH systems incorporating effective prefiltration and careful in-tank flow guidance. Cleaning tanks, say annually, should improve water quality, provided any remaining disturbed sediment is allowed to resettle for several days before the tank is used again. With the best pre-tank separators however, the rate of entry of organic material is so low that (provided no photo-synthesis occurs) such material can be entirely removed by aerobic bacterial action and no cleaning is required.

Water tanks are close to houses. Moreover they usually contain water during some or all of any dry season, a time when alternative breeding grounds for mosquitoes dry up. For both these reasons it is important that they do not act as significant breeding sites. The design of tanks and guttering to exclude insect breeding requires a mixture of common sense and professional engineering or entomological knowledge. It is common sense to so align gutters, and keep them clear of blockages, that they do not hold stagnant pools after rainfall finishes. It is engineering expertise or long experience that generates good designs for self-clearing gutters or filters. It perhaps requires entomological expertise to identify tank shapes that lower the chance of successful larval development. Mosquito eggs are sufficiently small that they could pass through most filters with entry water: such filters cannot be very fine if they are to be able to handle the sudden and large flows during intense tropical rainfall of up to 1 millimetre per minute. Mosquito control is therefore a matter of preventing the entry and exit of adult insects and interfering with larval growth. The former may be difficult to achieve if tank maintenance is poor or if users place greater importance on maximising tank inflow than on maintaining mosquito defences. It is therefore attractive to have the ‘defence-in-depth’ of larval control. This may take the form of active control with fish predators, surface oil films and suchlike but a more rewarding general policy is to starve larvae. Maintaining darkness in a tank prevents photo-synthesis and the growth of algae. Preventing the entry of suspended materials reduces the general nutrient levels supporting any biological chain. Research is underway into these factors and it seems likely that fairly straightforward measures can render a tank unsuitable for dry-season breeding of anopheles, aedes and culecine mosquitoes. Moreover, broadly speaking, if mosquitoes can be controlled it should be relatively easy to control larger disease vectors like cockroaches.

If a child falls into a tank, even if that child can swim, there is a real danger of drowning. Many existing tanks have no covers or easily displaced covers and stories of children deliberately bathing in free-standing RWH tanks are to be heard. Perhaps of most concern are underground tanks whose covers have been opened for inspection, maintenance or even for drawing water. It is not normal to fence underground tanks, to extend them above the ground high enough to deter access by crawling babies or to socially control children from playing on them. However fencing and/or partial raising could have advantages including reducing danger of contamination by surface water and lowering the chance of cover damage by vehicles as well as reducing the risk of children or night-moving adults falling in. Accidents like drowning are most likely where a new technology is being introduced and therefore should be the particular concern of technology-change agents.

Every technology has its obscure and rarely-met failure and danger modes. Clearly any aspect of RWH that involves human activity on high roofs, handling rusty metal or working ‘underground’ involves some risk of accident. A particular danger, known to have asphyxiated at least one builder recently, is the possibility of deoxygenation within a closed tank during the process of mortar setting and curing.

Finally, under health considerations, one might mention floods. If flood levels are higher than the entry point of RWH tank entrances, there is the real danger of serious contamination of the stored water. This danger may be avoided by suitable tank location or by the permanent presence of a say slow-sand entry filter.

Report A1 –Current Technology for Storing Domestic Rainwater (Part 2)

The work in this report forms the basis for the current DTU roofwater harvesting Web Site. The site can be found at:

http://www.eng.warwick.ac.uk/DTU/rainwaterharvesting/index.html

The work in this document is on-going and will be added to as the programme progresses. The aim is to collect examples of DRWH practice from around the world and to provide a useful resource for practitioners of DRWH. Many of the graphics shown in the Web Site are not shown in this document due to electronic storage requirement limitations. The report is in two parts so that the document remains manageable. This is Part 2.

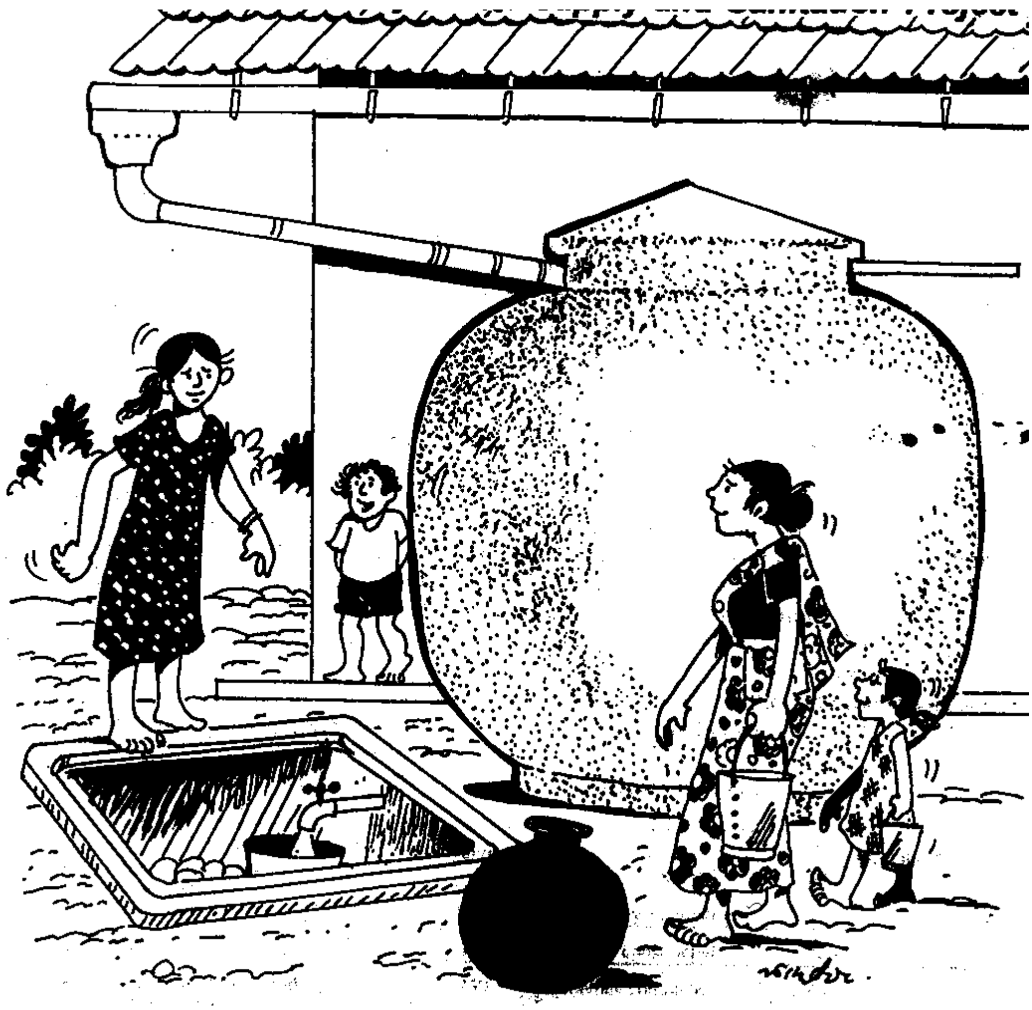
Contents of report (Part 2)

Case study 1 - The Sri Lankan Pumpkin Tank

Background Information

The Sri Lankan Pumpkin Tank, and the associated construction technique, was developed as part of a World Bank sponsored Water and Sanitation Programme which was implemented in the country between 1995 and 1998. The Community Water Supply and Sanitation Programme (CWSSP) covered 3 districts within the country – Badulla, Ratnapura and Matara Districts. Hundreds of these tanks were built in areas where conventional supply schemes, such as piped supplies or groundwater supplies, were difficult to provide. In some areas members of the target community were given the choice of a RWH system for individual households or a groundwater supply for a group of households. The choice varied. In all cases there was a choice of type of tank – either the Pumpkin tank or an underground tank which is described in Case Study 2. The choice was usually a function of ground conditions rather than personal preference. Both tanks have a capacity of approximately $5m^3$.

The Abikon family of Demetaralhina in Badulla District chose a pumpkin tank. Their village is in a rural highlands area of the country and the ground conditions were not suitable for a groundwater supply or for digging a pit for a below ground tank. Average annual rainfall is 2250mm with a bimodal rainfall pattern and a dry period, usually between December and April. Their per capita consumption was well below the 20 litres per day that each family member now consumes. The water is used for drinking (but only after boiling), cooking, personal and clothes washing. Mr Abikon also uses the water from their tank to water their 4 cows. Only towards the end of the dry season does the tank sometimes dry and then the family has to walk to the spring, about ½ a mile from their home.



Technical details

Rainwater is collected from only 1 side of the pitched roof, a collection area of 32m^2. The roofing material is a mix of zinc and asbestos sheeting. The guttering is a PVC U-channel, factory manufactured, found commonly in the nearby town, fitted to a facia board with similarly manufactured brackets, spaced at 300mm centres. The downpipe is a standard 3” PVC pipe, although some of the neighbours use less costly downpipes made from string and plastic tubing. The cost of the guttering is approximatley SLR5,600, about Sterling £86.00.

This pumpkin tank was built 3 years ago and is in very good condition. The construction is of ferrocement. The construction detail is given later. The cost of the tank is approximately SLR5,000 or Sterling £77.00. The materials and specialist labour for the tank were provided by CWSSP and the guttering was purchased by the Abikon family.

Water extraction is through a tap piped to a point slightly away from the tank, where the ground falls away and allows a bucket to be placed easily under the tap. There is a first flush mechanism fitted in the form of a simple PVC elbow with a length of pipe which diverts the dirty first water away from the
inlet chamber. The inlet chamber also acts as the prefilter chamber. The chamber is approximately 600mm cubed and contains subsequent layers of stone, charcoal and sand, through which rainwater passes.

Construction details:

The following construction details are given in the instructions which are handed out to masons during their training session:

Pumpkin (Wataka) Tank – Construction details

1. Prepare skeleton / framework legs (see Figure 1) as shown in the drawing. 10 no. required. Prepare the crown ring. This can be used again for many tanks.

(Figure 1 photo – one of the 10 framework legs used as the skeleton for the tank – shown in Web Page)

2. Lay the concrete base using two layers of chicken wire as reinforcing. Allow 300mm of chicken wire to protrude all around the edge of the base. This will be connected to the wall mesh later. Lay 10 anchor bolts for the legs in the base while casting (the diameter will depend on the diameter of the holes in the legs).
3. Leave the base for 7 days to cure, wetting each day.
4. Secure the 10 skeleton legs using the bolts and the crown ring.
5. Take 6mm steel rod and wrap it around the outside of the legs, starting at the bottom and working up at 10cm intervals.
6. Fix 2 layers of chicken wire over the outside of the skeleton. The filter tower can be added at this point if a filter is to be fitted.

(Figure 2 photo – a Pumpkin tank under construction – shown in Web Page)

7. Plaster the outside of the mesh. Leave for 1 day.
8. Go inside the tank and remove the skeleton.
9. Plaster inside the tank and cure for 7 days.

Water proofing can be added to the mortar. This can be a specialist additive or liquid dishwashing soap.

Cure the tank by wetting for 7 – 10 days. Fill the gradually starting on day 7, filling at a rate of approximately 300mm per day.

(Figure 3 photo – a finished Pumpkin tank)

Materials and labour breakdown

Material	Unit	Qty	Unit Cost	Total cost
Cement	Bag	8	265	2120

Sand	ft^3	55	3.5	192.5
Metal	ft^3	6	18	108
½" Chicken Mesh	ft^2	366	4	1464
Mould		1	325*	325
Transport				500
Skilled labour	hr	56	22	1232
Unskilled labour	hr	112	12.5	1400
				7341.5

*Assuming mould is used for 10 tanks
All costs given in Sri Lankan Rupees

65 SL Rupees = Sterling £1.00

Case study 2 – Underground brick dome tank, Sri Lanka

Background

This is another RWH system, as with the previous case study, which was developed by the CWSSP programme in Sri Lanka (see Case Study 1 for more detail). The tank, a 5m^3 underground brick built tank, is based loosely on the design of the Chinese below ground biogas tank. Indeed, the Sri Lankan engineer who designed the system had studied for some years in China. This is a good example of cross fertilisation of technologies across cultures, as well as the application of appropriate technology.

Again, this system was introduced due to the difficulties faced in bringing water to this community in a conventional manner. There was a lot of opposition to the RWH technology in the area at first, as it was a technology which was not widely known in the. Now, after 2 years using the rainwater falling on her roof, Mrs. Emsayakar, of Batalahena Village near the town Matara, sees things very differently.

The alternative offered by CWSSP was a handpump per 10 households. This still means walking to collect water. Mrs. Emsayakar joked that they can still use the handpump of their neighbours when they wish. She has not, however, had to do so yet as the harvested water meet all the needs of the family of 5, as long as they conserve water carefully. She also said, however, that she would prefer a piped / pumped supply which would mean that they could use as much water as they wish.

Technical detail

The tank

The tank is a 5m^3 below ground cylindrical brick construction based on the design of a Chinese biogas digester (see Figure 1 below). It has a diameter of 2.5m and a height of 1.3m to the base of the cover. The cover is a constructed using a clever brick dome design which can be left open to provide access. Water extraction is either by bucket, by handpump (more detail later) or by gravity through a pipe / tap arrangement where the topography and ground conditions are suitable. The cost of the tank is in the region of Rps.6,500 (UK£100). The construction details given to local masons are given below.

Figure 1 – detail drawing of the Sri Lankan brick dome tank (see Web Site)

The Chinese Brick Dome Tank – Construction details

1. Find suitable site
2. Dig pit 0.5m larger than the tank diameter
3. Plant an iron rod in the centre of the pit, making sure it is vertical.
4. Construct concrete base.
5. Start constructing walls using wire from iron rod to maintain the radius.
6. Once walls are complete backfill the gap between wall and pit with sand.
7. Make concrete ring beam to the shape shown. No reinforcing is required. Fit overflow pipe at this point if required.
8. Prepare two wooden sticks – one end an 'L' shape and the other a 'V' shape. The length of the stick is 2/3 that of the internal diameter of the tank.
9. Keeping the 'L' shaped end of the stick to top of the tank wall, place the 'V' end against the iron rod and wrap string or wire around the rod to support the stick.
10. Start to build the dome shaped roof of the tank with dry bricks.
11. To start, stick the first brick to the lintel with mortar and support it with the first stick.
12. For the second brick, stick this to the lintel and the first brick and support it with the second stick.
13. Push the third brick into place (with mortar) next to the second brick and move the second stick to hold the third brick.
14. Continue the process as with brick 3 until the first course is almost complete.
15. The final 'key' brick should be shaped to fit tightly allowing for the mortar.
16. Remove the sticks once the first course is complete.
17. Continue in this fashion for the subsequent courses.
18. The dome mouth is constructed in a similar way, but using the bricks length-ways.

19. Plaster the outside of the dome, then plaster the inside of the dome.
20. Plaster the inside of the tank.
21. Plaster the floor o the tank
22. Cure the tank by wetting for 7 – 10 days. Fill the gradually starting on day 7, filling at a rate of approximately 300mm per day.

Water proofing can be added to the mortar. This can be specialist additive or liquid dishwashing soap.

Water extraction is performed, at this sight, by two methods. A tap is fitted which allows water to flow by gravity from the tank, as shown if Figure 2. The second option is a simple handpump which has been developed, as part of the CWSSP programme, for use with below ground tanks. The pump is known as the Tamana pump, after the Pacific island on which it's predecessor was originally observed.

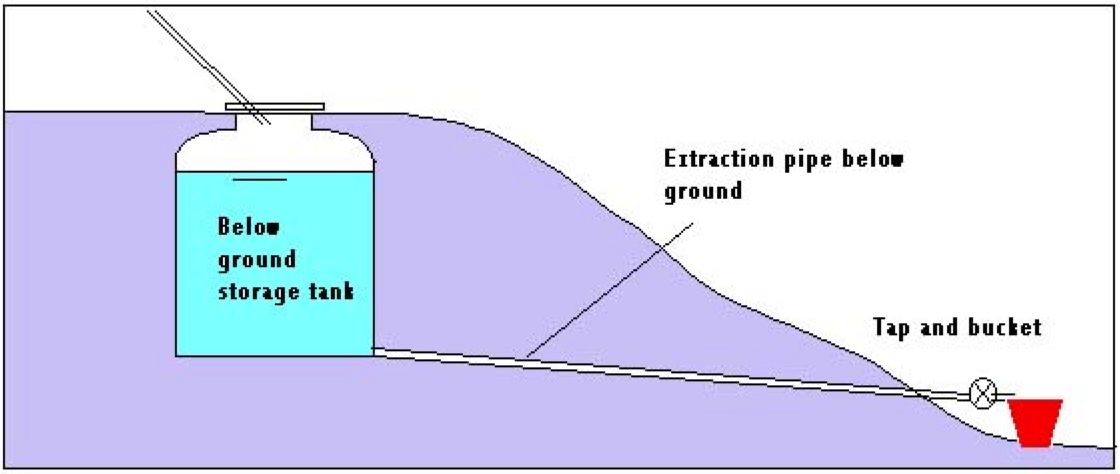


Figure 2 – Water is fed by gravity from the tank when the conditions are favourable

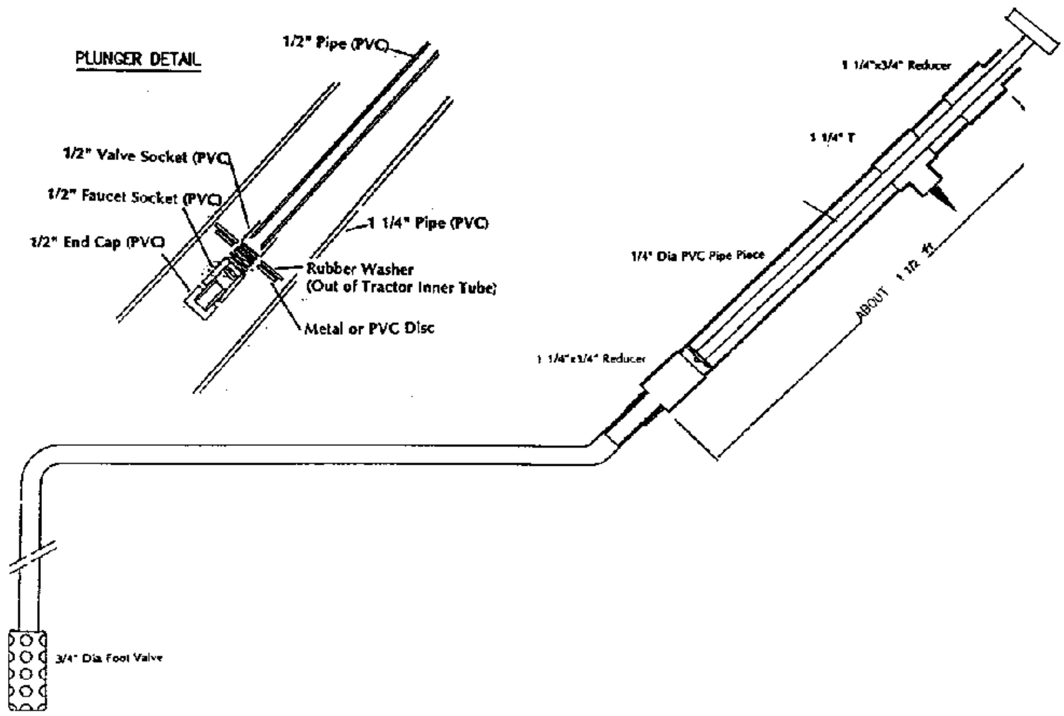


Figure 3 – The Tamana pump – design drawings

The Tamana pump is designed to be very low cost, approximately UK£5, using only locally available PVC fittings and rubber from a tractor inner tube. The location of the pump is shown in Figure 1 and technical details of the pump are shown in figure 3. This particular pump was fitted by the owners son, a mechanic, who has fitted many of these pumps for other community members. The pump has been brought via a ¾” PVC pipe to the kitchen of the house.

The first flush system is quite simple – the inlet chamber has a hole in its bottom, which is plugged with a bottle. When the bottle is removed water is allowed to flow away from the tank . The inlet chamber leads otherwise to a pre-filter chamber which contains layers of stone, charcoal and sand. The owner has experienced some problems with infestations of ants in this chamber. The inlet pipe to the tank has a protective mosquito mesh to stop mosquitoes entering and breeding in the tank.

Catchment area

The catchment area is the roof of the dwelling. This is a pitched roof of pan-tiles. Only one side of the roof is used. The other side is actually used to supply water for a neighbours tank which is situated at the other side of the house. The guttering is a factory manufactured U section type fitted to a facia board with specialist clips. The cost of the guttering is 1000 Rps. (UK£15.50). There is only about 8m of guttering for the 28m^2 of catchment surface.

User pattern

Average annual rainfall is 2600mm with a bimodal rainfall pattern and a dry season which lasts for 3 months. When properly managed the water collected can last throughout the dry period, with occasional trips to the nearby well for washing water. The average consumption rate for the whole family is about 75 litres per day but this is reduced during the dry season. The water is used for all domestic applications and there is no anxiety about the quality of the water, as is seen often where rainwater is used.

Item	Unit	Unit cost	Quantity	Cost (SL Rupees)
Cement	bag	310	8.5	2635
Sand	m^3	1700	0.4	680
¾ “ Metal bar	m^3	4000	0.1	400
Brick	Number	2.10	800	1680
Padlo cement	kg	100	0.5	50
Skilled labour	days	250	4	1000
Unskilled labour	days	150	12	1800
			Total	**8245**
The unskilled labour is often provided by the recipient hence reducing the cost of the tank.				

Case study 3

3.0m^3 brick built storage tank, Sri Lanka

Background

This case study is an example of local initiative in design and manufacture in DRWH. The tank in question was constructed in the village of Ahaspokuna, near Kandy, in the highlands area of Sri Lanka. The tank was built 10 years ago by a local mason for the Rajasomasari family and has since been copied so that there are now several of these tanks in the area. The setting is a high rainfall area (almost 3000mm annually) with a bimodal rainfall distribution. There is a dry season which lasts a maximum of 4 months. The Rajasomasari family fit the low to middle income range and their dwelling is a single storey bungalow with an electricity supply, latrine and shower room.

Technical detail

The tank

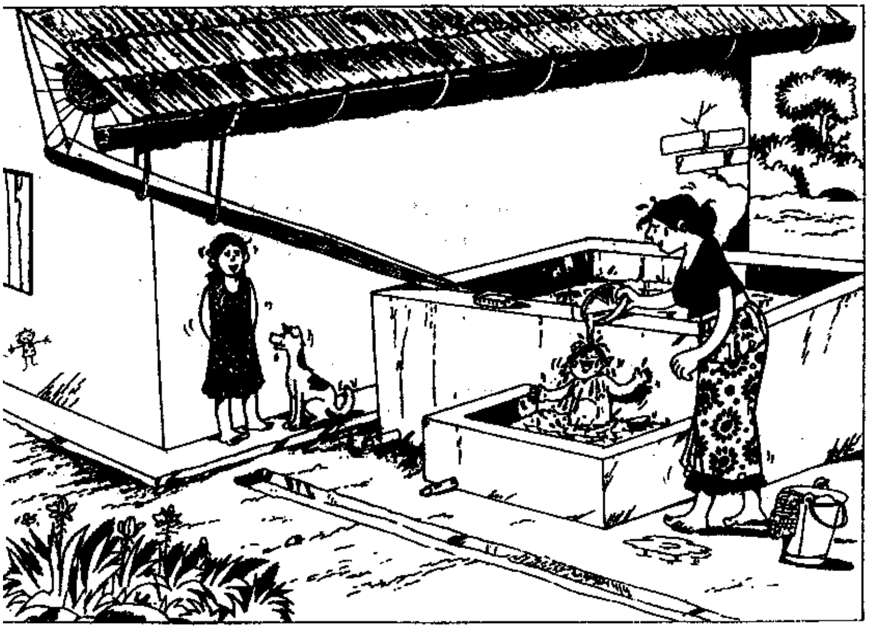
The tank is a simple brick built rectangular structure which has been cement rendered both inside and outside and sits at ground level. The tank has a concrete base. The cover is a removable wooden frame covered with a fine nylon mesh which filters out all larger debris such as leaves and twigs. The dimensions of the tank are 1.5 x 1.5 x 1.5m, giving a usable storage capacity of around 3m^3. The tank has an overflow and washout fitted. Small fish are kept in the tank, which helps to prevent algae growth and build up of organic materials. Water is extracted using a small bucket – there is a small removable section in the nylon mesh. The tank is usually cleaned when it is empty. The owner mentioned that the tank is very easy to clean because the top of the walls are only at chest height.

Catchment

The catchment area is the zinc sheet roofing of the house which totals about 90m^2. It is a pitched roof with a small gradient, say about 10^o. The guttering is, as is commonly found in Sri Lanka, S-Lon brand, U-channel, factory produced guttering as used for conventional rainwater protection /removal from houses. The cost of the guttering was approximately Rs. 5,000 (about £75.00). It is interesting to note that as much has been spent on the guttering as on the tank. Fittings for the guttering are also factory manufactured. The downpipe empties straight into the tank through the nylon mesh and can be diverted away from the tank as a first flush mechanism.

Water uses

The per capita consumption of water is in the region of 30 litres per day. They have a family of four people. The water is used for all domestic applications except drinking – water from the groundwater pump in the nearby valley is used for this purpose. The family is unsure of the cleanliness of the harvested water. The water in the tank lasts only about 15 days in the dry season, which is not very satisfactory in the eyes of the owner who would prefer a bigger tank.



Case study 4 – 10,000 litre partially below ground brick built tank, Sri Lanka

Background

This tank was built by Mr G. Victor A. Goonetilleke in the hill town of Kandy , Sri Lanka. Mr Goonetilleke decided to build his RWH tank after experiencing difficulty in sinking a well to sufficient depth to have a reliable perennial source of groundwater at the site of his newly built home. Drilling through the bedrock was too costly and there was no guarantee of securing a reliable supply. After 6 years of carrying water during the dry season Mr Goonetilleke started to research the idea of building a tank to store the rainwater that fell on his roof. There was very little encouragement from friends and neighbours who said that the water would provide an ideal site for mosquito breeding and algae growth. At the time there were no organisations to give advice on the benefits and drawbacks of roofwater harvesting. Three years after first contemplating the idea of a RWH tank, and after many helpful discussions with an Australian Radio Ham who convinced Mr Goonetilleke that RWH was a viable technolgy (and widely used in Australia), he decided to go ahead and build.

Technical details

The tank

Once the decision was made to build then certain design choices presented themselves. The determining factors for the tank capacity were:

sufficient capacity to store 100 litres of water for a period of 60 days
sufficient capacity to hold two bowser loads of water – during the dry season it is possible to purchase bowsers of water, but storage is required for this purpose. Each bowser contains 5,000 litres of water, but are not always available immediately upon request.

It was therefore decided to build a tank which would hold 10,000 litres. The next choice was what kind of tank to build. Mr. Goonetilleke had 3 obvious choices:

an underground tank - this type of tank needs excavation, care is needed to prevent roots penetrating the tank, contamination from ground can occur if not properly sealed, leaks are hard to detect and, finally, a strong cover is required to prevent children or animals from falling in. This type of tank does have the advantage, however, of being unobtrusive and benefitng from the support of the surrounding ground, making it cheaper to build.
a surface level tank – these require more space and can be ugly, but water is easier to extract under the influence of gravity and leakage is easier to detect. Covers need not be so sturdy, as little weight will be placed upon them.
an overhead tank – this type of tank is good in as much as little space is needed at ground level and the water is pressurised due to the ‘head’ of water. They are, however, expensive and it is difficult to transport water from the catchment system to the tank.

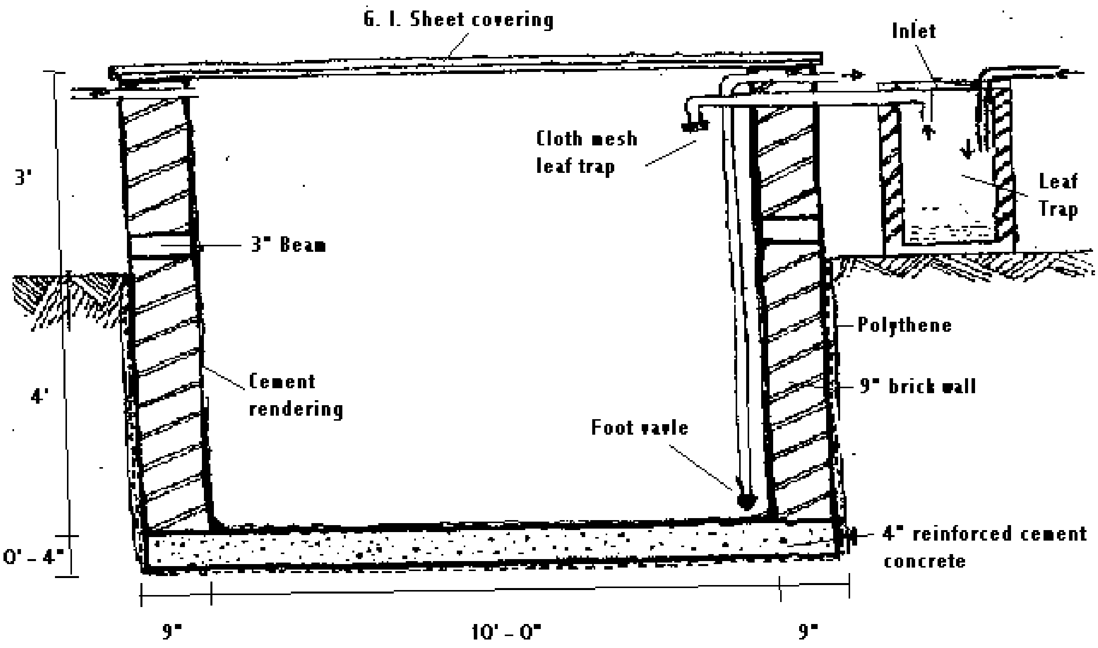


Figure 1 - sketch of 10m^3 brick-built tank.

The decision was made to build a tank which combined the advantages of the below ground and the surface tanks. Figure 1 shows a plan of the tank which was eventually built, partly below ground with 3 foot walls protruding from the surface of the ground. The next choice was what material to use for constructing the tank. Locally available plastic tanks were expensive and so Mr Goonitelleke decided to hire a mason to build a brick tank with a concrete base. Bricks are available locally.

The tank has a 4", 10 foot square, concrete base which is reinforced with ribbed steel bar. Polythene was laid underneath the concrete and brought up to ground level. The walls of the tank are of 9" brickwork. A 4" concrete ringbeam was cast at ground level to give added strength. The gap between the walls and the excavated pit were lined with concrete to allay fears of root penetration. The wall was then continued to 3 foot above ground level. The inside of the tank and exposed external walls of the tank were rendered – no waterproofing additive was added. The tank was covered with some galvanised steel sheets. A pump was fitted to pump water to a header tank situated in a tower near the house (which also houses an old 200 litre oil drum which collects rainwater for irrigating the garden). The overflow from the main tank goes into a shallow ditch where there is a flourishing stand of bamboo.

The total cost of the system was around Rs. 25,000 (US$550), but Mr Goonetilleke says that minimum cost was not the primary objective.

The roof and guttering
The roofing material is asbestos sheeting with an area of about 2000 square feet being used for catchment which is half the total roof area. S-lon brand, PVC, U-channel guttering is used to catch the water and the downpipe leads to a 1" PVC pipe (not ideal but it was freely available at the time) which then transports the water to the tank. The first flush system is a simple connector which enables the pipe to be diverted to the garden pond. The filtration system is simply a piece of mesh and some discarded mosquito net, but the aim is to improve on this.

Maintenance
During the 4 years the tank has been in use there has been no need for any major repair. The tank is cleaned once or twice a year and the cover is swept of leaves and dust regularly. Internal inspection of the tank is easy because of the low walls.

Uses for the harvested water
The water is used for mainly for washing and bathing and is occasionally used for drinking, but is then boiled. During the dry season waste (grey) water is used for watering the plants. There is little need to be overly conservative with the water because it is possible to order a bowser when the water level gets low (4500 litres at a cost of approximately US$7.00), although so far this has not been necessary.

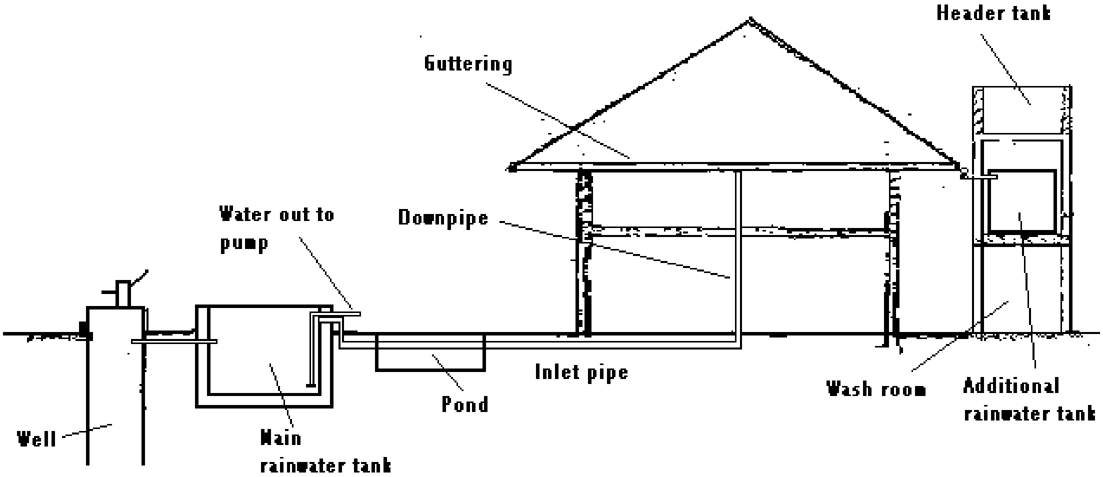


Figure 2 – Sketch showing the whole scheme including the water tower

Mr. Goonetilleke gives the following suggestions for improved awareness of RWH techniques:

- better information dissemination and educational awareness should be carried out at all levels.
- where possible, credit and technical advice should be made available in conjunction with other incentives.
- more concern should be given to the improper application of treated water – there is no need for water to be of exceptionally high standards if it to be used for clothes washing or bathing.
- architects should be aware of the principles of RWH and incorporate the technique in the house design where this is appropriate.
- care and attention are necessary (moreso than money), to maintain and improve the quality of harvested water.
- there are many myths associated with the concept of RHW which can be easily dispelled when the technology is put into practice.

Source: Mr. G. Victor A. Goonetilleke, Rainwater Harvesting, a case from Pilyandala, Proceedings of the Symposium on Rainwater Harvesting for Water Security, February 1998, Lanka Rainwater Harvesting Forum and the Open University of Sri Lanka.

Case study 5 – 10m^3 ferrocement tank, Nagercoil, India

Introduction and background

This RWH system is an example of a suburban solution to inadequate water supply from the municipal authorities. Although the family have a piped supply connected to the main town supply, the reliability of this supply is very poor – the piped supply provides water only every 8 to 10 days and the quality of the water is questionable. The family also has a groundwater supply but this, too, is unreliable. Their solution was to construct a RWH tank and harvest water from their roof. The traditional Nalluketta building style of this area, as shown in Figure 1, lends itself well to RWH. The central rooftop courtyard above a single storey dwelling makes an ideal collection area.

The system was installed as part of a programme run by the Centre for Appropriate Technology, an n.g.o. based in the town of Nagercoil. They have installed more that 100 such systems in the area.

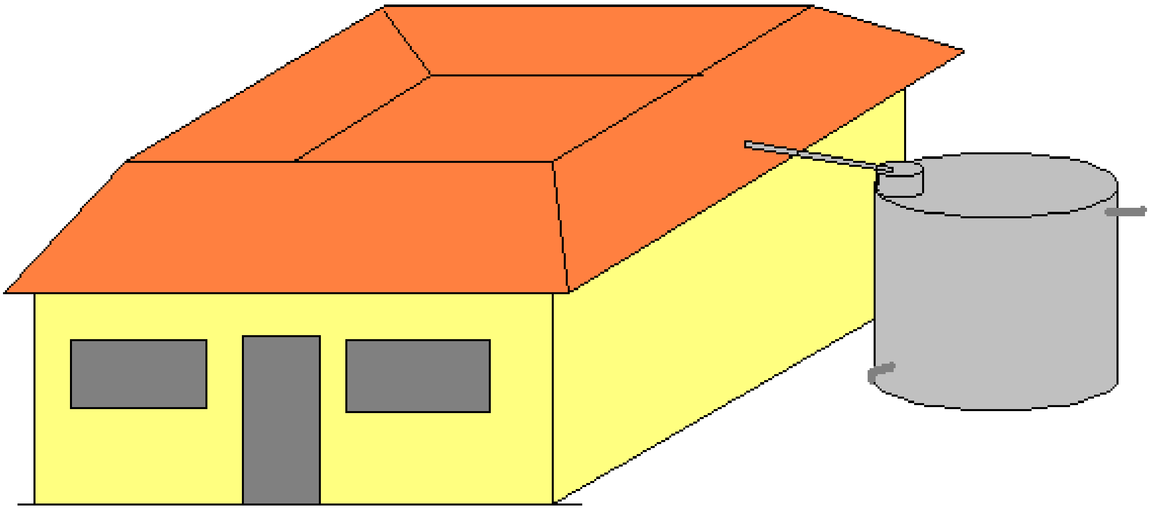


Figure 1. Traditional Nalluketta building style

Technical detail

Tank details

The tank is a cylindrical ferrocement structure, with a diameter of 2.5m and a height of 2m. There was little detail available regarding construction details but it appears from the corrugations on the exterior that a zinc sheet mould had been used to cast the tank. The tank is set on an 18” concrete plinth which means that a bucket can be placed comfortably under the extraction tap while still keeping the extraction pipe near the bottom of the storage area, therefore not wasting any storage capacity. The tank cover is formed using corrugated asbestos sheets. The tank is fitted with an overflow pipe near cover level.

First flush is achieved by simply moving the down pipe away from the inlet basin. There is a pre-filter basin which sits on top of the cover. It contains a plastic bowl which has been punctured repeatedly to allow water to pass through, as with a sieve. The bowl is filled with small stones and sand which acts as a filter. The bowl and basin have been cemented in place to prevent water entering the tank through the joints. A water level ‘sight tube’ has been fitted but is too discoloured by sun and silt to be of any use.

Catchment area

The catchment area, as mentioned earlier, is the rooftop of the house. Because the rooftop is rather unusual in design (see Figure 1) and the collection area is the internal rooftop courtyard, it is required that the downpipe pass through the roof structure (see Figure 3). This can present problems if sealing is not effectual. The catchment could take place on the outer faces of the pitched roof but this would entail fitting guttering and facia board, all extra expense. The catchment area is approximately 100m^2.

The pitched section of the roof slopes at approximately 35^o and is of pantiles. The central rooftop courtyard area is mostly flat with a a vriety of tiled and cement rendered surface. There are many trees overhanging the roof and it quickly becomes covered in leaves which could block the downpipe.

(Figure 2 photo – catchment area)

Annual rainfall in the region is about 1200mm. Normally there is a single wet season with a dry season which lasts about 4 months.

User pattern
The catchment is about twice the size required to fill the tank, so the tank can be filled early in the rainy season. The families procedure during the rains is:
to sweep the roof clean and divert the first rain runoff
to fill the tank early in the wet season
to then seal the tank (e.g. tape up the entry against mosquito entry)and divert flow away from it
to use the water for premium quality purposes only during the following dry season: the stored water usually lasts until the next rains

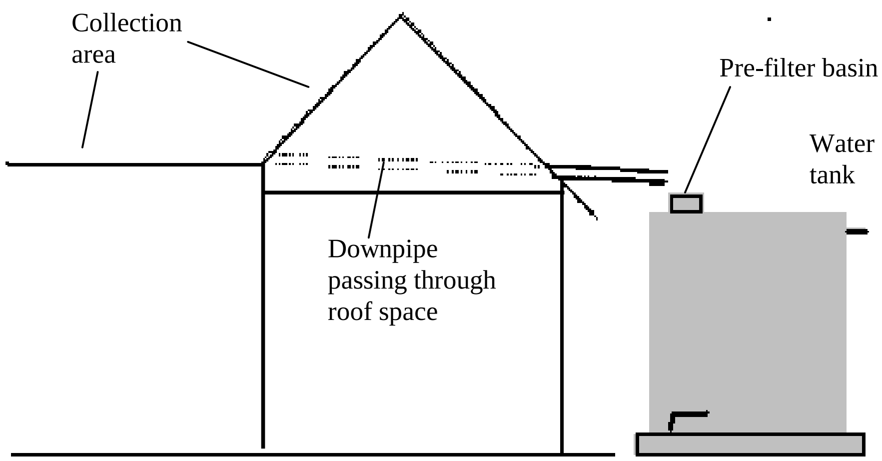


Figure 3. Cross section of the building showing the downpipe passing through the roof structure

Case study 6 – Below ground low-cost water storage cistern – 4 to 10 m^3 – Uganda

Introduction

This tank (or cistern) was developed in Uganda by members of the Development Technology Unit, Warwick University and members of the Uganda Rural Development and Training Programme (URDT), between 1995 and 1997. Work is still continuing on the refinement of the tank. URDT is a service NGO located at Kagadi in Mid-Western Uganda. Several of these cisterns were built and tested with the aim of developing a low-cost (under US$150), all-year, domestic, water storage technology for the surrounding region. The information for this Case Study is taken from a document titled 'Underground storage of rainwater for domestic use' by T. H. Thomas and B. McGeever, which is available as a working paper from the Development Technology Unit (see the list of partners on our home page).

Uganda is well suited to RWH practice for several reasons:
rainwater harvesting is a technology which is traditional to Uganda, albeit on a very ad hoc, very low-tech basis, e.g. buckets under the eaves to catch water during storms, or old 200 litre oild drums used for short term storage.
it has a bimodal rainfall pattern with very short dry seasons which are rarely completely dry.
annual rainfall in many parts of the country is in excess of 1200mm, which means that even the smallest house would have sufficient roof collection area to provide sufficient rainwater to meet demand (based on 15litres per capita per day).
corrugated iron roofs are becoming common, even in rural areas .
the lateritic soils in the area make well sinking a difficult task (yet provide ideal ground conditions for below ground tank construction).
there are many hilly areas where water (for irrigation and domestic use) has to be carried uphill from the valleys.
gravity-fed piped water is rare outside the main towns both because it is technically difficult (absence of strong high level springs, lack of mains electricity) and because the organisation to install and operate gravity water supplies is lacking in rural areas.

Ntale[1996] carried out a study of costs of existing water storage technologies based on a tank capacity of 8000litres. The results are shown below.
$340 in total for unreinforced mortar jars (at least 4 jars),
$390 for a brickwork tank, 50% more if reinforcing is deemed necessary,
$450 for a galvanised iron tank,
$1432 for a PVC tank,
$480 to $880 (various sources for E Africa) for a ferrocement tank,
$182 (quoted from Brazil) for a plastered tank of stabilised rammed earth, a material currently hardly known in Uganda..
These sums seem generally beyond the purchasing capacity of Ugandan rural households where even finding $200 for an iron roof is often not possible, although the last technique has promise.

Technical detail

Materials, tools and skills

The paper describes how to make a 6,000 to 10,000 litre underground cistern, suitable for construction where the soil is firm and hard but not rocky. *Variant A* has a 20 mm thick cement-mortar dome (mix = 1:3), a 25 mm cement mortar lining to its Chamber, and employs a little chicken mesh reinforcing. *Variant B* has a 20 mm cement/lime-plastered Chamber. Both variants have similar shapes and construction procedures. The materials necessary for the tank's construction meet the test of ready availability even in African small towns. They are, for an 8,000 litre cistern:

Material	Quantities	
	Variant A	Variant B
bags (ea. 50 kg) cement	$5^1/_2$	$3^1/_2$
bags (ea. 25 kg) lime	0	3
wheelbarrows of sand	15	15
lengths (ea. 12m) of 6mm reinforcing bar	1	1

chicken mesh (1.8m width)	1.5m	0
plastic bucket, say 10 litre	1	1

(also wood to make the template mentioned under *Step 1* below - 130 cm x 100 cm thin ply or 3m x 300 mm x 20 mm plank - and a large plastic washing bowl)

The tools needed for tank production are:

- ♦ digging and plastering tools
- ♦ a large plastic basin (say 45 cm diameter)
- ♦ a bucket on a rope for lifting out soil
- ♦ a spirit level
- ♦ a template for the dome (see *Step 1*)

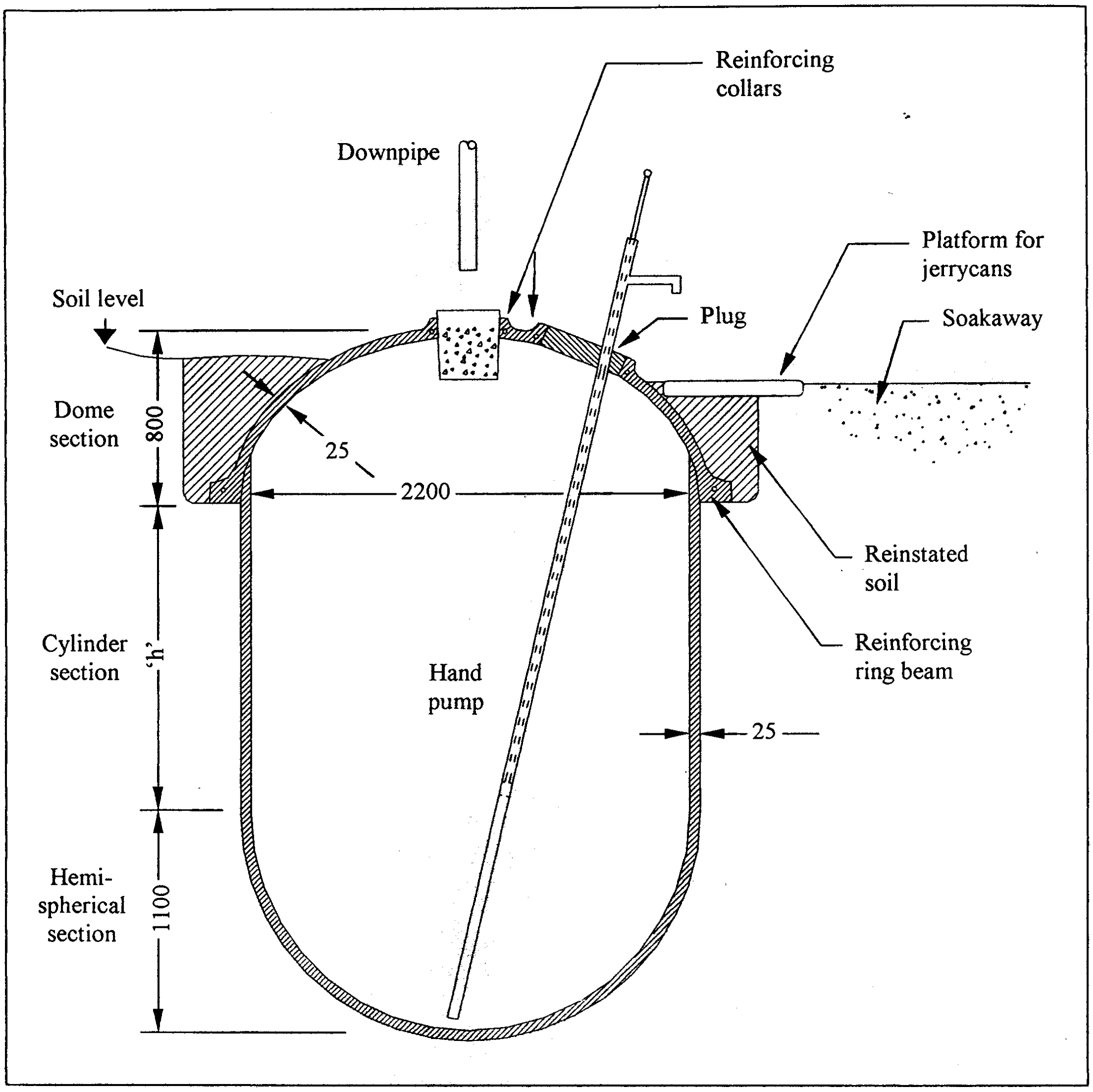


Figure 1 - General side view of cistern with pump

Parts of the DTU/URDT rainwater storage cistern and steps in its construction

The cistern is divided into four parts, namely the Chamber, the Cover, the Pump and Extras. Figure 1 shows a sectioned-elevation view of the tank and pump (what you would see if you could dig it out and cut it in half from top to bottom).

The Chamber has to have adequate volume and be waterproof. Because the overall cost of a cistern is dominated by the cost of the walls and cover, these should be as small as possible. For a given cistern volume, their total area is a minimum, for either a rectangular or cylindrical tank, when the tank's depth equals its width.

However for certain sorts of cover it is difficult to span widths of more than say 7 feet (2.2 meters). The cistern we are about to describe has a rounded Cover and a rounded bottom and has an internal diameter of 2.2 meters. The depth of the straight part of its sides for different capacities is as follows:

usable capacity in litres	4,000	6,000	8,000	10,000
depth of cylindrical sides	0 meters	0.5 m	1.0 m	1.5 m
depth from dome to bottom	1.9 m	2.4 m	2.9 m	3.4 m

The Cover has to stop the water from evaporating, keep the water clean, prevent anyone falling into it and keep out light and mosquitoes. It has to be pierced by a big hole to let the rainwater in very rapidly and smaller hole through which water can be pumped out. These holes must also be mosquito and light proof, and at least one of them must be large enough for a man to squeeze through in order to inspect or replaster the inside of the tank. It is recommended that the Chamber is excavated through the main hole in the Cover. This method allows the cover to be cast easily in situ without the need for shuttering or special tools. An earth mound is constructed for this purpose below ground level, as shown in Figure XX. The Cover should be shaped so that it leads any run-off from nearby ground away from its inlet. It must be strong enough to bear the weight of many people, provided that it has been covered with earth so that only the top of the dome is above the ground.

The pump. Thomas and McGeever discuss the requirements of a handpump for poor rural communities in Uganda:

A pump for a household cistern should

be cheap (in Uganda a ceiling price of USh.15,000 = $US15 was chosen);

permit an adult to raise 10 litres per minute (a rate generally obtained from protected wells) from a depth of 4 meters without undue effort and also be usable by a child of 6 years;

be 'self-priming', delivering water within a few strokes of starting to pump even when the pump has been out of use for some days;

reach water within 20 cm of the bottom of a tank;

fit into the mortar plug in the cover (dome) of a cistern so that light, mosquitoes and surface water cannot enter, yet permit the riser pipe and foot valve to be withdrawn through that plug whenever they need any maintenance;

lift at least 100,000 litres under household conditions of use before requiring replacement;

lift at least 10,000 litres before requiring maintenance, all such maintenance being possible using skills and materials available in most African villages;

be economically manufacturable in each country of use;

discharge conveniently into a jerrycan or other collection vessel.

In addition it is desirable that

♦ the foot valve does not leak faster than 0.1 litre per minute, so that if the pump is used twice within say 10 minutes it does not have to be (self) re-primed for the second use;
♦ the intake is constrained to avoid drawing up sediment in the tank by being located say 10 cm above the tank bottom; however for cleaning purposes it is helpful if dirtied wash water can be lifted from as little as 2 cm from the tank bottom.

Some development of a handpump which aimed at achieving this specification was carried out, but the authors feel that it was far from ideal. We will not, therefore, consider this pump in this case study.

The Extras include some means of seeing the water level inside the tank without having to open the Cover, a coarse filter for water entering the tank and provision for safe disposal of any overflow water. There is some interest in putting a layer of sand at the bottom of the tank as an output filter, however this would require the pump intake to be connected to a perforated pipe running under the sand. (Experiments to test such a filter's performance have yet to be done.)

During construction of any cistern, there are three choices in how one might combine the Cover and the Chamber. In some cistern designs, the Chamber is dug first and then the Cover built over the Chamber. In other designs the Cover and Chamber are made side-by-side and then the cover is lifted onto the top of the Chamber. For our design, we recommend a third method: the Cover is made first (in its final position at ground level) and then the Chamber is dug through an access hole in the Cover. It is not too difficult to do this if excavation is manual (although the procedure effectively excludes mechanical excavation and is therefore not recommended for high-wage countries) and it allows the use a cheaper dome-shaped Cover than if the cover had to be lifted. So the sequence for construction is as follows:

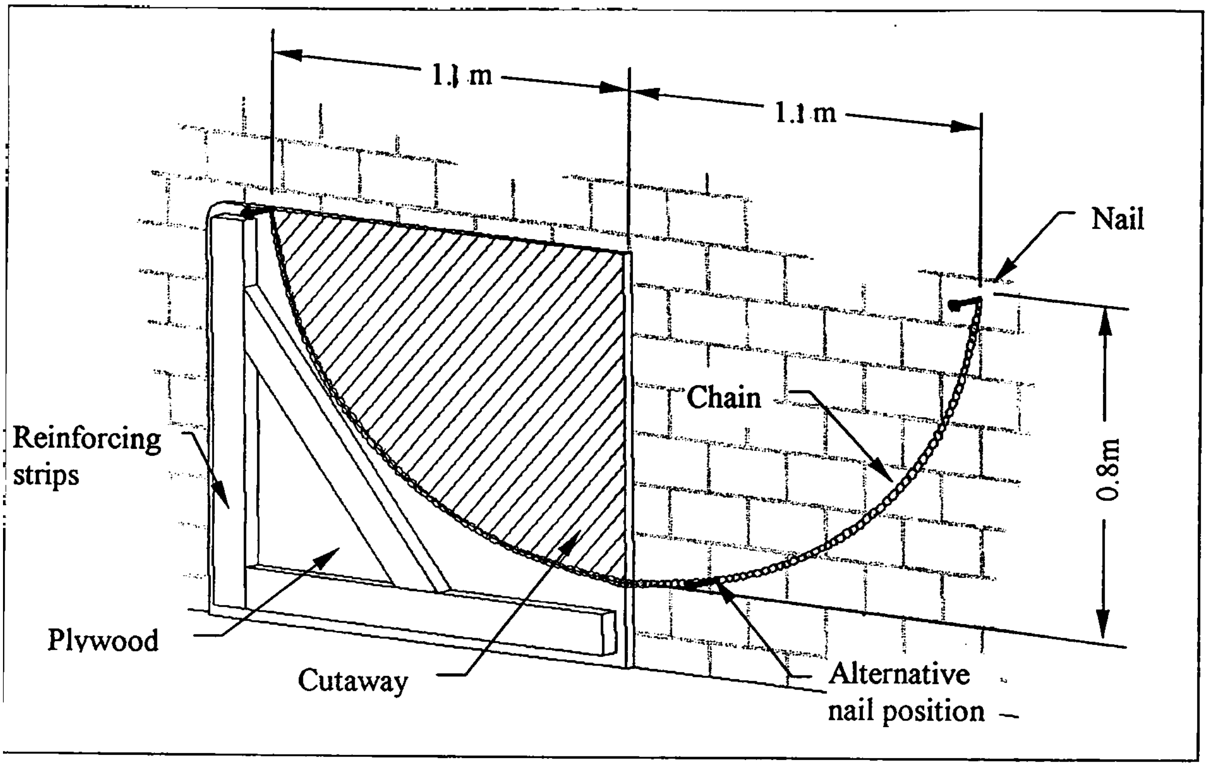


Figure 3 Making the template

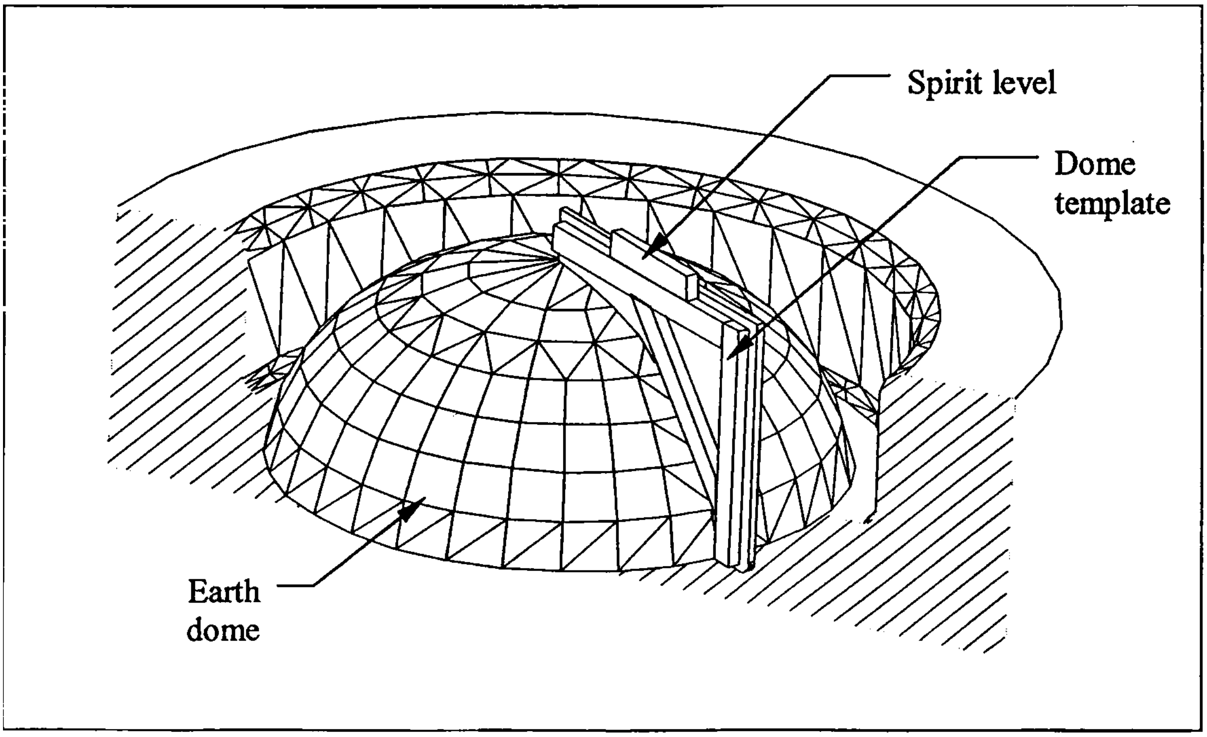


Figure 4 Forming the earth dome-mound

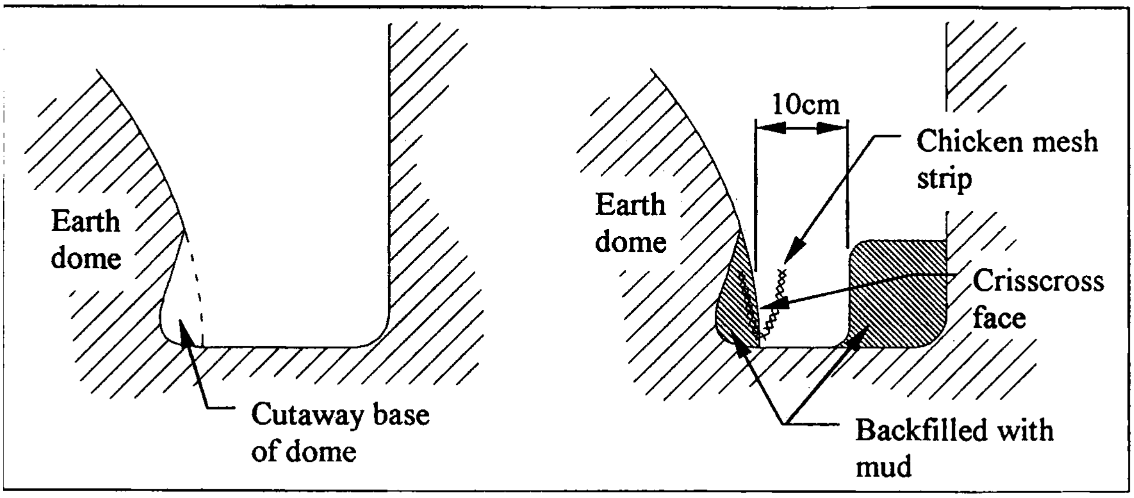


Figure 5 – Detail of joint between dome and wall

- Prepare reinforcing bar (and perhaps mesh) to place in the trench and round each hole in the dome
- Place mortar to form the ring beam and the dome with its two holes
- Cure the mortar then cover the dome with soil
- Through the larger hole dig out the Chamber
- Plaster the inside of the chamber and allow this plaster to cure
- Make the pump
- Set the pump into the dome
- Construct the tank inlet with its gravel filter

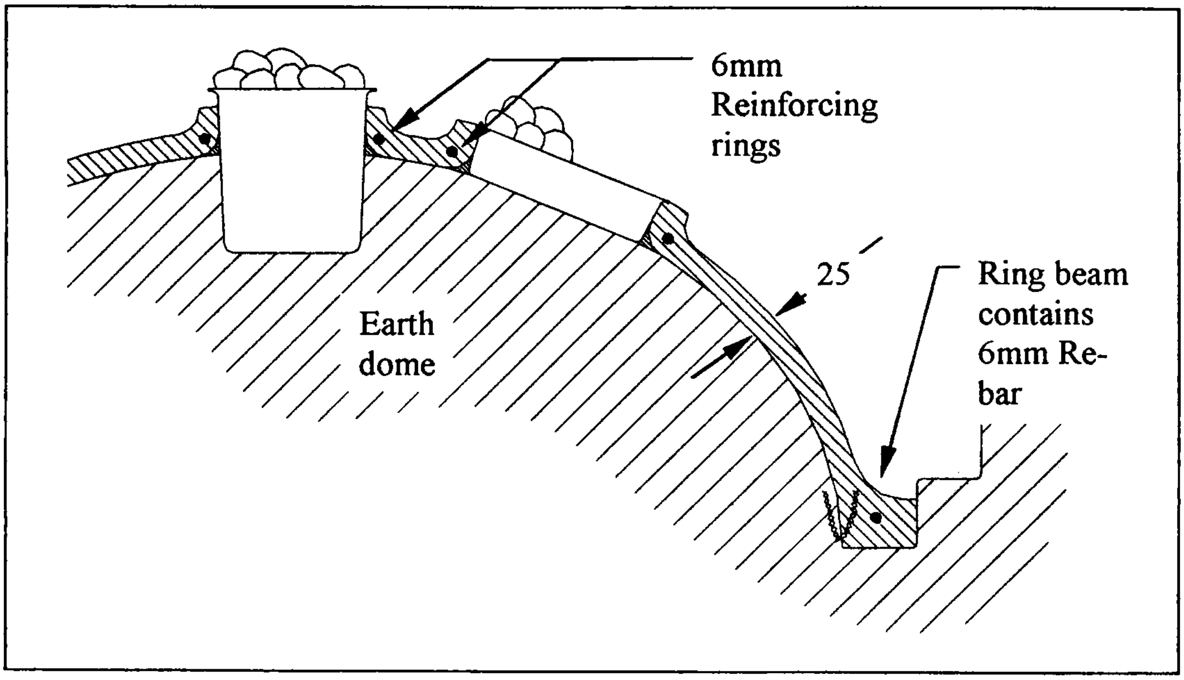


Figure 6 - Water inlet with coarse gravel filter

Provide drainage and arrange the hard-standing for pumper and water containers

The tank takes about 24 ‘man-days’ to construct. However the mortar dome and later the plaster in the chamber should each be left to cure for 2 weeks, so it needs a minimum of 6 weeks from when construction starts to when the tank can be used. Most of the work is digging but for 2 days an experienced plasterer is required. The pump can be made in a few hours.

Further work and field trials

Three tanks of 8000 litres were built and tested in the town of Kagadi. Tests on dome strength, leakage and chamber integrity and flexure were carried out and the results were very reassuring. Tests were also carried out a very low-cost pump design which proved to be unreliable and has therefore not been included in this Case Study.

Tank costs

Cistern costs (8,000 litre capacity with 20 mm dome and 2-coat chamber lining)

Item	*Quantity*	*Cost (US$)*
Cement/lime (including transport)	250 kg	65
Sand (assumed from a nearby source)	18 wheel-barrows	3
6 mm reinforcing bar	12m	5
Chicken mesh	3 m^2	4
PVC Bucket + 0.5 m of 50 mm piping		3
Unskilled labour for digging (9 m^3) etc.	20 person days	40
Plasterer	2 person days	8
Supervisor + say 25 km travel	1 person day	10
Tools (say)		5
	Total	143

Design variants

Some design variations have been discussed in this paper.
The dome of the tanks built in 1996 were 25 to 30 mm thick. Those recently tested for strength were 20 mm thick and performed well. 20 mm will be used henceforth as a norm. Moreover both mortar and concrete have been used for the dome. Concrete uses less cement, but requires fine aggregate (which is not widely available in rural areas) and is much harder to work smoothly as a plaster. There is some danger that these workability problems could lead to serious cracks in inexperienced hands. The mortar dome looks better. Mortar is more vulnerable than concrete to shrinkage during curing, but this should not matter in a largely unconstrained dome. On balance we recommend mortar despite the 33% higher cement requirement.
The chambers of the 1996 cisterns were single plastered to a thickness of 30 mm. The later tanks are using 20 mm applied as two layers (e.g. 15 mm plus 5 mm) rather than one. The tank most in danger of earth tremors has just been plastered with a 2-layer lime-cement mortar; it may take some years before the benefits of using this slower curing but more flexible plaster can be assessed.

Case study 7 – 10m³ ferrocement water tank using former

Introduction

This example has been taken from 'Ferrocement water tanks and their construction' by S. B. Watt and published by Intermediate Technology Publications (more information about this book and this publisher is given in the information section later in this document). All figures in the Case Study are taken from this book.

Watt states that these tanks have been used for many years in parts of Africa and have been designed to be as simple as possible to build in self-help programmes. The users, who are at first unskilled in this sort of construction, can contribute their time and efforts in collecting sand and water, digging the foundations and preparing the mortar under the general guidance of a trained builder. With experience they quickly learn how to make the tanks without further guidance.
A trained builder with 5 helpers takes approximately 3 days to complete the tank. The users often contribute some money towards the cost of the tank, which helps to cover the builders' wages, the cement, reinforcement and the hire of the formwork.

Design

The tanks have been designed for construction by relatively unskilled workers. They have a diameter of 2.5m, a height of 2m, giving a capacity of 10cubic metres. The final wall thickness will be about 4cms. The tanks are built on site and should not be moved.

Formwork

The 2m high formwork is made from 16 sheets of standard galvanised roofing iron, 0.6mm thick with 7.5cm corrugations rolled into a cylinder with a radius of 1.25m. Steel angle iron (40 x 40 x 5mm) is bolted vertically on the inside face at the ends of each set of 4 sheets – this allows the sheets to be bolted together to form a circle. Between the edge of each section is placed a wedge which is pulled out to allow the formwork to be dismantled (see Figure1).

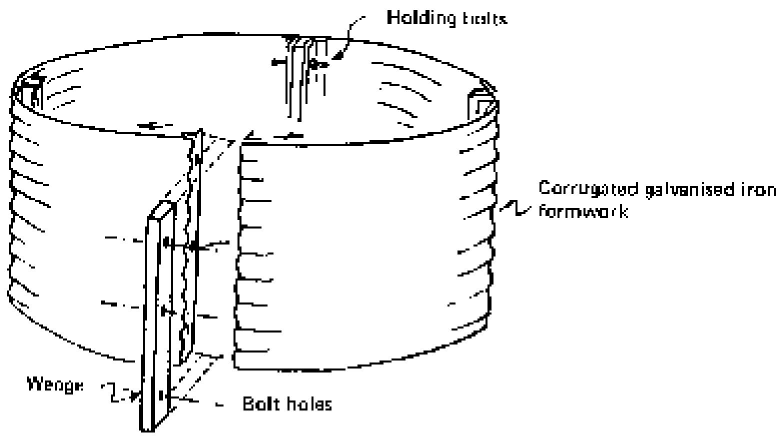


Figure 1 – Assembling the formwork

Construction

A circular area 2.8m in diameter is cleared at the required site for the tank and excavated down through the loose topsoil. A 10cm layer of sand is laid evenly over the excavation and a 7.5cm layer of concrete mix of 1:2:4 (cement:sand:gravel by volume) will form the foundation slab under the tank. Into this concrete foundation is cast a 1m length of 20mm bore steel water pipe with a tap on the outside end. The pipe is curved so that it projects 10cm above the floor of the tank; a piece of wire is threaded through the pipe to act as a pull through after the tank has been built (see Figure 2).

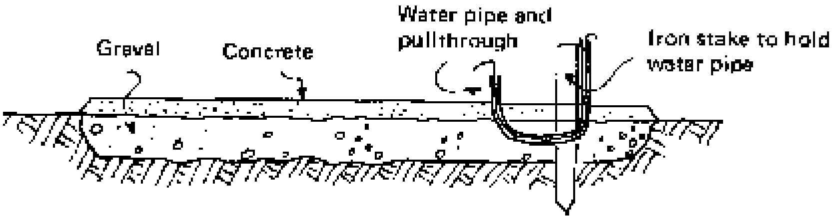


Figure 2 – Foundation of tank

When this concrete slab has hardened the formwork of the tank is erected. The bolts passing through the angle iron and wedges are tightened to provide a rigid cylindrical form. This is cleaned free of cement and dirt, oiled and the wire netting wrapped around it to a single thickness and tucked under the forms. The netting has a 50mm mesh, and is made from 1.0mm wire.
To form the hoop reinforcements, the straight galvanised iron wire, 2.5mm diameter, is wound tightly around the tank from the base at the following spaces:
2 wires in each corrugation for the first eight corrugations
1 wire in each corrugation for the remaining corrugations
2 wires in the top corrugation

About 200m of 2.5mm diameter wire will be needed, weight 8 kg. The netting provides vertical reinforcement to the tank and also holds the hoop wire out of the corrugations. The outside is then plastered with a layer of mortar made from a mix of 1:3 (cement : sand by volume)and as soon as this has begun to stiffen a second layer of mortar is trowelled on to cover the reinforcing wire to a depth of 15mm. The surface is finished smooth with a wooden float.
After a day or so the formwork is dismantled by removing the holding bolts and by pulling out the wedges which will leave the shuttering free to be stripped away from the inside mortar wall. The sections are lifted clear of the tank to be thoroughly cleaned of any mortar or cement. A 20cm length of 8cm diameter downpipe is built into the wall at the top of the tank to act as an overflow and the inside of the tank of plastered with mortatr to fill up the corrugations. When this has hardened sufficiently, a second final coat is trowelled onto the inside and finished with a wooden float.
A 5cm thick layer of mortar is next laid onto the floor of the tank and the junction of the floor with the walls built into a coving. The floors are not reinforced and so the tank wold fracture if it were moved. Take care that the mortar does not fill up the outlet pipe. Before the mortar on the floor has stiffened, form a shallow depression in the middle; this will allow the tank to be cleaned at a later date – the sediment can be brushed to the hole and cupped out (see Figure 3).

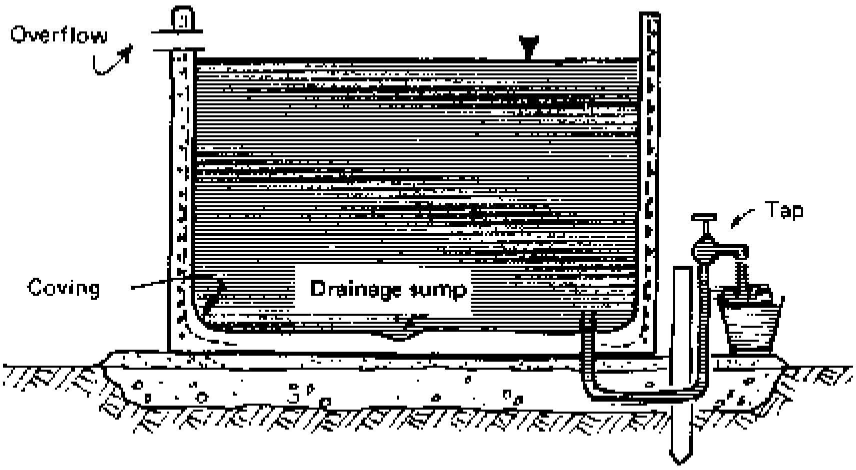


Figure 3 – The completed tank

The inside of the tank is painted with a thick cement slurry to seal the inside of the tank the a small of water is allowed to stand in the bottom of the tank and the tank is covered and cured for seven days.

Roof

The tank is covered with sheets of 0.5mm galvanised sheeting supported on two lengths of angle iron. Alternatively, a reinforced mortar roof may be built in the way described in Case Study 8 (Factory Made Tanks – New Zealand). Building a mortar roof is not difficult but does require more formwork.

Materials

Materials required for a 10m^3 tank with galvanised iron roof.

Cement	600 kg
Plain wire 2.5mm diameter	200m
Chicken mesh – 1m wide	16m
Water pipe – 20mm bore	1m
Water tap	1 No
Overflow pipe – 8cm bore iron or concrete	0.2m
Sand	1.0m^3
Gravel	0.5m^3
Galvanised iron sheet and angle iron	

Case study 8 – Factory made tanks, 1 to 25m³, New Zealand

This example has been taken from 'Ferrocement water tanks and their construction' by S. B. Watt and published by Intermediate Technology Publications (http://www.oneworld.org/itdg/publications.html). All figures taken from this publication also.

Introduction

Ferrocement tanks have been made commercially in New Zealand for many years and have now largely replaced the traditional corrugated galvanised iron tanks. They are used mainly to store water for domestic and dairy purposes on the farm but they are also winning acceptance for industrial liquid storage. The cost of the smaller tanks is comparable with that of tanks made from other materials such as galvanised iron; the cost per unit volume decreases rapidly with increase in size.

Tank sizes

The tanks are constructed in various sizes, with capacities from $1m^3$ to $25m^3$, diameters from 1m to 3.6m, and heights from 1.3m to 2.9m. With specially built formwork and machine mortar mixers each tank takes 2 – 5 person days to build. There are several manufacturers producing such tanks in New Zealand. The high wage costs in New Zealand are reflected in the prices of the tanks.

Design

The water pressure in a tank full of water generates stresses in a tank that are difficult to calculate structurally. The tanks have been designed to resist only hoop stresses and a layer of woven netting is included as nominal reinforcement; this netting in fact provides the only reinforcement at the base of the wall where it joins with the floor – the point of greatest stress. This section is thickened during construction and from information given by the manufacturers there is no evidence that cracks appear under normal loads. The only causes of failure have resulted from damage during delivery.
All the tanks are built with an integral roof and a covered access hatch.

Capacity (m^3)	Diameter (m)	Height (m)	Weight (tonne)
0.9	1.2	1.3	0.25
1.8	1.55	1.3	0.3
2.7	1.85	1.3	0.45
3.6	2.0	1.45	0.8
4.5	2.0	1.95	1.25
9.0	2.9	1.95	2.1
13.5	2.9	2.6	3.0
18.0	3.65	2.6	4.0
22.5	3.65	2.9	5.0

Table 1 – Size and weight of New Zealand tanks

Cement	740 kg
Sand	$1.0m^3$
Plain wire – 4mm diameter	330m
Wire mesh – 1m wide	28m
Weld mesh for slab	$7m^2$

Table 2 – Materials needed to make a $9m^3$ tank

The quantities shown in Table 2 are higher than a comparable tank, which would be built in situ. The tanks described here have to be stronger than the self-help tanks to be able to withstand the extra stresses during transportation

Construction

The tanks are constructed on special fabricated steel formwork which is quickly erected (see fig 1), or on a temporary timber formwork. Usually, the floor of the tank is cast first; this is reinforced with welded steel mesh made from 8mm diameter rods at 20cm centres with a floor thickness between 6cm and 10cm, depending on the size of the tank. Loops of 8mm steel are allowed to project from the base to allow for easy handling; this also reduces the stresses that will be set up in the tanks as they are being lifted or winched. A strip of chicken wire is also cast into the sides of the floor and is bent up into the walls.

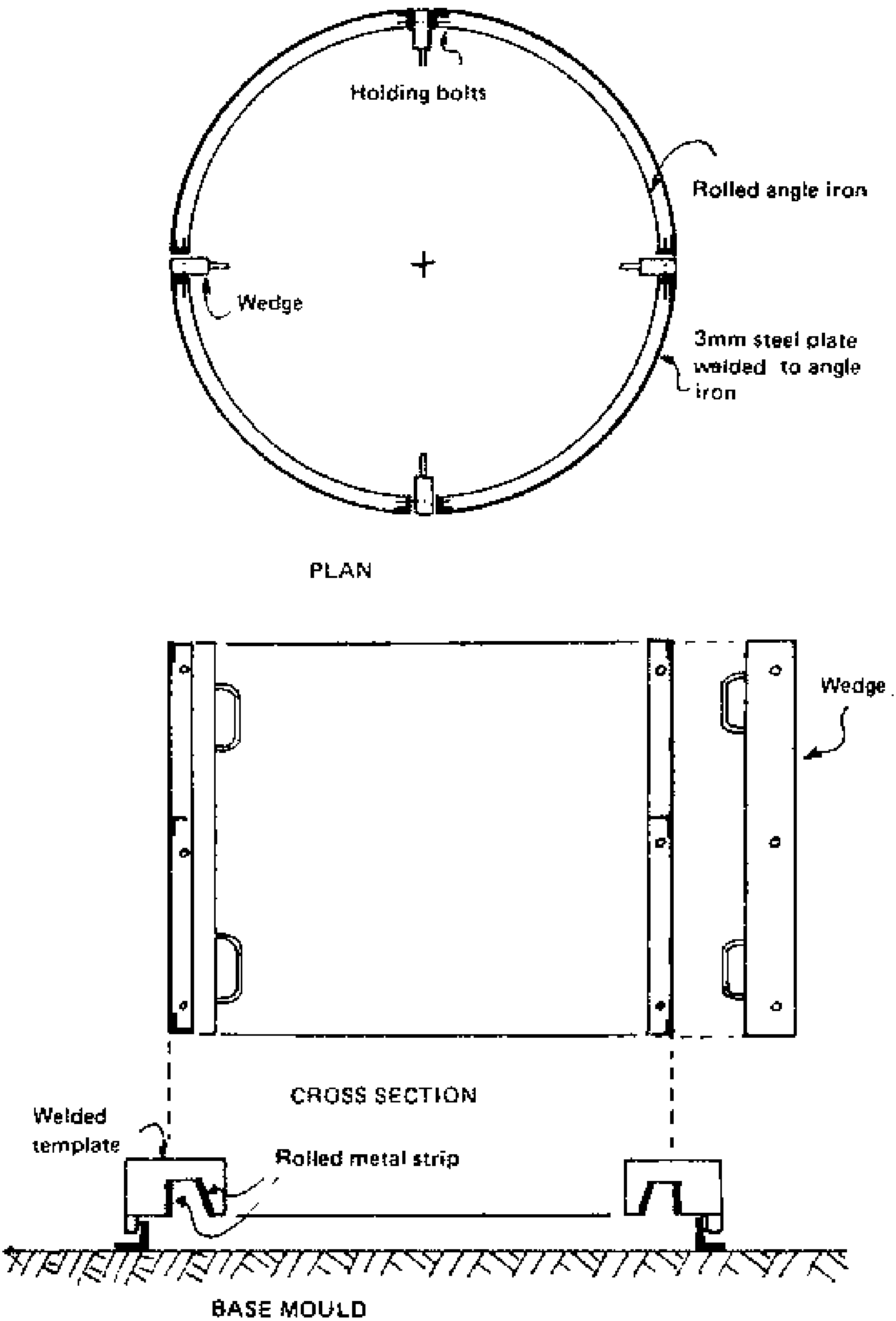


Figure 1 – formwork used for tank construction

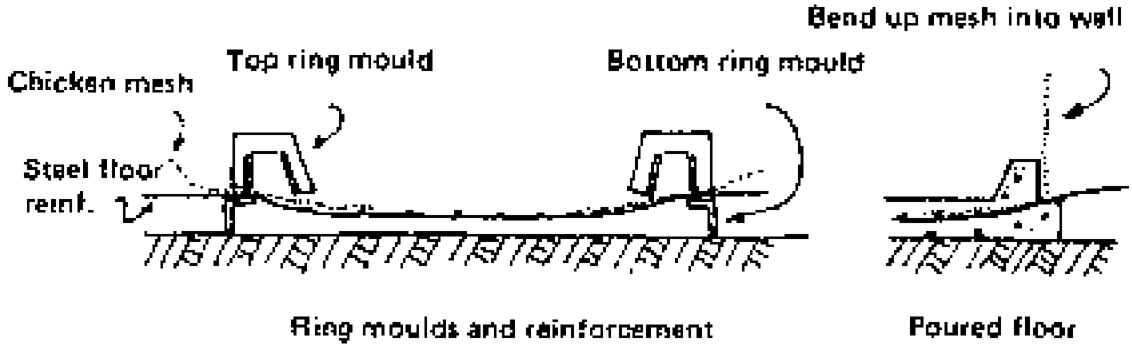


Figure 2 – Casting the base slab

When the floor slab has been cast, the formwork is erected and the chicken wire folded up against the shuttering. A layer of chicken wire or weld mesh made from 2mm wire at 5cm centres is wrapped around the tank to cover the shuttering from top to bottom (see Figure 3).

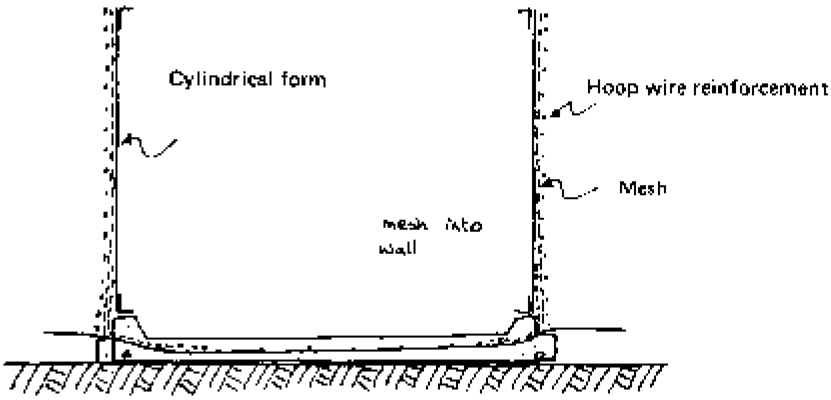


Figure 3 – Assembling the formwork and reinforcement

The main reinforcement, 4mm diameter straight wire, is wrapped tightly around the tank in a spiral with a 5cm gap between the wires. Theoretically this gap should be much smaller at the bottom of the tank than at the top to take the extra stresses, but in practice the spacing is left constant. This prevents mistakes during construction and does not add appreciably to the overall cost. The same spacing is often used on all of the tanks, both small and large.

The first mortar layer (1:3 cement:sand by volume)is trowelled onto the tank 1cm thick and given 24hrs to harden. A second layer of mortar is then trowelled on and finished smooth with a plasterer's float; this is also given 24hrs to harden.

The formwork is now carefully stripped and removed from inside the tank and a third layer of mortar is trowelled onto the inside of the tank to completely cover up the reinforcement. A thick un-reinforced coving is added to strengthen the joint between the floor and the tank.

Finally, the roof is built onto the tank y laying mortar onto shaped formwork which is propped from underneath. the roof is reinforced with two layers of wire mesh, which is tied onto the mesh protruding up from the walls (see Figure 4).

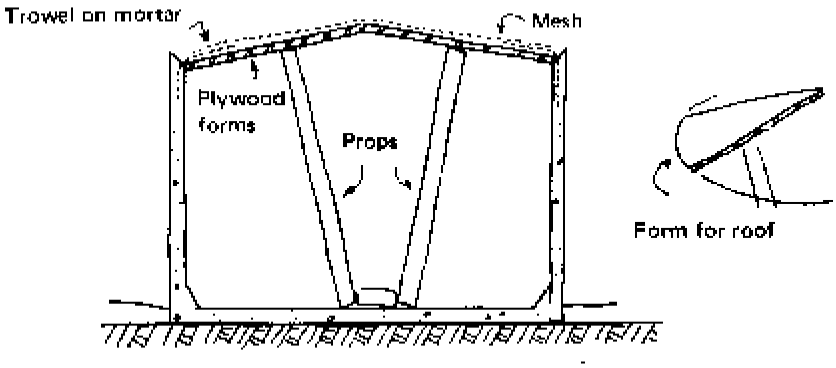


Figure 4 – Constructing the roof

A prefabricated angle iron frame is set into the wire mesh to provide formwork for an access hatch into the finished tank. This is removed after the mortar has set (see Figure XX5). Mortar is trowelled on in a 3cm layer and allowed to cue for 3 days. When it is strong enough the roof and access hatch formwork is stripped and a layer of mortar trowelled onto the inside of tank roof.
The tank is finally painted on the inside with a coat of cement and water slurry, a small volume of water is allowed to sit in the bottom of the tank and the tank is covered and cured for at least 7 days.

Transporting the tanks

The factory tanks of less than 25m^3 capacity are light enough to be carried by lorry. They are taken to the prepared site and joined directly to the necessary pipe connections; tanks of larger capacity are usually built on site.
The smaller, lighter tanks are lifted onto and off of the lorry with a truck-mounted hoist. The larger tanks are winched onto the truck with a sling. The first step is to jack one edge of the tank clear of the ground. The truck is then so positioned so that a pair of steel runners resting on its carrying platform can be placed under the tank to form a ramp. A wire rope sling is fitted around the tank, which is then drawn up the ramp by a winch mounted on the truck. Steel pipes are used as rollers when moving the tank.
For unloading the platform of the truck is raised slightly and the tank slides down the ramp. The steel pipes are again used as rollers and the downward movement is controlled by the winch.

Case study 9 – RWH in the barrios of Tegucigalpa

Introduction

This case study is drawn for a report produced by UNICEF in 1991. The 'Barrios' of Tegucigalpa, Honduras, are the low-income urban settlements that have developed around the city as tens of thousands of people move, each year, to the city from rural areas. They come in search of better living conditions but often end up in these barrios, where public services and amenities are poor or non-existent. Health statistics show that the residents of the barrios are suffering from a number of water related diseases that could easily be avoided with provision of a reliable, clean water supply. Unfortunately, more than 150,000 residents have to find their own water.

Water vendors sell water at extortionate prices, some families having to spend 30 or 40% of their income on water alone. In 1987, UNICEF, SANAA (National Water and Sewage Service) and UEBM (Unit for Marginal Barrios) started work on an integrated water supply project that would help the residents to direct their money into providing themselves with clean water. The programme studies several water supply options, including piped networks, groundwater wells, trucking of water and rainwater harvesting.

The report from which this Case Study is drawn studies the indigenous RWH systems in use in two barrios - Israel Norte and Villa Vueva. Although technically unsophisticated and lacking good health practice, the systems described here show what urban settlement have done to improve their own lot. Many of the systems make use of recycled or scavenged materials and some examples show high levels of initiative.

Water use

In the two barrios mentioned above, about 90% of the families collect rainwater. The quantity of rainwater collected varies from home to home. Figure 1 shows the percentage of needs met by rainwater in the barrio of Israel Norte.

Figure 2 shows the various uses of rainwater and the percentage of people who use the rainwater for a particular application.

Fig. 1 -Percentage of water needs met by Rainwater in the barrio of Israel Norte

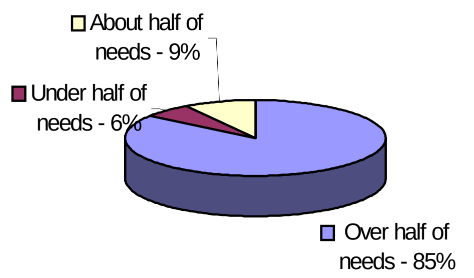
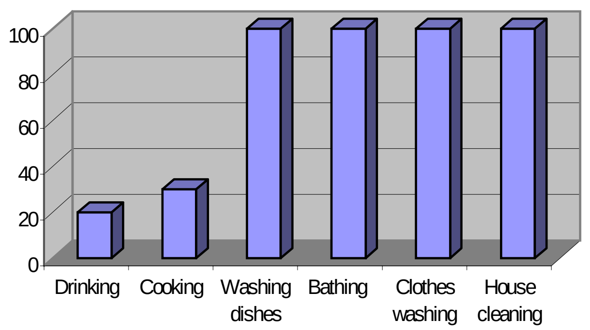


Fig. 2 - Rainwater uses in Israel Norte - %age of residents using rainwater for the following applications



The deficit in drinking and cooking water is usually met by water which is purchased form vendors or from nearby standpipes in middle class residential areas. The rainwater is not seen as being a high-quality source of water.

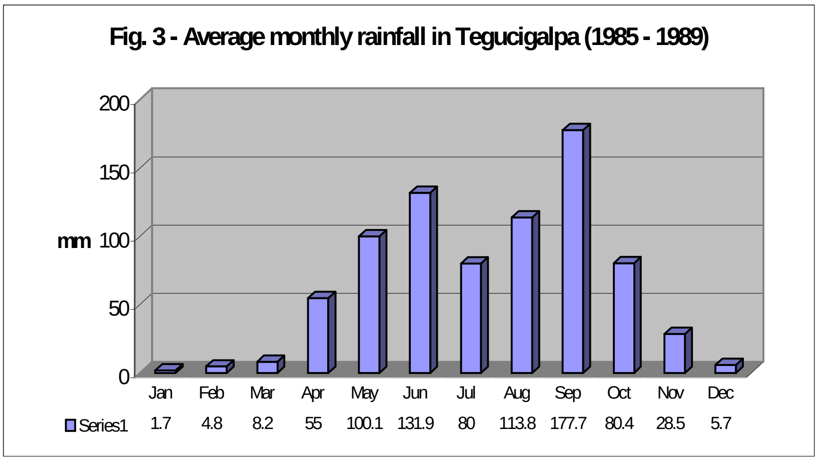
Rainwater harvesting is popular as there is a direct saving for every litre of water saved. For a household with a $45m^2$ collection surface (the average roof area) the saving is over US$100 annually.

Technical detail

The RWCS's used in the barrios are rudimentary. The basic system usually has the following components:

Roof – collection surface

The average roof area in the Villa Nueva barrio is $45m^2$, with typically half of this area being used for collecting water. The recommended roof area to provide adequate collection area for total rainwater harvesting is $100m^2$. The average rainfall in the area is 788mm, which is quite low and hence the large collection area requirement. The majority of the rainfall (as shown in Figure 3) falls between April and November with two peaks, one in June and one in September.



Roofing material varies, but by far the most commonly used material is iron sheet. Other materials used are asbestos/cement sheets, clay tiles, *techon* (a locally produced asphalt treated pressed paper sheet) and a variety of discarded plastics and sheet materials.

Gutters to collect the water from the roof

Again, a variety of materials have been used to make gutters. In the barrio of Villa Norte 75% of the gutters are made from sheet steel. The fabrication technique of steel gutters varies also – some have been made from scraps of steel sheet or old, flattened steel drums. Pre-fabricated gutters are also seen - these are rolled to give a semi-circular trough, and are fitted with a neck to attach the downpipe (where fitted), which can be of PVC. The authors state that the cost of these gutters was US$36 for a 20 foot length (1991). There are a number of different methods for fixing the gutters, but where high quality gutters are used the quality of the bracket is usually better also, being formed of wood or bent reinforcing bar. Some gutters were poorly mounted with depressions which allows water to stand and corrode the steel. Gutters are typically fitted to one side of the building only.

PVC gutters are formed from 8” PVC pipe which has been cut in half. The cost of a 20 foot length of PVC pipe is US$38 which provides 2 lengths of guttering when split. The PVC guttering is preferred because it is cheaper and lighter. Many other scavenged materials are used for guttering, including wood and asbestos sheeting.

Downpipe

In Israel Norte barrio, 90% of the systems have no downpipe. The water runs from the gutter directly into the storage vessel. The remainder used either plastic hose, PVC pipe or sheet metal to transport the water to a remote water storage container.

None of the systems studies were fitted with any kind of screen, filter or first flush mechanism.

Storage

Water storage facilities at the barrios are, again, basic. The majority are old 200 litre steel barrels. These are bought (the average price is US$13) or scavenged and most contained pesticides, chemicals or toxic materials – so are not well-suited to water storage. The second most common type of storage is the *pila, a* concrete water tanks of about 500 litre capacity which has an integrated washing board (see Figure 4). These are built by local masons and cost approximately of US$25. The tanks can be sized to suit the needs and means of the user. Fifteen to 30 % of the residents of the barrios have these pilas.

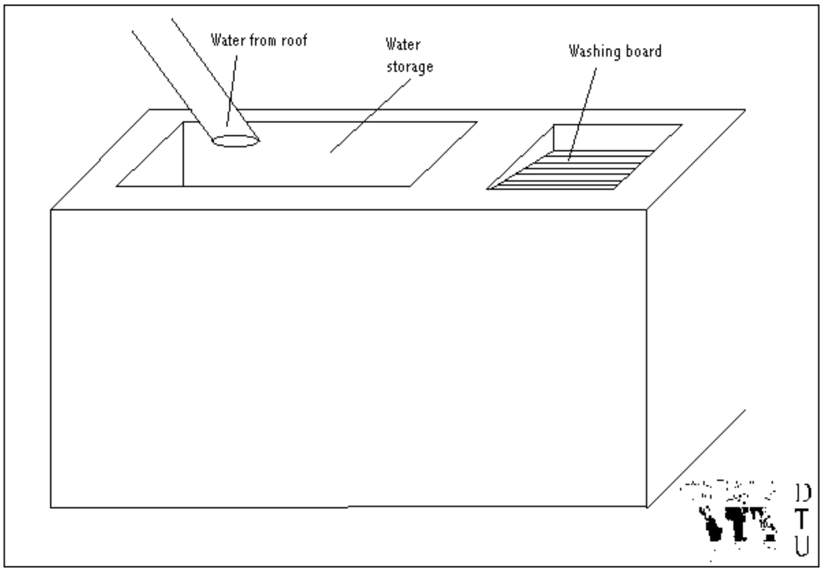


Figure 4 – The brick and mortar *pila*, as found in the barrios of Tegucigalpa

Some people have also acquired plastic barrels which may have contained paint, oil or other substance. Only very few of the systems studied had a cover fitted.

Water quality and alternative sources of water

The study team sampled the stored rainwater to find the level of bacteriological contamination present. It was found that where the water was used for drinking, 63% of the water samples taken contained E.Coli. Where the water was used for other domestic purposes only, 71% of the samples were contaminated. All sample were taken from the storage vessels.
The study team also sampled the alternative sources of water for the two barrios included in the study. Table 1 below shows the results.

Source	Number of Coliforms present (WHO guidelines recommend 0 coliforms in drinking water)
SANAA / UNICEF public taps	0 coliforms
Private water vedors – sample taken from hose	Uncountable
Unprotected superficial cells	varies between 650 and uncountable
Store reportedly selling water bought from SANNA truck	0 coliforms

Table 1 – Alternative water sources and their quality – Villa Nueva barrio

Research into single skin, externally reinforced, brick tanks

Interim Report
April 1999
Mr. D. G. Rees
DTU, University of Warwick

Table of Contents

1. Introduction

1.1 Project background

This work is being carried out as part of a programme titled 'Domestic Roofwater Harvesting in the Humid Tropics', which is an international 3 year, four partner, European Union funded, programme which started in August 1998.

Warwick is leading the sub-programme titled 'Low cost storage', the aim of which is to develop a number of techniques for construction of low cost water storage tanks or cisterns. As part of the programme we will look at several techniques for reducing costs, improving quality and improving health through good design and construction practice.

One such possible low cost design that will be investigated in detail is the externally reinforced, single skin brick tank. This Interim Report discusses the early research work that has been carried out on this design idea with the aim of clarifying the findings of this work and the further work that is required to provide enough useful detail to confidently promote such a technology.

1.2 Brick tanks – design philosophy for Developing Countries

The design philosophy adopted for water storage tank design is one of local manufacture using materials that are available with relative ease in the locality. Cost control is of major concern in order that the technology becomes more accessible to the poor of developing countries.

Brick is one such locally manufactured, widely used, readily available material which is ideally suited to wall construction, but not quite so well suited to conventional larger volume tank construction. In this report we look at methods of improving the suitability of brick to low cost tank manufacture by using external steel reinforcing to give additional hoop strength to cylindrical brick tanks. We also look at methods of lining such tanks for water tightness – at this stage limited mainly to internal cement render. Plastic lining of tanks will be discussed in a later report.

2. The theory of stresses in cylindrical tanks

Cylindrical tank walls experience a ‘hoop stress’ which is proportional to the diameter, D, of the tank, the pressure, p, on the walls of the tank and the thickness of the tank wall, t (Equation 1). The stress on a cylindrical tank wall is also affected by

$$\sigma_t = \frac{pD}{2t} \qquad \textit{Equation\_1 – Hoop Stress}$$

the type of joint between the tank base and tank wall. There are two obvious cases to consider as illustrated in Figure 1 below. Taking Case 1, if the tank wall is free to move at its base and still maintain a watertight seal, then the strain induced in the wall will cause the diameter of the tank to increase until the hoop stress is taken up by reinforcing in the wall (obviously the increase in diameter is exaggerated here for effect). In this case the force exerted by the water pressure on the tank walls will be taken up solely by the hoop tension in the walls. The maximum hoop stress will be experienced at the base of the wall and will decrease proportionally (linearly) according to Equation 1.

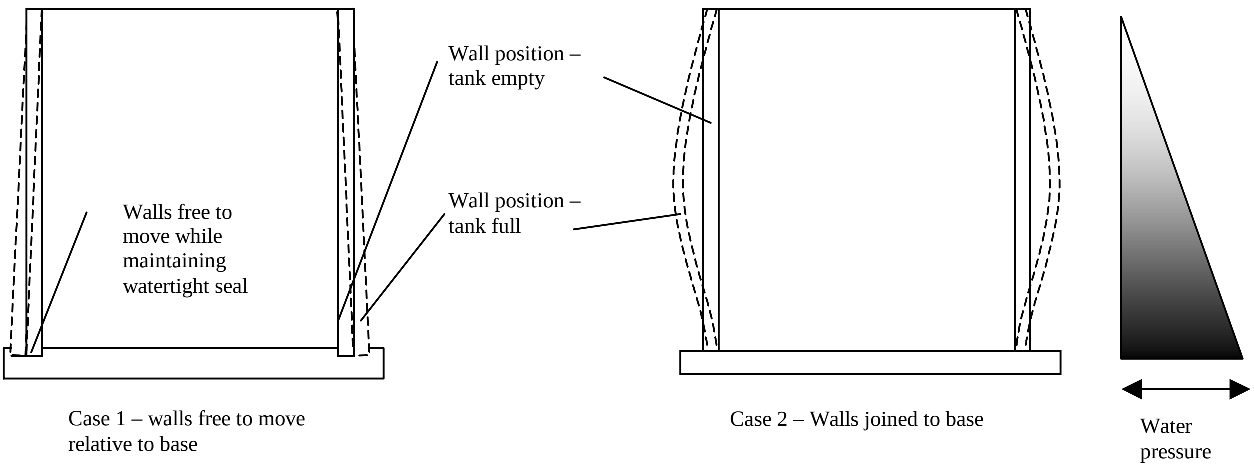


Figure 1 – The two cases for wall and base union in cylindrical tanks

Taking Case 2, if the wall and base are monolithic i.e. the wall and base are continuous, the situation becomes more complex as bending stresses are set up in the wall as a result of the restraining effect of the base slab. There now exists a complex combination of bending, shear and hoop stresses. Gray and Manning[4] suggest that if the wall is not free to move at its base, then the loading caused by the outward pressure is counteracted by a combination of hoop resistance and cantilever resistance. This is illustrated in Figure 2 which shows the load distribution diagram suggested by Gray and Manning. As the base of the wall is now restrained there is no freedom for the wall to move and take up the hoop stress and so the hoop stress there is reduced to zero. Maximum hoop stress is now experienced higher up the wall of the tank. All the restraining forces acting at the base are due to the cantilever.

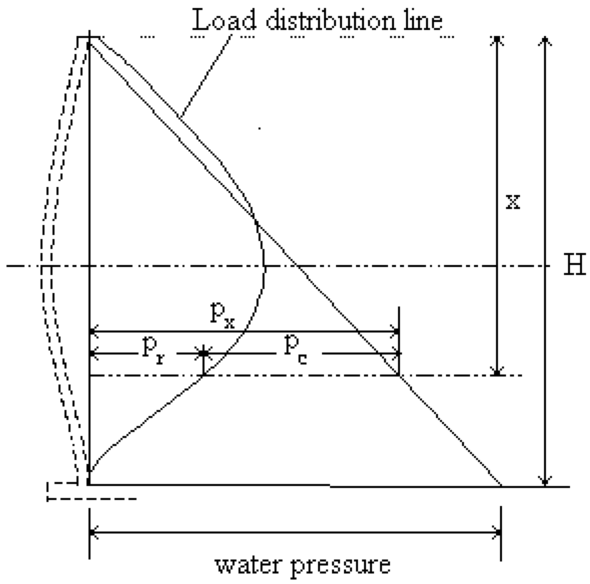


Figure 2 – Typical Load distribution diagram for a tank wall which is monolithic with base (modified slightly from Gray and Manning[4].

p_x – Total outward pressure load to be restrained
p_r – Portion of the load restrained by hoop stresses or radial constraints
p_c –Portion of the load restrained by cantilever
x – distance from top of tank
H – total depth of tank

As illustrated in Figure 2, the total pressure p_x at any depth of wall is composed of p_r (the portion of the load carried by the hoop restraints) and p_c (the portion of the load carried by the cantilever), such that,

$$p_x = p_r + p_c$$

The profile of the 'load distribution curve' is governed by the profile of the tank. Gray and Manning give a number of load distribution curves for a variety of tank profiles (see Figure 3). The tank profile is related to the distribution curve by Equation 2, all tanks with equivalent values of K having similar load distribution curves.

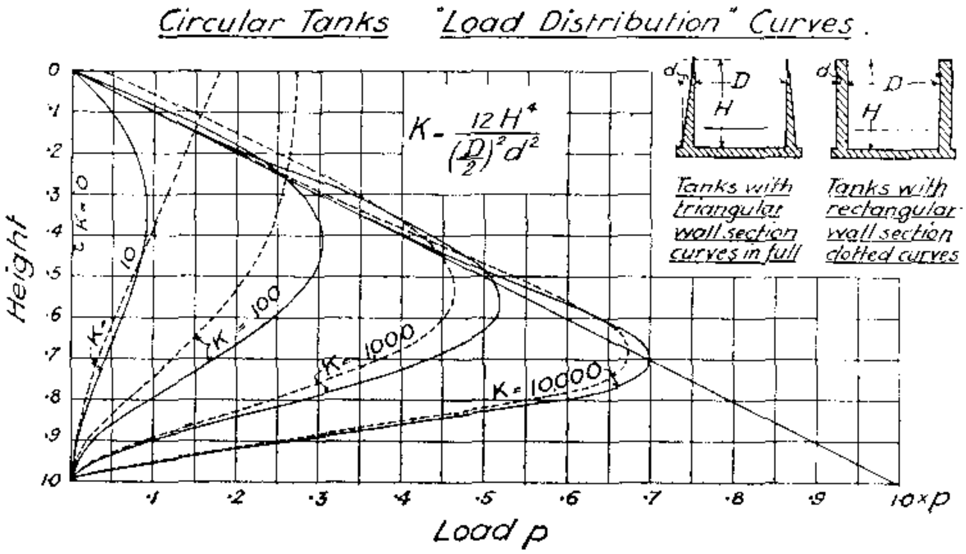


Figure 3 – Load distribution curves for a variety of K values (Gray and Manning[4])

$$K = \frac{12H^4}{\left(\frac{D}{2}\right)^2 d^2} \text{....................} Equation\_2$$

where,
H = height of tank
D = diameter of tank
d = thickness of wall at foot

It is noted that cantilever load increases for diminishing values of K. K is tightly controlled by the value of H such that K increases to the fourth power of H. We also see that K falls as tank radius and wall thickness increase.

If we consider a typical cylindrical single skin, brick tank with homogenous wall and base of diameter 2m, height 2m and wall thickness of 0.1m, then we obtain a value of K equal to 19,200. This is beyond the value shown in Figure 3 and suggests a regime where p_r dominates i.e. the outward pressure load is resisted predominantly by hoop stresses. Cantilever forces act only at the extreme base of the tank wall (within the bottom 20%). The tank can therefore be dealt with as if the hoop stresses induced are similar to those of a tank with a free base joint. This still leaves the high bending and shear stresses in the base of the wall to be evaluated. Watt[2] suggests that the maximum bending stress on the inside face of the tank wall will be almost double the induced hoop stress for a similar tank profile. This can easily be compensated by increasing wall thickness at the base of the tank. As demonstrated by Equations 3 doubling the wall thickness, d, effectively decreases the bending stress in the wall by a factor of four.

Bending stress, $$\sigma_b = \frac{M\left(\frac{d}{2}\right)}{\left(\frac{bd^3}{12}\right)} = \frac{6M}{b}\left(\frac{1}{d^2}\right) \text{ ....} \textit{Equation 3}$$

where, $\frac{bd^3}{12}$ is the Second Moment of area

and b is the width of a strip of wall carrying local bending moment, M

Shear stress is very small compared with the bending and hoop stresses (Watt) and can therefore be neglected for the sake of this analysis.

Work is currently being carried out to develop a computer software programme for a fuller analysis of stresses set up in a variety of tank shapes and sizes.

2.1 Calculating hoop stresses in reinforced brick tanks

The task of calculating hoop stresses in the tank under consideration now becomes a simple task. We treat the tank as if its walls were unconstrained at the base and can thus use Equation 1. We will then deal with bending stresses separately by increasing wall thickness locally at the base as required.

For a tank with a brick wall and external steel reinforcing we need to understand what stresses are induced in each material and what function each material is performing. Table 1 shows a hoop stress analysis for the two materials that have been used during tests at the University. These materials are high tensile steel packaging straps (of 13mm width x 0.5mm thickness), and brick masonry (in this case with cement mortar). If we analyse the two materials separately we can see from Table 1 that the ability of the brick masonry to withstand the imposed hoop stress is unpredictable due to the unpredictable nature of the quality of the material. High tensile steel, on the other hand, has reliable and predictable strength and will afford almost four times the required hoop strength. The steel, then, provides the tensile strength required to prevent failure due to excessive hoop stress.

The brick masonry, on the other hand, provides the stiffness required to prevent failure due to bending stresses. For the example shown below the stiffness of the brick is many orders of magnitude greater that the 'virtual' steel wall stiffness (virtual wall thickness is the thickness that would exist if the straps formed a continuous steel shell - the straps themselves actually provide no vertical stiffness).

We therefore see that one material compliments the other. The steel provides a hoop strength up to eight times that of the brick masonry whilst the brick masonry provides the stiffness that could only be achieved in a steel tank with hundreds of times the quantity of steel used in the example given.

Properties	**High tensile steel strap (13mm x 0.5mm) – two straps per course of bricks**	**Brick masonry**	**Units**
UTS (Mpa)	833	0.1 – 1.0	Mpa
Yield strength	450		Mpa
Youngs modulus	210	25 - 100	Gpa
Wall thickness (equivilant)	173×10^{-6}	0.1	m
Tank diameter	2	2	m
Water depth	2	2	m
Hoop stress exerted by water force	113.19	0.196	MPa
Safety factor	3.98	0.51 – 5.10	

Table 1 – Hoop stress analysis for steel and brick masonry in cylindrical tank of 2m diameter and 2m height

2.2 Construction and application regimes to ensure strength

Some thought has to be given to the interaction between the brick masonry and the steel strap. The two materials must interact in such a way as to enhance the properties of the other as explained in the previous section. This can only be achieved when the construction of the walls and application of the steel strapping is performed in a certain way. The steel is applied externally to the brick wall, using a tool specially designed for applying this kind of strap. The strap should be applied with sufficient pre-tension to be able to counteract the normal water force that will be exerted upon the wall upon filling the tank. In consequence, the masonry is initially put into compression and remains in compression as the tank is filled. This will protect the brick masonry against failure (cracking) should the hoop strength of the masonry be unable to accept the load (due to inferior workmanship or poor material quality). Referring to Table 1, the hoop stress exerted by the water force is 113.19 MPa (say 114 MPa). Experiments were carried out at the University to confirm that such a pre-tension can be applied to the tank and the description and results of these experiments are shown in a subsequent chapter.

2.3 Stresses in tank foundations and base

Little work has yet been done to calculate stresses in the tank base. This will be included in the next report.

2.4 Stresses due to other factors

Stresses due to other factors such as wind forces, cyclic loading and seismic activity have not been considered in this analysis.

3. The theory of cracking in cement render

(This section is the work of Dr. T. H. Thomas from a paper titled 'The Causes and Prevention of Leakage through Cementitious Renders in Water Tanks')

3.1 Cementitious renders

Renders of cement or lime mortar are commonly used in water tanks when the tank itself is constructed of rather permeable materials such as brick, stabilised soil or even (in the case of underground tanks) of unstabilised soil. The render's primary purpose is water-proofing, for which it should have a sufficiently low permeability to protect the main tank material and to reduce water loss through walls to a tolerable level. Thus we might demand that the render keep wall leakage in a 10 m^3 tank to under 1 litre per day. The render may have secondary functions such as reducing the roughness of a masonry surface so that it can be easily cleaned down, or even of providing a little stability to the wall behind it.

Cementitious renders are usually sufficiently impermeable in themselves, but are so brittle and so intolerant of *tensile* strains that they commonly crack. It is the cracks that leak and preventing cracks should be a focus of research effort.

3.2 Sharing out any shrinkage between 'many' cracks

Shrinkage of a render constrained by underlying masonry is the main mechanism for producing cracks. Tensile stresses in the render are relieved by cracking. Tensile strains are replaced by a combination of strain-free mortar interspaced by cracks carrying no tension forces. For a given shrinkage and a given material, the total volume or total width of cracks may be more or less fixed. However that total may be variously distributed between few or many distinct cracks. Consider for example the inside of a masonry tank of diameter 2 m where a render incapable of supporting any tensile strain has contracted by 1000 microstrain (0.1%) relative to the masonry. Around a circumference we might expect a total crack width of about 6 mm divided between say n individual cracks. We would certainly expect the leakage through an individual crack to be a rising function of its individual width. We might expect the total leakage to fall with rise in n - e.g. for two 1 mm cracks to leak less than one 2 mm crack.

Consider an individual crack of length L and width W penetrating a render of thickness T across which there is a pressure drop, from liquid to liquid, of p. The leakage velocities are very low and the key dimension (W) is small, so the Reynolds Number will certainly indicate laminar flow. As water behaves as a fairly Newtonian fluid, we can assume a viscous shear stress in the fluid proportional to the transverse velocity gradient in that fluid. Provided velocities are small compared with that corresponding to conversion of pressure head into velocity head, (i.e. $v^2 << 2p/\rho$) this gives a velocity profile of :

$v = \frac{p}{2T\mu}\left(Wy - y^2\right)$ where v is water velocity in the layer distance y from the side of the crack.

The consequent flowrate through the whole (assumed rectangular) crack is:

$Q = L\int_0^W v\,dy = \frac{pL}{12T\mu}W^3$ which is proportional to W^3, hence Q/W is proportional to W^2.

This suggests that replacing one large crack by two smaller (half-width) cracks will usefully reduce leakage by a factor of 4.

Inserting numbers: a 100 mm long crack of width 0.1 mm in a render 10 mm thick and subject to a water pressure of 10 kPa (1 m head) will leak 20^{o}C water (μ = 1 mPa s) at nearly 750 litres per day. Replacing this crack by ten 0.01 mm cracks will reduce leakage to 7.5 litres per day. Even this is usually unacceptable, so we are looking to get crack widths below 10 μm. However at these capillary sizes, surface tension effects become dominant.

Surface tension is likely to prevent *any* flow through a plane crack whose thickness W is less than $2\gamma/p$ where γ is the surface tension of water (in Nm^{-1}). Thus for water at 20^{o}C for which γ = 72.5 mN/m, a head of 1 m will not cause leakage through a gap smaller than 14.5 μm.

3.3 Prevention of cracking or of leakage through cracks.

Render cracks may form during ‘manufacture’ of a tank or when water pressure is first applied after manufacture or during moisture cycling through the life of a tank. We are studying the various shrinkage and crack-formation mechanisms associated with cementitious materials. Generally such materials experience shrinkage of up to 2000 μstrain.

Strategies to prevent or reduce leakage through cracks include:

	Approach	Technique
a	Use non-shrinking render	1. Use mortar with very low water content 2. Pre-shrink the render 3. Cure under water
b	Remove the constraint provided by underlying masonry/reinforcement	1. Add reinforcement after curing 2. Use flexible mortar to lay bricks/blocks 3. Use very-slow setting mortar to lay them 4. Masonry same as render
c	Reduce constraint of base plate	1. Flexible or sliding wall-base joint 2. Thicken wall to reduce bending stresses
d	Distribute cracks (convert into more but smaller cracks)	1. Reinforce with mesh 2. Reinforce with fibre 3. Manipulate bond with masonry
e	Stagger cracks in multi-layer render	1. Plaster, cure, replaster 2. Plaster, cure, groove, fill
f	Use a flexible render	1. More (hydraulic) lime in mix 2. Add polymers like latex to mix
g	Prevent cracks opening under stress (strain induced by water pressure)	1. Hoop tension the reinforcement after curing the render 2. Do so before curing the render. 3. Thicken the masonry to reduce changes in strain when under load.

Table 2 – strategies for preventing cracking in cementitious renders

4. Building materials

One of the aims of this research is to investigate the behaviour and suitability of the materials listed in this chapter for use in construction of a water storage tank and to make recommendations for tank construction techniques using these materials. Tests and test results are shown later in this document. Now we will discuss the general nature of the materials and the general theory postulated for design.

Fired clay brick is a rigid, brittle building material. It is usually used in conjunction with a cementitious binder (such as cement or lime mortar) to form a wall which is strong in compression but weak in tension. There are a number of techniques used to give extra strength where required: this usually takes the form of laying the bricks in a regular pattern to form an interlocking matrix of brick and mortar. Ties, buttresses, and other building aids and techniques give extra, localised strength where required. It is difficult to improve the tensile strength of brick masonry without creating a composite material by adding steel or timber.

The predominant stresses that will be set up in a tank wall are tensile stresses. As explained in an earlier section, these stresses are due to the water in the tank exerting hoop stresses and bending stresses in the structure. The brick and mortar construction alone cannot be expected to withstand these stresses and it is for this reason that steel strapping is used to take up the predominant tensile stresses. The brick gives rigidity (i.e. stiffness) and mass to the structure while the steel, with it's high tensile strength, performs the task of providing hoop strength. Together, using the cylindrical wall structure, they combine properties to provide a rigid structure with sufficiently high tensile strength.

A further requirement of the tank wall material is that it be watertight, and that it remain watertight for the lifetime of the tank. Brick masonry rarely achieves this requirement and so we will also look at methods of providing a waterproof membrane or lining within the tank. Therefore, we also look at cementitious renders which are applied to the inner surface of the tank wall to provide a continuous waterproof membrane. Later (in a later report) we look at low-cost plastic liners for water tanks.

We investigate the properties of these materials used together (brick masonry – steel – mortar render) to determine the optimum design specification that will give adequate strength, rigidity and watertightness, while minimising the quantity of material used (with the aim of keeping costs to a minimum). Each of the individual materials mentioned above has it's own unique behaviour and the biggest challenge comes in trying to match these materials in such a way that they act in synergistic fashion to provide a composite that gives the required properties.

Much thought has been given to the process required to achieve this synergy. A number of mortar and render types have been investigated and the methods of application and construction regimes have been considered carefully. In this report we will look in some detail at the following aspects of tank design and construction:

- behaviour of lime and cement mortars
- application of steel strapping to cylindrical steel tanks
- stresses in steel strapping in cylindrical brick tanks during application and under load conditions
- initial investigations into shrinkage and cracking in thin renders

4.1 Tank construction materials

Fired clay brick.

Low quality fired clay brick is a building material which is commonly found in many, many countries throughout the world. Brick masonry is a building technique that has existed for more than 4000 years. A variety of soils are available for the process of brick making and the process itself varies in complexity from small field based batch kilns capable of producing a few thousand bricks to large scale continuous mechanised technology capable of producing hundreds of thousands of bricks per day.

Cement mortar.
Nowadays, the most common methods of bonding fired clay bricks is with cement mortar. Cement mortar is a hydraulic binder made from a controlled mixture of sand and Ordinary Portland Cement (OPC), with or without a variety of admixtures that improve the properties of the mortar. Plasticisers are used to reduce the water requirement of mortar and hence improve strength, or to improve the workability for a given water content. Bagged lime is sometimes added to improve workability. A typical mortar would be made up of 1 part OPC to 4 parts clean, well-graded sand, although the quantity can vary enormously depending on strength requirements.

Lime mortar.
Lime has been used as a building material for over 2000 years. To make a lime binder or mortar, calcium carbonate (limestone) is burnt at a temperature of about 900^oC to drive off the carbon dioxide and produce calcium oxide (quicklime). The quicklime is then ‘slaked’ (water is added) to produce hydrated lime powder. If further water is added a lime milk is formed and this is allowed to settle and mature for some time (months) in a pit to form a lime putty. This can then be used for a variety of applications including mixing with sand to form a lime mortar.

Lime mortars have certain benefits over cement mortars. The lime mortar slowly absorbs carbon dioxide from the atmosphere and this causes the mortar to harden as it returns to it’s initial state as limestone. This process can take many years and in the meantime the mortar remains plastic. This can be beneficial where flexibility is required or where other less flexible materials are used in close proximity.

5. Reinforcing materials

5.1 Choice of reinforcing material

A number of materials were considered for use as external reinforcing for the tank. There are examples of similar designs from Thailand of brick masonry and steel wire tanks (Vadhanavikkit[6]), and from Uganda of stabilised soil cement blocks with barbed wire reinforcing (personal correspondence with Mr. Kimanzi Gilbert of the Uganda Rainwater Association) and other examples of brick and barbed wire tanks from Botswana.

As mentioned in the introductory chapter it is important that the materials used for the tank construction be available widely in the areas where they will be built (less developed tropical regions) and that the materials be of low cost. Those materials mentioned above i.e. steel wire and barbed wire, as well a number of other similar materials are readily found for other construction and agricultural applications.

In the cases mentioned above there is some concern (on our part) as to the control of the application of the reinforcing material and the amount of tension that can be achieved and maintained. Tying knots in wire, and maintaining tension at the same time, is a difficult business, especially when the wire needs to be tight against the surface of a wall. We are therefore considering packaging strap as an alternative for the following reasons:

- it is widely available in any country where there is a reasonable manufacturing base;
- the strap is ideally suited to this application with a high tensile strength and flat surface that lays flat against the brick face;
- it is cheap once the tools have been purchased (even the tools themselves are not prohibitively expensive for a mason of reasonable standing). More research is currently underway into prices of strap and tools in developing countries;
- application is easy with the dispenser, tensioning and crimping tool
- a high pre-tension can be easily applied to the strap applying compression forces to the masonry.

Depending on feedback from a number of countries (Sri Lanka, Uganda, India, Kenya in particular), we will later reassess the suitability of this material and possibly look at alternatives.

5.2 Spacing of steel straps

Following on from Chapter 2.1 we are now able to calculate the spacing for the steel strapping, assuming pure hoop stresses are induced by the outward water pressure.

Virtual wall thickness, $t = \frac{w \times t_s}{s}$ .... *Equation 5*

where, w = strap width
t_s = strap thickness
s = maximum strap spacing

Maximum strap spacing, $s = \frac{2\sigma_{ma} w t_s}{\rho g H d}$ .... *Equation 6*

For the tank considered in Table 1, we can rearrange Equations 1 and 5 to give Equation 6. If we specify a value for maximum acceptable hoop stress in the strap, $\sigma_{ma,}$ of say 150 Mpa (one third of the yield strength of the steel), we can easily calculate the strap spacing using a simple spreadsheet calculation. Table 3 below is the data output from such a spreadsheet calculation for a small variety of tanks of a similar size and profile to that being considered.

	Distance from top water level						
Tank diameter (m)	<0.5m	0.5m - 0.75m	0.75m - 1.0m	1.0m - 1.25m	1.25m - 1.5m	1.5m - 1.75m	1.75m - 2.0m
1.00	398	265	199	159			
1.25	318	212	159	127			
1.50	265	177	133	106	88		
1.75	227	151	114	91	76	65	
2.00	**199**	**133**	**99**	**80**	**66**	**57**	**50**
2.25			88	71	59	50	44
2.50				64	53	45	40

Table 3 – Maximum strap spacing for a variety of tank diameters and depths

It can be seen that the minimum strap spacing (at the base of the wall) is less than the thickness of one course of bricks (80mm) and so two straps will be used on each course of bricks, giving a spacing of approximately 40mm.

6. Laboratory experiments

6.1 Introduction

The experiments described in this chapter were carried out at the University of Warwick between January and April 1999. Forming the basis for the tests were a number of brick masonry cylinder specimens built on a civil engineering 'strong floor'. Such a specimen is shown in Figure 4 below. The specimens were of internal diameter 1.5m and varying height. These specimens allowed experimentation and observation of mortar behaviour, cracking of renders with a number of different additives, application and loading of steel packaging strap, as well as providing through practice an insight into the merits of the construction technique.

Figure 4 – Brick masonry cylinder specimen

6.2 Mortar observation – lime and cement mortars

Two types of mortar were used for constructing the cement mortar specimens; cement mortar and lime mortar. The main purpose of this test was to observe the following:

- the plasticity of the mortar and the associated ability of the brick masonry to ‘move’ in compression;
- the behaviour of the brick masonry under load – particularly the load transfer characteristics of the masonry;
- render behaviour (particularly cracking) when applied to each type of brick masonry.

The cement mortar is composed of the following:

Sand cement ratio	5:1
Water cement ratio	0.5 (i.e. water content 50% by weight of cement)
Mortar plasticiser	10% of water content

The lime mortar is made up of 3 parts well-graded sand to one part lime putty. No water is required.

The first part of the experiment consisted of simply observing the brick masonry when applying the steel strap for signs of movement and, secondly, observing the tension in the strap to determine the ability of the masonry to go into compression without distortion (i.e. that the masonry is sufficiently rigid). Early in the experiment it was decided that the lime masonry was insufficiently rigid in the early days after construction and that, without further investigation of its properties it would be unwise to use this material for this application. Although the characteristic property of lime mortar to remain plastic can, in some cases, be a positive advantage (we subsequently see that render cracking is greatly reduced - almost eliminated even - when using lime mortar), it does mean that a wall built using lime mortar will be far less rigid than its cement mortar equivalent.

Cement mortar gives a strong, rigid wall with no sign of movement during application of the strapping. The load that can be applied to a cement mortar, brick masonry tank is outlined in Graphs 1 and 2. The figures shown in this graph demonstrate the rigidity of the material at characteristic loads (and beyond). Detailed tests of this nature have not been carried out to date on the lime mortar specimen, but visually the wall of the lime mortar specimen can be seen to move when strapping is applied.

The behaviour of renders on each of the specimens is detailed in the following chapter.

6.3 Render – shrinkage and cracking

A number of tests were carried out to observe cracking of renders of different types on specimens constructed with two different mortar types (cement and lime mortar). The base render was made up of 4 parts sand to one part OPC and a water content of 0.4 (this was slightly exceeded in most cases to provide a workable render) Mortar plasticiser was added in all cases at 10% of water content. The render thickness varied due to the uneven surface onto which it was being applied, but in general the thickness remained within 5 – 15mm. After a day the wall was painted with a thin white paint to help with the location of cracks (see Figure 5 below). Any visible crack was measured using a hand held x20 microscope. Shrinkage was measured by observing the separation of the render from the wall at the top of the specimen. In

Table 5, separation is indicated as a percentage of the visible joint line over which separation occurred and the minimum and maximum value of separation width (in mm).

Figure 5 – Cracks in renders

The additives used during the tests are shown below:

- **Mortar plasticiser** – a proprietary plasticiser used for mortars and renders in the building industry.
- **Re-in fibre** – a polypropylene fibre of 50 micron square cross-section and 6mm in length. This is a UK construction industry building material used for preventing cracking in thin renders and screeds.
- **Febond SBR** – a proprietary waterproofing solution for use in renders and for other application. It is a styrene-butadiene co-polymer latex specifically designed to improve water resistance and durability.

Five render types were tested. All were cement based renders. These are listed below in Table 4.

Render type number	Sand : cement ratio (by weight)	Water cement ratio (by weight)	Mortar plasticiser content (as %age of water)	Other additive	Curing regime
01	4:1	0.4 – 0.5	10%	none	basic*
02	4:1	0.4 – 0.5	10%	none	7 days**
03	4:1	0.4 – 0.5	10%	re-in fibre	basic*
04	4:1	0.4 – 0.5	10%	re-in fibre	7 days**
05	4:1	0.4 – 0.5	10%	sbr	7 days**

Table 4 – composition of renders used for cracking and shrinkage tests
*** basic infers no special curing regime employed – render left to cure in open air**
**** 7 days curing under soaked Hessian cloth**

Cement mortar used to lay bricks					
Render type (see Table XX)	Ave. crack length (mm)	Crack length per (mm /m^2)	Ave. crack width (mm)	Maximum crack width (mm)	Separation *
01	70.37	1135	0.13	0.35	90% 0.05 – 0.75mm
02	54.5	2546	0.29	0.55	10% 0.1 – 0.75mm
03	31.0	53	0.1	0.1	90% 0.05 – 1.5mm
04	No test				
05**	No test				
Lime mortar used to lay bricks					
Render type (see Table XX)	Ave. crack length (mm)	Crack length per (mm /m^2)	Ave. crack width (mm)	Maximum crack width (mm)	Separation *
01	No test				
02	0	0	0	0	no separation
03	No test				
04	0	0	0	0	no separation
05**	69.33	2207	0.32	1.3	no separation

Table 5 – Cracking in cement renders – values for a number of test results after 7 days
*** Separation given as a percentage of separation at the visible joint around the perimeter at the top of the specimen wall. Variation in crack width also given.**
**** although manufacturers instructions were followed carefully there is some concern about the validity of these results – possibly an incorrect quantity of SBR was added to the render.**

The results given in Table 5 lead us to a number of tentative conclusions:

- the re-in fibre (renders 03 and 04) significantly reduces cracking in renders;
- improved curing of renders on cement mortar wall (renders 02, 04 and 05) helps prevent separation at the cost of increasing cracking i.e. adhesion to the wall improves causing greater cracking – this is advantageous if we then seal the cracks with a nil coat (cement slurry) or other proprietary sealant ;
- separation of render is much greater on cement mortar walls due to the rigidity of the wall. In the case of lime mortar, the wall moves as the render shrinks, preventing cracking – as mentioned in an earlier chapter this is very beneficial for achieving crack-free renders but not so beneficial in terms of loss of rigidity.
- no conclusions are made about the characteristics of SBR render due to uncertain results

Further work is required to gain a better understanding of renders and their behaviour on internal tank walls. This work is outlined in the final chapter of this report.

6.4 Reinforcing straps – initial tensile strength tests

Initial tensile strength tests were carried out on a number of packaging straps to determine the strength of each (manufacturers specification was not available for all). The aim of the tests were to investigate the strength of woven polypropylene strap and the effect of crimping steel straps. The tests were carried out at the Universities civil engineering laboratory using the tensometer machine (see Figure 6).

Without going into detail here it can be stated that the *polypropylene* strap was insufficiently strong for the application under consideration with an Ultimate Tensile Strength half that of steel with ten times the strain.

In all cases the singly crimped steel band broke at the crimp at well below maximum UTS. During practical tests on the brick specimens, in which the strap had been fitted with two crimps, the strap broke remote from the crimp at a value close to UTS.

6.5 Reinforcing straps – applying the strap

This experiment aimed to investigate:

a) the pre-tension set up in the reinforcing straps during application with the tensioning tool - as mentioned in chapter 2 it is necessary that sufficient pre-tension exists in the strap after application to support the masonry in compression and prevent cracking when the tank is initially (and subsequently) loaded.
b) the distribution of the tension in the strap upon application – this experiment was to test the assumption that circumferential tension in the strap would vary due to the friction between the wall and the strap, from a maximum near the tensioner, to a minimum on the opposite side of the tank.

The experiment involved measurement of the strain in the reinforcing strap using strain gauges. The strain gauges had been calibrated beforehand (see Figure 6 below) so that the load in the strap could be derived directly and accurately from the strain values. Three strain gauges were placed on a single strap, at 0, 90 and 180 degrees around the specimen circumference, and readings taken as the strap was applied (see Figure 7).

Figure 6 – Calibration of strain gauges using tensometer

Figure 7 – Showing strain gauge fitted to the steel strap

Box 1

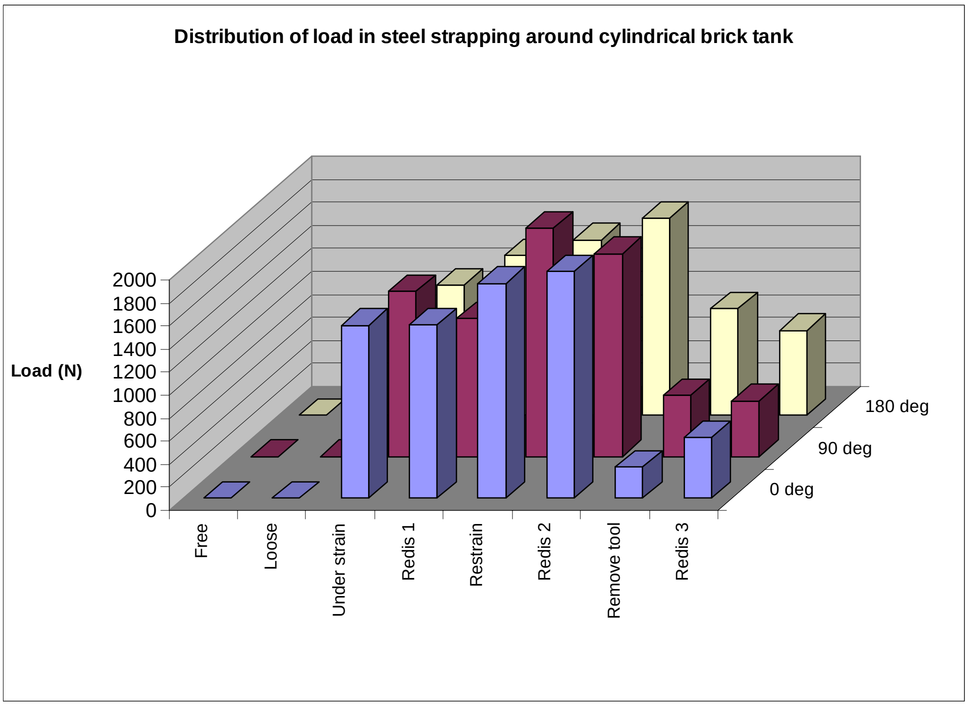
Applying the steel strap
The strap is applied to the brick masonry specimen using a manually operated tensioning tool. Once fully tensioned the strap is crimped (see Figure 9 below) using specially designed crimps and crimping tool and then the tensioning tool is removed. It can be seen from Figure 8 below that the tensioning tool holds the strap away from the wall in order to allow access for the jaws of the crimping tool. After initial experiments there was some concern about the loss of tension when the tool is removed.

Graph 1 shows the steps taken during the experiment and the load registered by the strain gauges. The term 'redis' in the graph means redistribution of the load. This was achieved by using a screwdriver to prise the strap away from the masonry slightly (moving around one brick at a time) to allow the tension in the strap to be redistributed.

Box 2

Strap pretension required to balance water pressure forces and prevent tensile stress being set up in masonry
- From Table 1, maximum hoop stress in steel due to water pressure is 114 Mpa
- Area of cross section of steel strap = 6.5mm^2
- Tension required in strap = 6.5 x 114 = 741 N

An analysis of the graph clearly shows the increase of tension in the strap as the strap is tightened and also shows that a certain amount of redistribution of that load takes place during the 'redis' phase. The drop off in tension during removal of the tool without packing (as illustrated in Figure 8) is completely unacceptable with the final tension being a small fraction of the tension created by the tool and insufficient to put the masonry into compression (see Box2). We see, however, that the tool is capable of tensioning the strap sufficiently – values of almost 2000 N being achieved during the tensioning phase.



Graph 1 – Distribution of load in steel strap around cylindrical brick tank during application

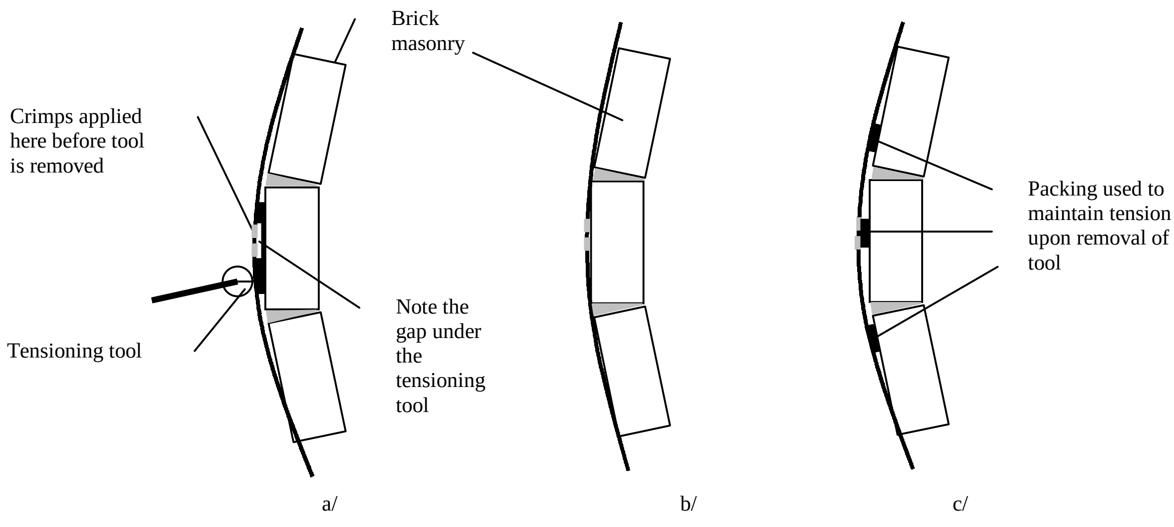
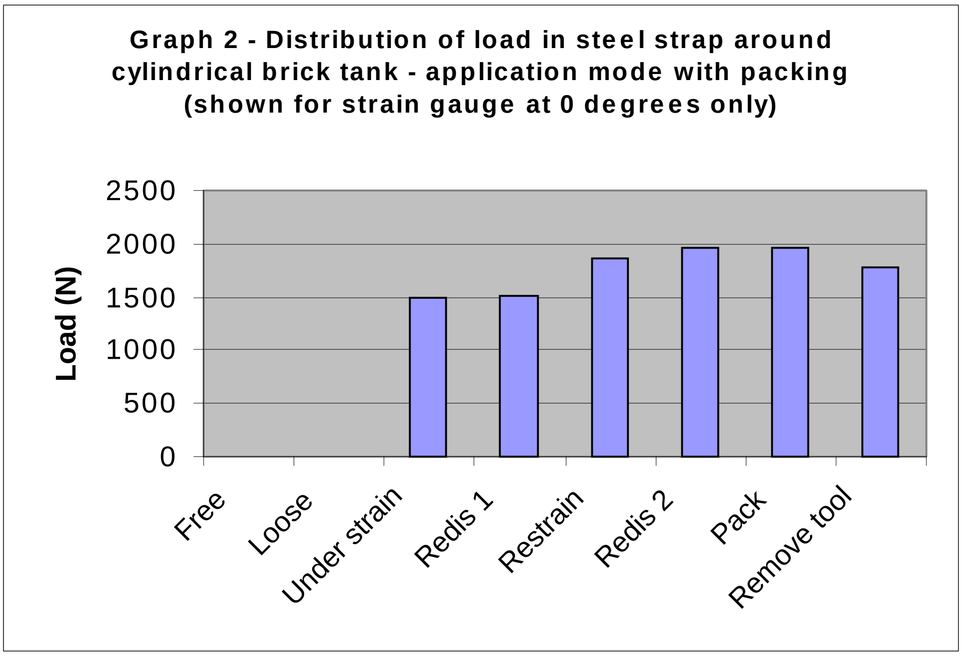


Figure 8 – showing tensioning and crimping arrangement for steel strapping a/ during the tensioning and crimping process b/ when crimping is complete and the tensioning tool has been removed and tension reduced c/ maintaining tension by using packing

Figure 9 – photograph of straps showing crimps

Figure 8 shows the method used to pack the strap to prevent loss of tension upon removing the tensioning tool. If this process is carried out the tension can be maintained as shown in Graph 2. In this case the tension, at 1777 Newtons, is more than double that required to prevent the masonry going into tension upon pressure loading. Practically, this involves placing some packing material behind each strap as it is applied. Further work is needed to define a practical method and a suitable material for this purpose.



Graph 2 - Distribution of load in steel strap around cylindrical brick tank - application mode with packing (shown for strain gauge at 0 degrees only)

6.6 Reinforcing straps – loading and ultimate tensile strength

A test was carried out to determine the maximum load that can be applied to an externally reinforced brick tank. The aim of the test was to simulate water pressure acting on the wall of the tank and to observe the behaviour and failure mode of the tank as pressure was increased. Ideally the test would have been carried out using a sealed tank that could be pressurised to the point of failure but this was not practical at the University, especially indoors on the civil engineering strong floor. To simulate water pressure a steel expansion ring was manufactured as shown in Figure 10 below. The ring was made from 3mm mild steel to provide enough rigidity to prevent buckling but enough flexibility to take up the shape of the interior surface with which it came into contact. The ring was expanded using two 1 tonne hydraulic jacks, as shown in Figure 11. A load cell was placed in series with the jack to measure the applied load and is also shown in Figure 11 (this had been calibrated earlier). The ring spanned 2 courses of bricks that with hindsight, should have been free floating, but were not. The straps on the specimen were fitted with strain gauges at 0^o, 90^o, and 180^o to measure the stress induced in the strap. Thus we could monitor the increase in load in the strap due to increased (simulated) water pressure.

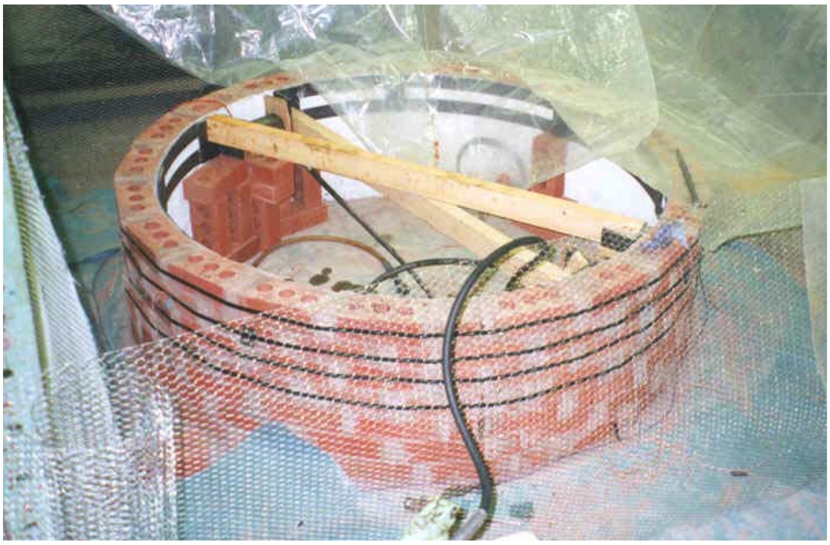


Figure 10 – Specimen under test, showing expansion ring

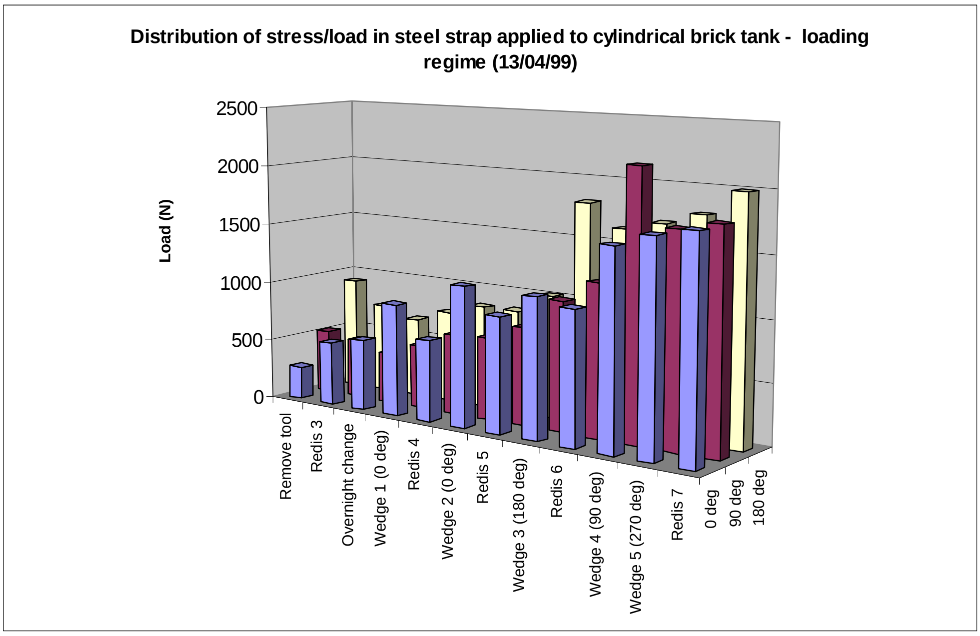
As the hydraulic jacks were extended the ring slowly expanded putting the strap into tension. The jacks continued to be extended until the straps failed and the masonry broke. During this time observations were made of the following:

- the applied load
- the cracking of the render
- the cracking of the masonry
- the strain (and hence load) in the strap

Graph 3 (histogram) shows the general trend of increased load in the straps as the expansion ring is opened. Noting the horizontal axis of the histogram we see that jack pressure in the expansion ring is not rising continuously. Graph 4 shows a clearer representation of what is happening in one of the straps – namely the strap at 90^o. There are some interesting phenomenon to observe:

1. Firstly, the force in the expansion ring is much greater that the force in the strap. There are a number of possible explanations for this:
    - the two courses of brick under test are not free-floating and so some energy is being required to stress the lower courses of bricks
    - in the early stages of the test, until the masonry fails, some of the load is taken by the masonry itself
    - there are four straps fitted on the specimen, all of which are accepting some of the load – two strap are directly under load

2. We see two distinct regions (see Graph 4 below), one where the pressure in the ring rises linearly with the load in the strap (region 1) and then a region where the pressure in the ring rises little, and indeed starts to fall, as the load in the strap increase and the ring continues to expand (region 2). This can be explained as follows (either or both of the following acting at any time):
    - as the cracking in the specimen worsens, the energy that was taken up 'bending' the lower part of the wall is now redistributed in the upper part of the wall (the test area) as the joint between the two fails.
    - the masonry breaks locally, and there is a significant repositioning of the brick (local to the ring) within the masonry as the ring expands further.

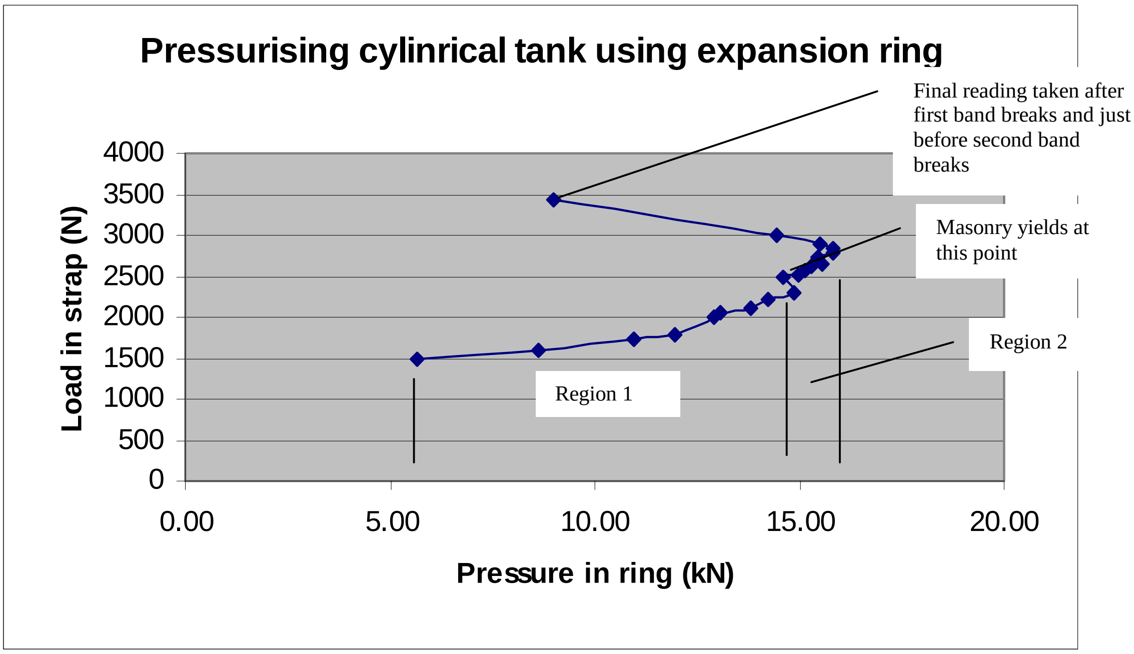


Graph 3 – Loading cylindrical tank using expansion ring

3. Large vertical and horizontal cracks appear (see Figure 12):
    - the horizontal cracks are due to shear as the upper section of the specimen wall shears away from the lower section
    - the vertical cracks are obviously the result of tension as the pressure is applied to the ring

Figure 11 – Specimen under test, showing close up of hydraulic jack and load cell

Figure 12 – Cracking in specimen after pressure test



Graph 4 – Pressurising cylindrical tank using expansion ring – strain gauge readings at 90 degrees

In conclusion it should be strongly noted that although what we see is a realistic representation of what was going on in the test specimen during the test, and is of some value to us in understanding the behaviour of cylindrical tanks, it is not what we were looking for! This was due to improper experimental design. A further test will be carried out on a free-floating specimen i.e. a specimen set on a sliding ring to simulate an elemental horizontal slice of tank. This will hopefully yield results that will be of more interest to us.

7. Conclusions

The general conclusion is that the single skin, externally reinforced, brick tank is a viable alternative to more costly tank types available at present. The experiments carried out so far have shown that the construction technique adopted is sound, but further work is still required to achieve a full analysis of the ideas under discussion which will lead to a final design and set of construction guidelines for this type of tank. The work that still needs to be carried out is outlined in the following Chapter

8. Further work to be undertaken

The following have been identifies as areas needing further work:

From section number:	Description of work
1.2	Research into plastic linings for tanks
2.1	Carry out full computer analysis of stresses in cylindrical tanks
2.3	Analysis of stresses in foundations of tanks Foundation design
2.4	Analysis of stresses due to other forces (e.g. wind, earthquakes)
5.1	Research the alternatives to steel strap e.g. methods of adequately tensioning steel or barbed wire
6.2	Further tests on lime mortar specimens - strap application and loading tests
6.3	Further tests to characterise the behaviour of renders – cracking, shrinkage and adhesion with a variety of admixtures and curing regimes
6.6	Tests of free-floating brick cylinder specimens using both cement and lime mortars
Other	Preparation of design and construction guidelines for this style of tank. Construction of full size tank to look at the following: ▪ tank construction techniques ▪ stress analysis in full size tank – including cyclic loading and temperature effects ▪ shrinkage and cracking due to water pressure loading ▪ leakage

9. References

1. Stulz, Roland and Mukerji, Kiran, Appropriate Building Materials, SKAT, Second impression, 1998
2. Watt, S.B., Ferrocement Water Tanks and their Construction, IT Publications, 1978
3. Holmes, Stafford and Wingate, Michael, Building with Lime, IT Publications, 1997
4. Gray W.S. (revised by Manning, G. P.), Reinforced Concrete Reservoirs and Tanks, Concrete Publications Ltd, 1960
5. Shigley, J. E. and Mischke C.R., Mechanical Engineering Design, McGraw-Hill International Editions, 1989
6. Vadhanavikkit, Chayatit, Brick Water Tank, Paper presented at the Regional Rainwater Catchment Seminar and Workshop, Khon Kaen, Thailand, IRDC, 1983

Low-cost, thin-shell, ferrocement tank cover

Instructions for manufacture

Development Technology Unit
School of Engineering
University of Warwick
Coventry CV4 7AL
Tel: +44 (0)1203 522339
Fax: +44 (0)1203 418922
dgr@eng.warwick.ac.uk
http://www.eng.warwick.ac.uk/DTU/

February 1999

Introduction
The thin-shell ferrocement tank cover is designed in such a way that it can be manufactured without the use of a mould or shuttering. It can also be manufactured remote from the tank to which it is to be fitted and moved into place once complete. The aim is to reduce the cost of the tank (cover) by eliminating costly shuttering or moulds and by reducing the quantity of material used to manufacture the cover. It also means that the cover can be removed at a later date for maintenance, refurbishment or cleaning. The cover can be manufactured by two persons (one skilled and one unskilled) in a single day (with some time required after that for curing) using tools required for the construction of a simple cylindrical ferrocement tank.

The design is based on a frame known as a reciprocal frame, that has spokes that, when loaded, put little radial loading onto the structure on which it sits. The frame is covered with a wire mesh that is then rendered with a sand cement mix.

Details of the construction process are given here for a 2.0m diameter cover that has an inspection chamber opening of 0.5m. The cover pitch is 25^{o}. Strength tests have proved acceptable up to this diameter. No guarantee is given for greater diameters. The spoke angles have to be recalculated for different diameters – this is one disadvantage of the cover design.

Benefits of the thin-shell ferrocement tank

- low cost – reduced use of materials
- no shuttering or mould required
- strong and lightweight – the tank cover is designed to be strong (through good quality control) and light at the same time
- good quality control can be achieved through easy working environment
- can be manufactured by two people in a single day (one skilled and one unskilled)
- no clambering on top of tanks required during construction
- can be cured easily – in the shade and at ground level
- can be batch produced at one site

Reciprocal frame construction guidelines – for 2m diameter cover

Materials and tools

Materials	Tools
♦ 8mm reinforcing bar – 40m	♦ hacksaw
♦ tie wire – 0.5kg	♦ pliers
♦ chicken mesh (0.9m wide; 10m long; ½” mesh)	♦ tin snips
♦ sand	♦ vice (handy if available)
♦ cement	♦ masons trowel (small)
♦ mortar plasticiser	♦ masons trowel (large)
♦ water	♦ plasterers float
♦ plastic sheeting (reusable)	♦ shovel
	♦ buckets (2)
	♦ wheel barrow (optional)

Stage 1 – making the frame

- Choose a location with plenty of space to work. The procedure requires bending long lengths of reinforcing steel and so a clear working area is essential. Also a ground space of 2m diameter will be needed where no other activity will be carried out for a week (while the cover is cured).
- The first step is to set up a jig for bending the reinforcing bar. The jig is made up of two pegs 5cms long, set about 5cm apart. The steel is placed in the jig and bent as shown in Figure 1. The jig needs to be fixed so that it cannot move when the steel is bent. A workbench is ideal where the pegs can be put into the vice. Alternatively the pegs can be driven into a heavy piece of timber and this arrangement can be used effectively. Steel re-bar (8mm) can be used to form the pegs, but slightly heavier steel is better.

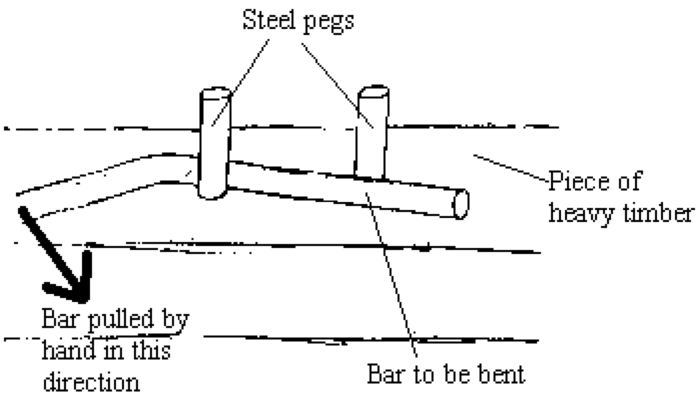


Figure 1 – Jig for bending steel reinforcing bar

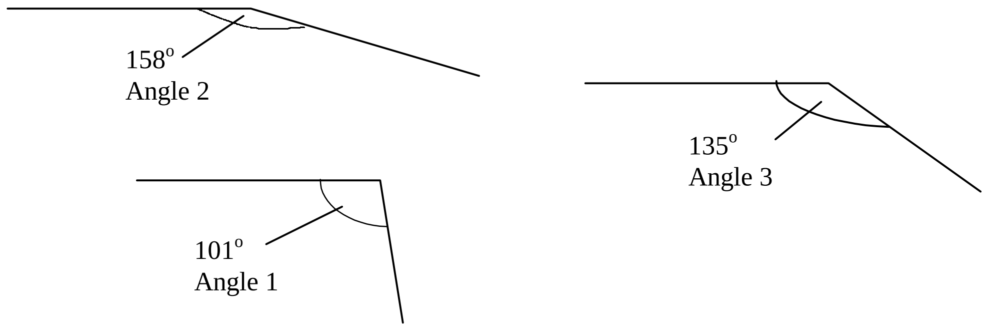
> Tip:
> When bending the re-bar it does not bend exactly where it makes contact with the jig peg. The bending takes place a cm or two on the pulling side. This has to be allowed for when bending. The bending radius can be quite large because of the thickness of the steel. This doesn't present any real problems here.

- The next step is to bend the 8mm reinforcing steel into hoops. Four hoops, diameter 0.55m, 1.0m, 1.5m and 2.0m are required. To make the procedure easy, a peg can be knocked into the ground and used as a centre around which the four circles can be drawn using string and a marker (also mark the positions of the 8 spokes at 45^o intervals for later use). The steel can then be bent gently in the jig to match the circles. The hoops ends are tied with two or three pieces of tie wire. For this the steel is cut slightly oversize to allow for tying. The cutting lengths are given in Table 1. Where the cover is to be fitted to an existing tank the outer hoop should be bent to fit the mean radius of the top of the tank wall and any irregularities in the shape should be taken into consideration.

Diameter	Steel cutting length (add 0.2m for overlap for tying in all cases)
0.55m	1.72m (1.92m)
1.0m	3.14m (3.34m)

1.5m	4.71m (4.91m)	
2.0m	6.28m (2.48m)	

- At this point all but the outer (largest) hoop can be put aside until later.
- The next step is to bend the spokes. There are eight in number and are bent in the jig to the dimensions shown in Figure 2. The cutting length is 1.33m. To aid the bending, the angles can be marked out on the ground (or on a bench) beforehand and then the bent steel can be matched against this. The angles to mark are:



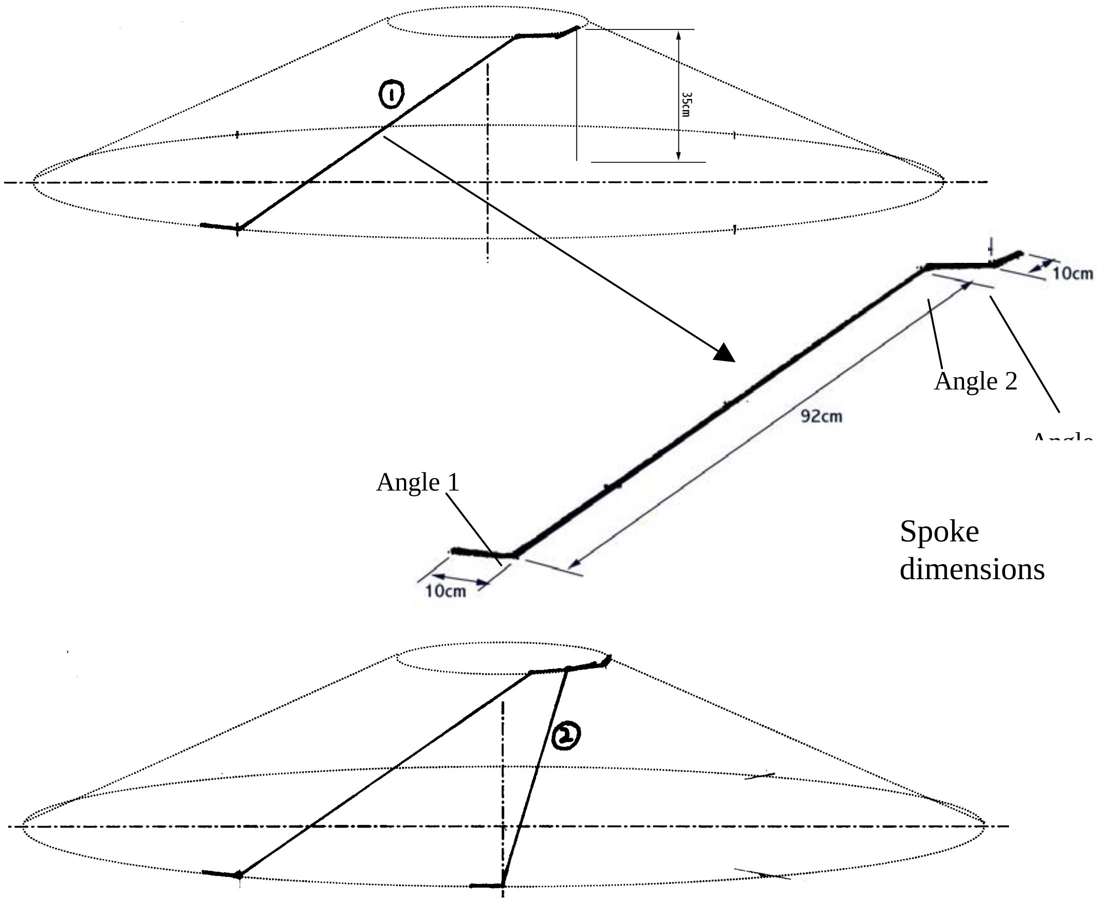


Figure 2. Dimensions and locations of frame spokes

- It is recommended that Angles 1 and 3 are bent first. These are bent in the same plane. The spoke is then turned through 90^{o} and Angle 2 is bent.
- Eight secondary spokes are also cut to a length of 75cm. These are wired to the frame as shown in Figure 3 and support the mesh to reduce the 'panel' size.

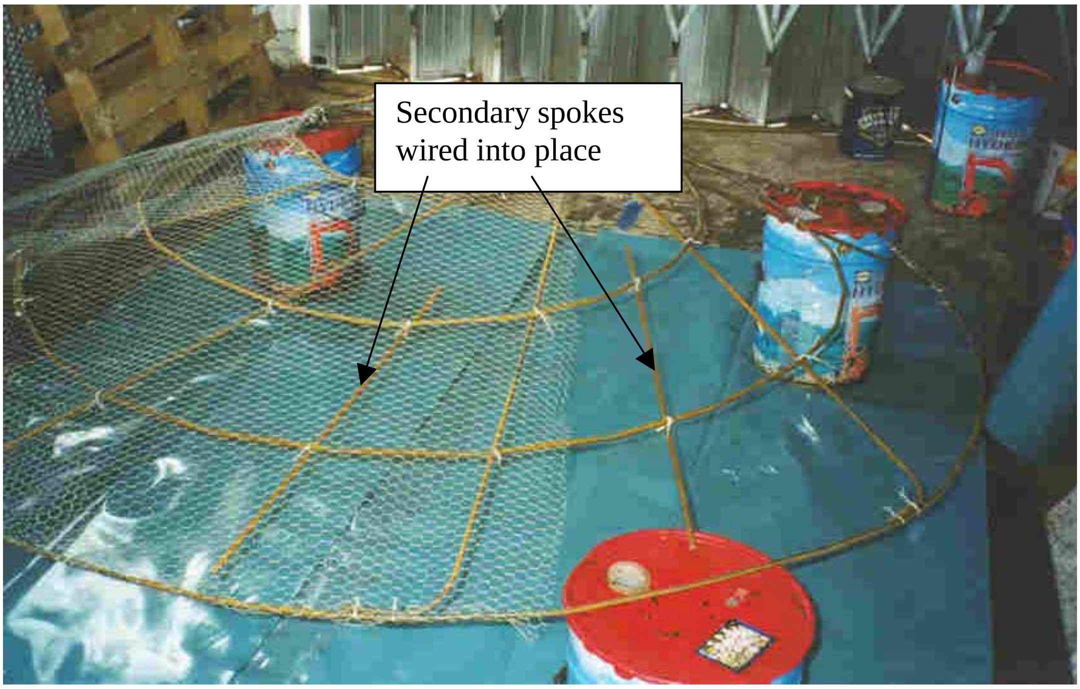


Figure 3. Secondary spokes in place

- Now the spokes are placed one by one inside the outer hoop (as shown in fig 2.) to slowly form the cover frame. It is convenient to have the outer hoop sitting on the ring marked out earlier with the position for the 8 spokes marked also. THERE IS NO INNER RING. This is made up as the separate spokes are joined together. (See Figure 4). Spoke one is placed on a support (a box or piece of wood) which is 35cm high. This is the height of the frame from the ground to the plane of the circular cover opening.

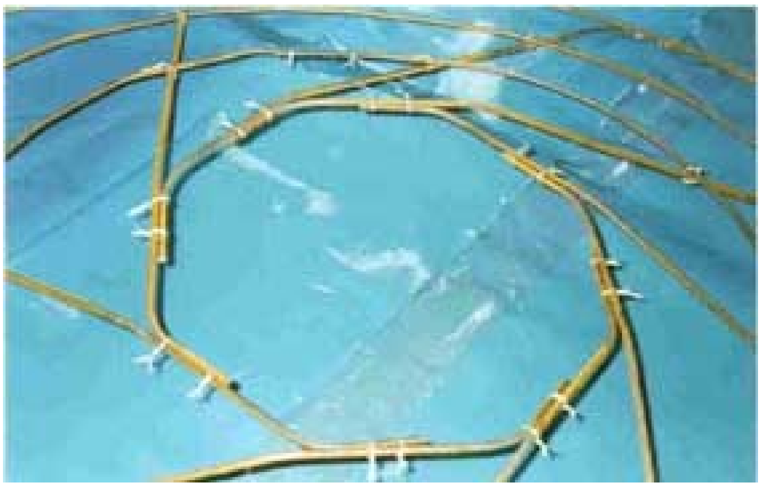


Figure 4. Showing the formation of the inner ring from individual spokes

- Tie the first spoke to the inner side of the outer hoop as shown in Figure 5.**(no 11).**

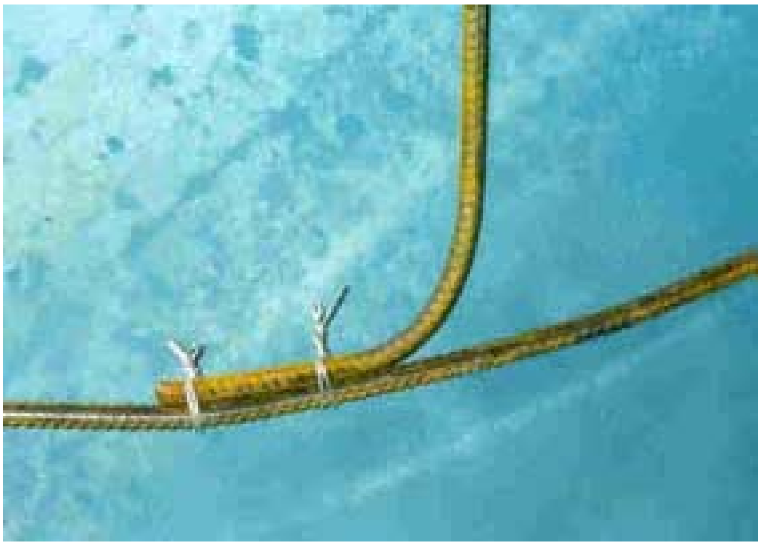


Figure 5. Showing arrangement for tying spoke to outer hoop.

- Place the next spoke 45^{o} around the perimeter hoop (these spacings were marked earlier) and tie it to the first spoke as shown in Figure 6. Continue in this way until the final spoke is tied to the first spoke and all eight spokes are in place.

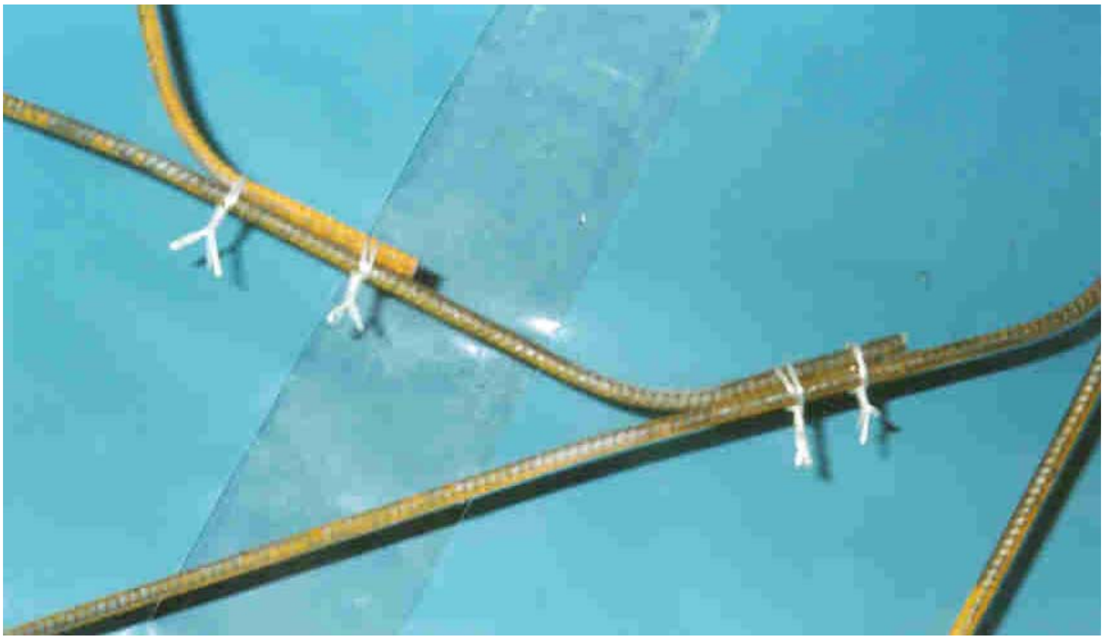


Figure 6. Arrangment for tying spokes to each other.

- Put the two inner hoops in position and tie them in place (Figure 7). The small inner hoop that was formed earlier will be used when the access hatch lip is made later.
- The frame in now ready to have the chicken mesh attached.

Figure 7. The frame with hoops in place.

- Use chicken wire of 0.9m roll width with a mesh size of ½ inch. Ten metres length is required. Two layers of chicken netting are applied.
- Eight pieces of chicken wire are cut to the dimensions shown in Figure 8. Two pieces can be cut from a 2.4m length of netting if cut as shown. A template can be drawn on the ground to aid cutting.

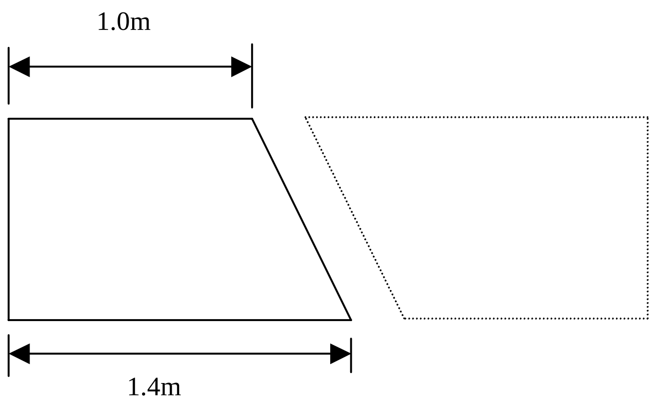


Figure 8. Cutting size for chicken mesh

- The pieces of netting are placed on the frame as shown in Figure 9 and Figure 10 and the overlapping edges are folded over and tied in place, pulling the wire as tight as is possible without distorting the mesh. Use the rough edges of the netting to tie the folded edges into place. Use as little tie wire as possible at this point, as the netting will tied securely when the second layer is in place.

Tip: a screwdriver can be used to pull the loose wires or end loops through holes in the mesh to tie the mesh in place.

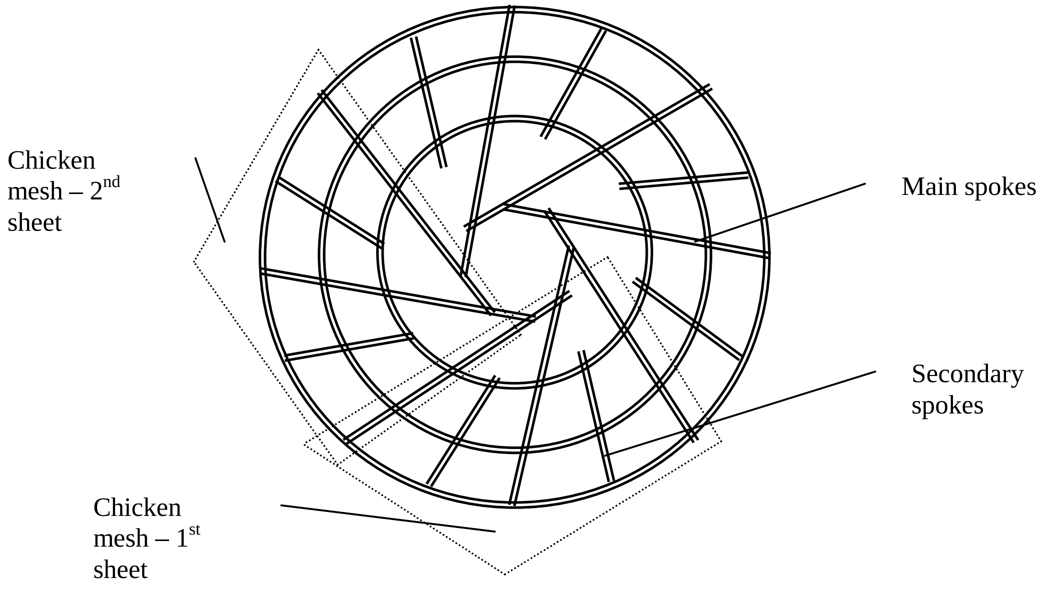


Figure 9. Pattern for application of chicken mesh

Figure 10. Applying the chicken mesh.

- When the first layer is complete start the second layer one spoke out of phase with the first and complete in the same manner.
- Carefully check that the mesh is folded as flat as possible and that both layers are close together. Tie the netting at regular intervals using the tie wire so that the netting is close to the rebar. Bend all tie wires into the plane of the cover. Remember that we are trying to keep the cover as thin as possible.
- The cover is now ready for the rendering (Figure 11).

Figure 11. Cover ready for rendering

Stage 2 - Rendering the cover

Materials

Sharp sand – 150kg
OPC – 30kg
Mortar plasticiser – 0.5 litres (soap powder can be used as a substitute)
Water - 20 litres (approximately)

Procedure

- It is important to use good quality materials and to maintain good standards of workmanship throughout the rendering process. The aim is to apply a layer of mortar to the chicken mesh that is as thin as possible. This, in practice, will vary between about 15mm and 25mm with an average thickness of about 20mm. The first coat is applied from the top and second coat applied from below.
- Put a plastic sheet on the ground so that render mix which falls through during rendering can be reused.
- Elevate the frame so that work can be carried out from above or below. Waist height is most suitable. The frame should be raised on 4 posts or boxes so that it is stable and can withstand the forces applied during rendering. A support should also be placed in the centre to prevent the centre sagging under the weight of the render (see figure 11 above).
- Render preparation: a mix of 3:1 (sand:cement by *volume*) is used. A sharp sand should be used i.e. not a fine sand but sand with a moderately large grain size. There should be no silt or other contaminant in the sand. (**See guidelines for checking sand quality)**. Ordinary Portland Cement (OPC) is used. The quantities should be carefully measured using a container – a bucket for example (do not measure using a shovel as this can be very inaccurate).
- The consistency of the render is very important. It should be dry enough not to fall through the netting while being plastic enough to be workable with a trowel. A mortar plasticiser is required to improve the workability of the render. This means that the water:cement ratio can be kept low while still keeping the render plastic. This ratio should be kept to approximately 0.4 by weight (i.e. 10 parts cement to 4 parts water by *weight*). Low water content not only gives a render which is easily applied to the mesh, but also gives improvements in strength and permeability of

the cured render. In practice it is difficult to control the water:cement ratio because there is usually an unknown quantity of water in damp sand and plasticity is often achieved before the minimum measured ratio is met. The practical method involves experimentation to achieve the desired plasticity with minimum water content. The plasticiser should be used according to the manufacturers instructions.

Tip: use soap powder instead of mortar plasticiser. Experiment to find a suitable quantity.

- Keep mixes small because the render 'goes off' quickly. It may be wise to mix enough render for the whole job and then add water to small amounts as required.
- Applying the render: this is fairly simple to do. Use a plasterers float and a small trowel. Put the float behind the mesh and work the mortar through the mesh onto the float as shown in Figure 12. Wipe the float away so that the mortar is slightly smoothed on the underside. Work small areas – take one 'panel' at a time and complete it. Some of the mortar will fall through onto the plastic sheet – this can be picked up immediately for reuse. Remember that the aim is to apply a very thin layer of mortar. The technique can be easily learned with a little practice.

Figure 12. Applying render to the chicken mesh.

- Where the cover stands on the supports, leave a small section of the outer hoop un-rendered. Wires can be threaded through these gaps later for lifting the cover into place and any securing to the tank body can be done here.
- The outer edge of the tank should be rendered roughly as this will blended into the tank wall when it is put into place.
- Once the first layer of mortar has been applied the cover should be left for a day to allow the render to gain strength.
- The area within 10cms of the inspection opening should be roughened for keying in the lip. A strip 20cms wide from the outer edge to the inner edge should also be roughened to take the access strip (see Figure 13). Tie wires should also be poked through from the underside to tie the access strip reinforcing in place when the render has gained strength.

Figure 13. Showing finished access strip and old tyre used as former for access hatch lip.

When the rendered cover has been sitting for one day the following work can be done:

- Three lengths of steel should be cut and placed radially where the access strip is to be located. They are tied in place. The access strip is then laid using 3:1 mix render to a depth of 2cms. This is then scored to give grip when climbing to the access hatch.
- The lip of the inspection chamber is built up with mortar to a total thickness of 4cms. The 0.55m steel hoop of is placed on top of the existing render and the lip built up to the desired shape. A former can be manufactured to aid in this process or an old car tyre can be cut to give the correct diameter and supported in place (see figure 133 above). A greater lip thickness gives a greater feeling of security to people working on or in the tank.
- The cover is then cured for 7 days. The tank should be wetted twice daily and covered with plastic sheeting to prevent evaporation of the curing water. It is essential that curing is carried out properly.
- A coat of ‘nil’ (pure cement water slurry) is applied to top and bottom after two days of curing.

Putting the cover in place on the tank

- When the cylindrical tank body is being constructed some thought should be given to the method of fixing the cover to the tank. If the cover is to be fitted to a thin walled ferrocement tank four (or more) tie wires should be left protruding from the tank wall and these are tied to the cover when it is in place. For brick, block or masonry walls, the cover can be laid on a bed of stiff mortar and then blended with the tank as shown in Figure 14 below.
- The cover can be lifted into place by four strong people. Strong wire can be placed around the outer hoop where the cover was left un-rendered. Two strong timber poles can be placed through these wires. These poles are then lifted by four (or eight) people Alternatively two poles can be placed under the rim of the cover, but this makes it more difficult to set the cover down.
- Special care should be taken not twist the cover or put any undue stress on it as this could cause it to crack.
- If the tank wall is quite high then a raised platform should be constructed (from earth or timber) to stand on.

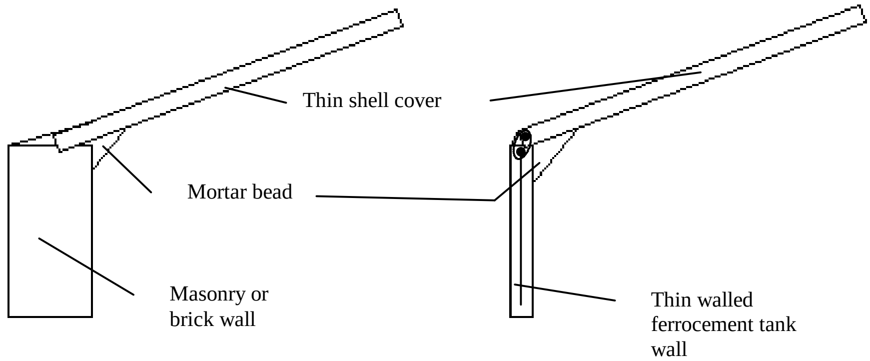


Figure 14. Blending the cover with the tank wall.

- A suitably sized ferrocement disc can be cast as the access hatch cover or another option used if so desired. This should be well fitting to prevent insects and contaminants entering the tank.

Tests for thin-shell, ferrocement tank cover

1. Point loading

The following load was applied with no adverse effect to the cover:

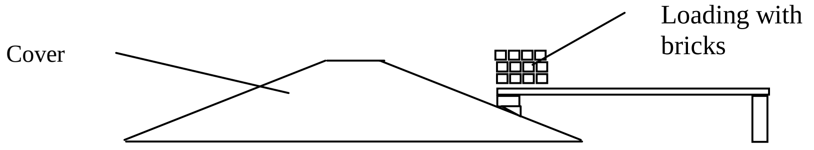


Figure 1 – Loading for point load test

Area of point load = 200 x 100mm = 20,000mm^2

Load applied 160 kg

i.e. 8kg / mm^2

2. Uniform loading

The cover was tested in two modes: constrained at the periphery to prevent slipping and unconstrained. In both cases the cover was loaded to approximately one thousand kg using house bricks (see Figure 4 below) and the deflection at the centre was less than 2mm in both cases, measured with a dial micrometer (Figure 3).

Figure 2 below shows the deflection against load for the constrained and unconstrained uniform loading.

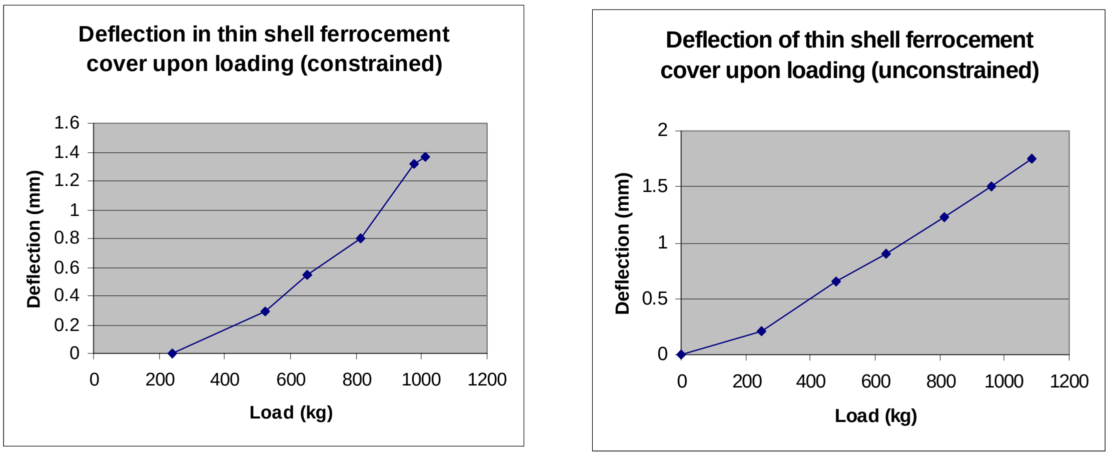


Figure 2 – Deflection of thin-shell ferrocement cover under load a/ constrained at edge b/ unconstrained

Figure 3 – dial gauge in place to test deflection of cover under load

Figure 4 – Thin-shell cover under a load of 1000kg of bricks – an evenly distributed load

The mortar dome cover – guidelines for construction

(Taken form a DTU working by Terry Thomas, Ben McGeever and members of URDT, Uganda)

1. Introduction

The mortar dome cover was developed as part of a collaborative project between the Development Technology Unit and the Uganda Rural Development and Training Programme (URDT), Kampala, Uganda. The cover is being reported here as an example of very low-cost technology for Roofwater Harvesting. The cover was part of a design of tank for underground storage of rainwater. The cover is also suitable for use on above ground tanks.

The Cover is a dome of mortar (containing almost no reinforcement) connected to a reinforced ‘ring beam’ set into the ground. The mortar dome and the ring are made at the same time over a carefully shaped mound of earth. Set into the mound are a bucket and a large plastic bowl. The bucket is to create a way for the rainwater to enter. The bowl is to create a hole to hold the plug in which the pump is set. It has to be large enough (e.g. 0.45 meter diameter) for a man to enter through. The 5 steps in making the dome will now be explained in turn.

2. Construction steps

Step 1 - Making the ‘template’ for shaping the dome

The shape of the mortar dome comes from the shape of the mound of earth it is built on. We therefore need a template to accurately form that mound of earth. Before building the first tank it is necessary to cut this wooden template. Once made, the template becomes a tool that can be used for many more tanks. The template must be the right shape and also strong enough to carry around and use without getting broken. It therefore consists of a piece of plywood, or thin planks, cut to that shape and stiffened by strips of thicker wood.

The right shape for the dome is approximately a upwards ‘catenary’. A downwards catenary is the shape taken by a chain hanging between two nails on a wall, so we mark the template out using such a chain (e.g. 1 or 2 lengths of bicycle chain) and then turn it upside down.

First cut the plywood so that it measures 125 cm by 100 cm and has square corners. Figure 3a shows 2 nails spaced 2.2 meters apart on a horizontal line drawn across a flat wall using a spirit level. Draw a vertical line down the wall from midway between these two nails and mark a short line (the ‘mark’) across it 80 cm below the horizontal line. Hang a light chain between the two outside nails and adjust its length until it just reaches down to this mark. (If you do not have enough chain to do this, see the alternative below.) Slide the thin plywood behind the chain without touching it, so that the long top of the plywood touches the left-hand nail and the right side of the plywood lies along the vertical line. With a pen, copy the shape of the hanging chain onto the plywood, remove the plywood from the wall and saw along the line you have just marked. (Using planks instead of plywood, first nail them rigidly to their stiffening bar so that they can be placed behind the hanging chain; then continue as for plywood).

Although it is easiest to make the catenary with two bicycle chains joined end to end, it can also be done with only one. This has to be hung so that it forms just over half the full U-shaped catenary: one end of the chain is attached to the left-hand nail, the other end is held low and pulled until the lowest point of the chain falls exactly over the ‘mark’. You can now drive in another nail (‘alternative nail position’ in Figure 1) to attach the chain to, while you are copying the chain’s shape onto the plywood.

It is necessary that the chain has no twists and that it hangs freely, otherwise it might take up the wrong shape. The right shape ensures that the mortar dome is strong (by being everywhere ‘in compression’). Rope is not usually suitable instead of chain, because most ropes twist and are not heavy enough to hang properly.

To finish the template, stiffen it with good wooden strips. Now turn the template over so that the long straight side is on top and write the word ‘TOP’ next to it. Smooth the sharp corners to make it safer to carry.

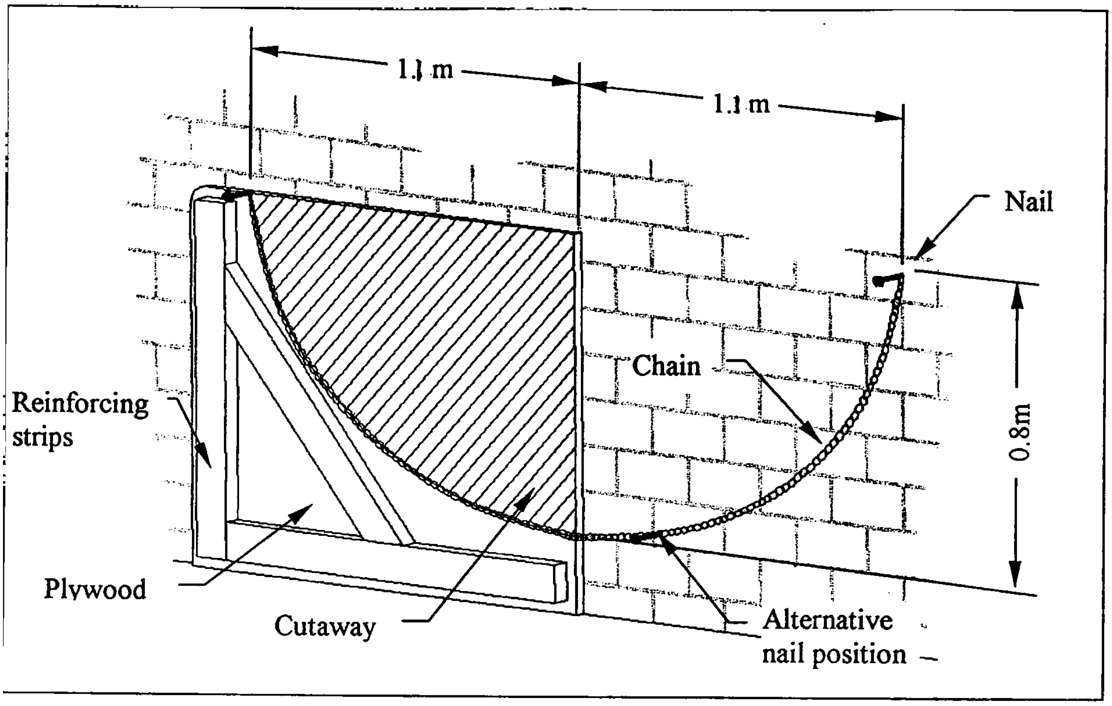


Figure 1 Making the template

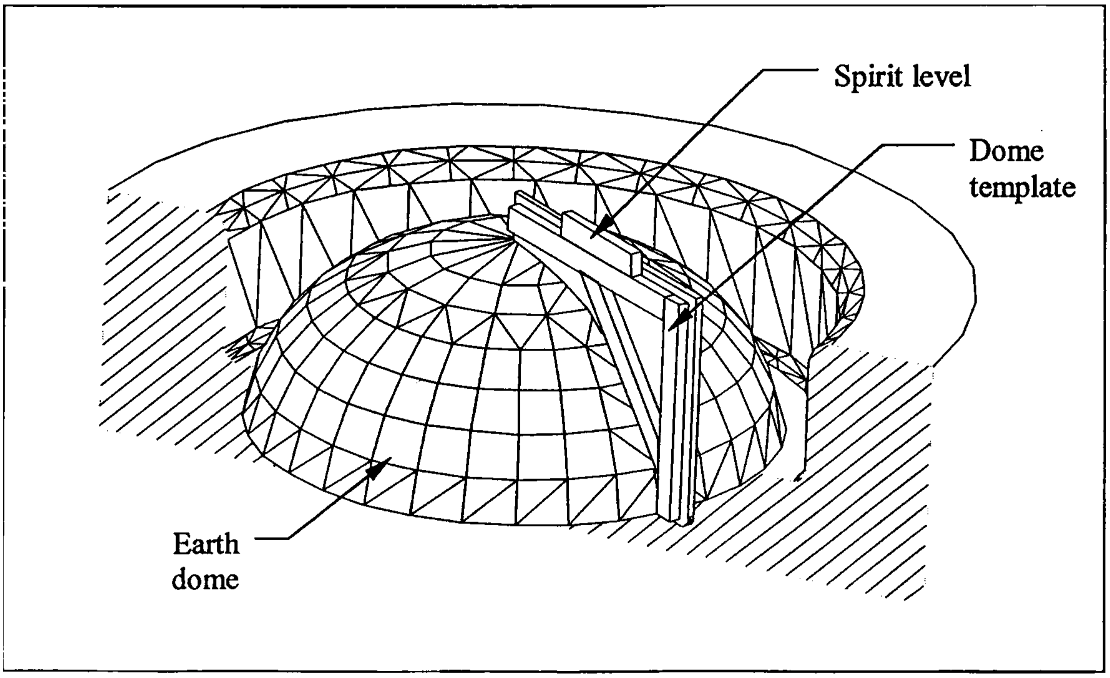


Figure 2 Forming the earth mound

Step 2 Marking out and making the trench and earth mound

The centre of the cover should be marked by a firm and vertical (use a spirit level) thin stake. Make a clear ink mark or cut a ring round the stake about 30 cm above the ground. Using a string 110 cm long looped once round the stake, mark out a circle of diameter 220 cm on the ground. This circle marks the inside edge of the trench in which the ring beam will be cast.

Dig a narrow trench (one hoe's width) outside this circle and throw some of the soil into the centre round the pole. The idea is to dig down 50 cm leaving a mound of firm soil inside the ring rising up to the ring round the stake. The shape can constantly be checked using the template - now with 'TOP' at the top - placed against the stake and rotated like a scraper. The template should be kept level by means of a spirit level and at the right height with its lower corner touching the ring marked on the stake. This is shown in Figure 2.

If the mound is rough or loose or fissured by drying, it can be plastered with more mud and wooden 'floated' to make it smooth and firm.

Chicken mesh can be fixed in the trench so that later on it can be used to improve the joint between the mortar lining the tank wall and the mortar of the ring beam. Make a single strip of mesh by cutting a 1.5 meter length into 5 strips each about 18 cm wide and twist joining them end to end - the final strip should be adjusted to fit round the inside face of the trench like a ring. This ring should now be folded longwise into the vee-shape shown in Figure 3 and the inside half buried in the earth of the dome. To do this you will have to cut out some earth from the inside of the trench, place the chicken mesh then plaster back the earth again.

The trench is now too wide for the ring beam, so fill back a step 10 cm high round its outside so that its bottom becomes only as wide as your foot - about 10 cm. (You will need to walk round this slot when you are plastering the dome). This too is shown in Figure 3. The bottom of the earth dome that faces into the trench should be grooved with a trowel or stick: these grooves will be ‘copied’ onto the inner edge of the ring beam and will later help ‘key’ the plaster joint to be formed there.

Finally place the bucket and the basin on the dome as shown in Figure 4. The bucket (the inlet) should be on the side nearest the house, with its edge touching the stake. The large basin (for the excavation access and later the pump hole) should be on the other side of the stake and with its edge 25 cm from the stake. Weight down the bucket and basin with stones and push them into the soil mound so that they do not rock; local excavation will allow the bucket to be sunk a desirable 20 cm into the soil. Put a small fillet of mud round each bowl as shown.

Pull out the stake without disturbing the mound.

Step 3 Preparing the reinforcing bars

Use 6 mm bar; it does not matter whether it is round or knobbly. Make a ring whose diameter is 230 cm, folding over and linking the ends and hammered the link tight so that there is no play in the joint. This ring will take about 8 meters of bar. Test that the ring will sit in the middle of trench without getting close to either its inner or outer edge.

Make two further such rings but much smaller, one each for the bucket and the bowl. Each ring should have a diameter bigger than its bucket/bowl so as to leave a clearance of 3 cm all round it where it enters the soil dome.

Step 4 Casting the ring beam and the pierced dome

The dome and the ring beam that forms its bottom edge are made of strong mortar in the manner shown in Figure 5. The mix is 1:3 (cement : sand) and 2 bags of cement should be ample. Concrete, mixed 1:4:2 (cement : sand : small sharp aggregate), is an alternative where such aggregate is available or can be made; a concrete dome needs only 1.5 bags of cement. (Concrete is more difficult to place as a plaster than is mortar and the surface finish achievable is not so good.) The ring beam is about 10 cm x 10 cm, while the rest of the dome is covered with 2 cm of mortar. However round the bucket and bowl this depth is increased locally to about 8 cm to make a good lip to hold the bucket/bowl and to cover the reinforcing rings there. As usual all three rings of reinforcing bar must be in the middle of the mortar with several centimetres of cover on all sides. So they must be placed as the mortaring progresses. The big ring, in the ring beam, is therefore placed only after 5 cm of mortar is already in the trench.

It is important to check the mortar thickness nowhere gets less than 2 cm as you work up the dome. There should be no joints in the mortar: the whole dome and ring beam should be made (plastered) in a single session with a mix that is dry enough not to slump. As the soil dome may suck water out of the mortar or concrete applied on top of it, it should be thoroughly wetted before plastering the dome starts. Moreover in a hot climate it is wise to do this plastering early in the day so that the new dome can be covered with wet straw before the sun gets very hot.

Step 5 Curing the dome

As soon as the mortar is firm, gently remove the bucket and basin from the top of the dome.

Once the dome is cast it needs to cure under moist conditions for 14 days to develop a high strength. The simplest way to ensure it is kept moist is to cover it with plenty of grass and douse this with a jerrycan of water every morning and afternoon.

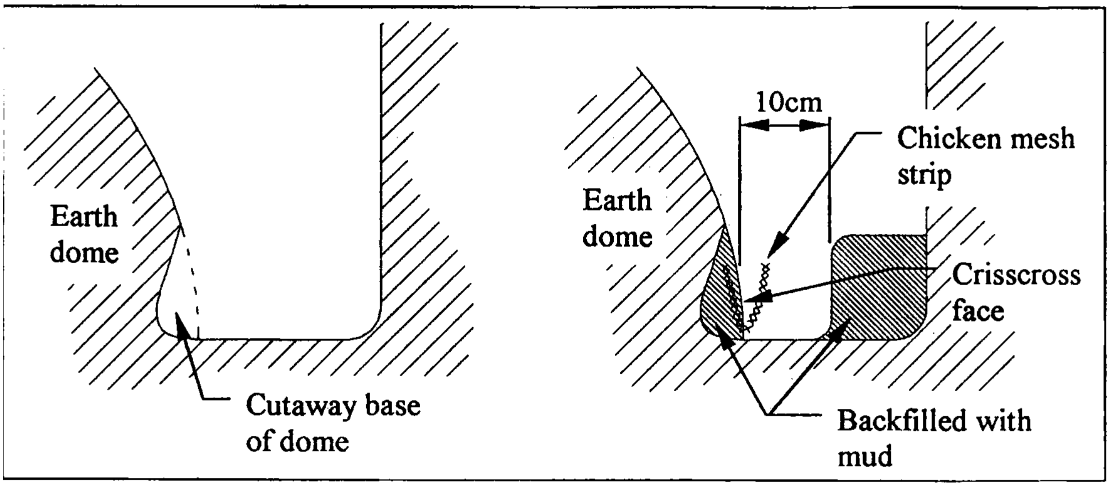


Figure 3 Details of trench (mesh is optional)

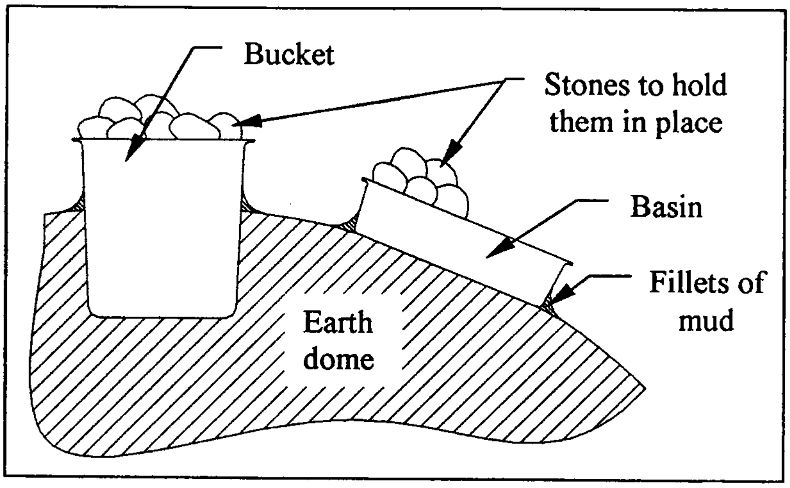


Figure 4 Basin and bucket on mound

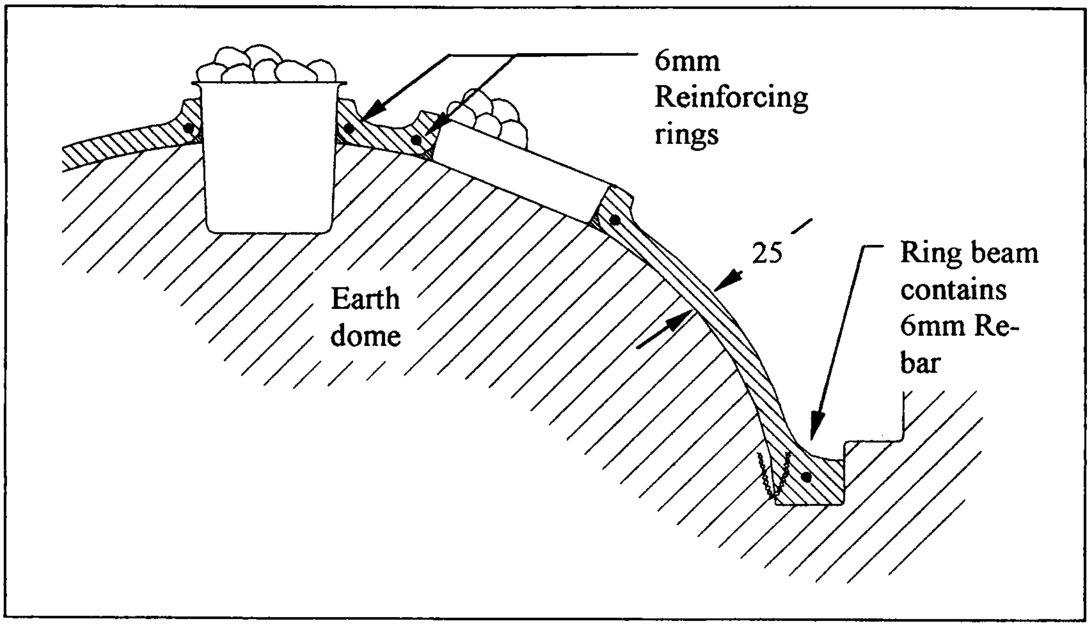


Figure 5 Completed dome

Milestone A3

'Completion of one externally reinforced brick tank, two brick test cylinders, one partially below ground tanks and five thin-shell ferrocement tank covers'.

(Originally titled 'Completion of 6 underground tanks with instrumentation')*

University of Warwick

September 1999

Contents..........1

*Early in the programme, after the initial desk study had identified the areas where further research is required, it was decided to broaden the original scope of the water storage (technology) component of the programme to include above ground, free-standing water storage tanks as well as underground tanks. The focus of this work, as reported in Milestone A2, has been primarily the research into externally reinforced, single-skin brick tanks. It was also decided that we would put some resources into developing a simple, cheap, lightweight, tank cover, as this was seen as an area where little work has been done and where there was much scope for improvement for relatively little input. The title of this Milestone has therefore been altered to reflect the practical research work that has actually been carried out under the programme.

1. Externally reinforced brick tanks

The early work on this topic was reported in Milestone A2 and covered the early tests that were carried out on brick cylinder specimens that were constructed at the University's civil engineering laboratory.

The aim of the tests were to determine the suitability of single-skin, burnt-brick, externally-reinforced (with packaging strap in this case) cylindrical tanks for water storage. The specimens were subjected to pressure forces in order to determine the hoop stresses induced in the strapping and the behaviour of the brick-strapping combination under load and at failure. The test results were then compared with a spreadsheet model that had been generated earlier.

The early results proved promising but were inconclusive due to some uncertainties in the experimental procedure. Further tests are currently underway at the University to more fully model the behaviour of the tank under load.

Figure 1 below shows one of the specimens under test, showing the internal expansion ring used to apply pressure. Figure 2 shows the steel strapping with a strain gauge that was used to measure the hoop stress generated. For fuller details of the tests, please refer to the Milestone A2 report.

Further work has since been carried out on the construction of a full-sized tank for rainwater storage at a field site in Herefordshire. The tank has a capacity of approximately six cubic metres, with a diameter of 2m and a height of 2m. The tank is at present complete, but without instrumentation as yet. The aim of the experimental tank (shown under construction in Figure 3 below), is to further investigate the behaviour of the tank with particular regard to induced stresses under real conditions and also to investigate the feasibility of a number of peripherals designs.

The tank has been built at the site of a Herefordshire based educational charity who run regular residential courses. The tank will store water captured from the roof of a large shower and toilet complex and the water is then pumped to a header tank from where it is piped to the toilets for flushing. The tank will experience regular cycling (emptying and filling) and hence will allow regular analysis of the stresses and achieve a simulation of accelerated ageing. The cost of the system was shared between the University of Warwick and the charity. The schematic diagrams for the system are shown in appendix 1.

The site will also give us a valuable UK based field testing station at which we can pre-test any idea or component before passing them on to partners or field contractors for further testing and development. The site has already been used to test plastic tank liners which are being developed at the University, as well as being fitted with a novel overflow/ washout system. The system is fitted with WISY filters, which will be tested for efficiency at a later stage.

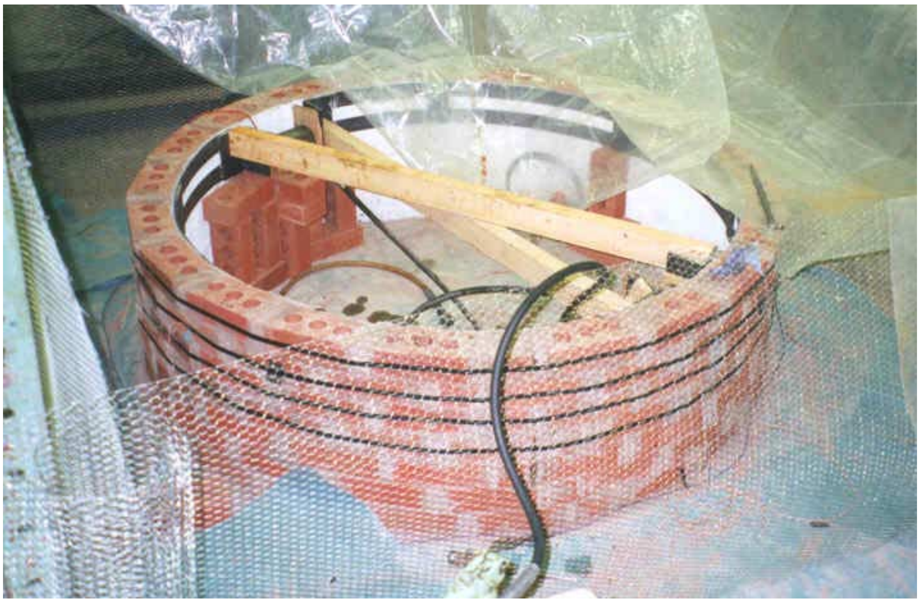


Figure 1 – Tests arrangement for externally reinforced brick cylinders

Figure 2 – Strain gauge used to measure hoop stress generated in steel strap during testing.

Future work in this area will include research into the use of cement stabilised soil blocks in place of brick, used in conjunction with a plastic tank liner, with the aim of reducing costs further.


Figure 3 – Full-size experimental tank under construction at field site.


Figure 4 – Strapping applied to experimental tank.

Results of the tests carried out at the University and at the field site will be reported in Milestone A5.

2. Partially below ground tanks

The work on partially below ground tanks is still in the very early stages and, to date, has been primarily field based. The work builds on previous DTU work in Uganda with fully below-ground tanks. Modifications were made to the original (fully below-ground) design to eliminate some of the problems commonly encountered with this type of tank, while still taking advantage of the beneficial aspects. These problems are:

- ingress of water from above ground causing contamination
- danger to children (and adults) if cover is left off the tank
- danger of collapse if driven over with a vehicle
- difficulties in excavation due to the construction process (see DTU Working Paper number 'Underground Storage of Rainwater for Domestic Use' at http://www.eng.warwick.ac.uk/DTU/workingpapers/wp49/index.html)
- no suitable overflow when tank overfills

The previous tank design is shown below in Figure 5. Approximately 15 of these tanks, which range from 8 to 10 cubic metres, have been built in western Uganda over the past 3 years with good results.

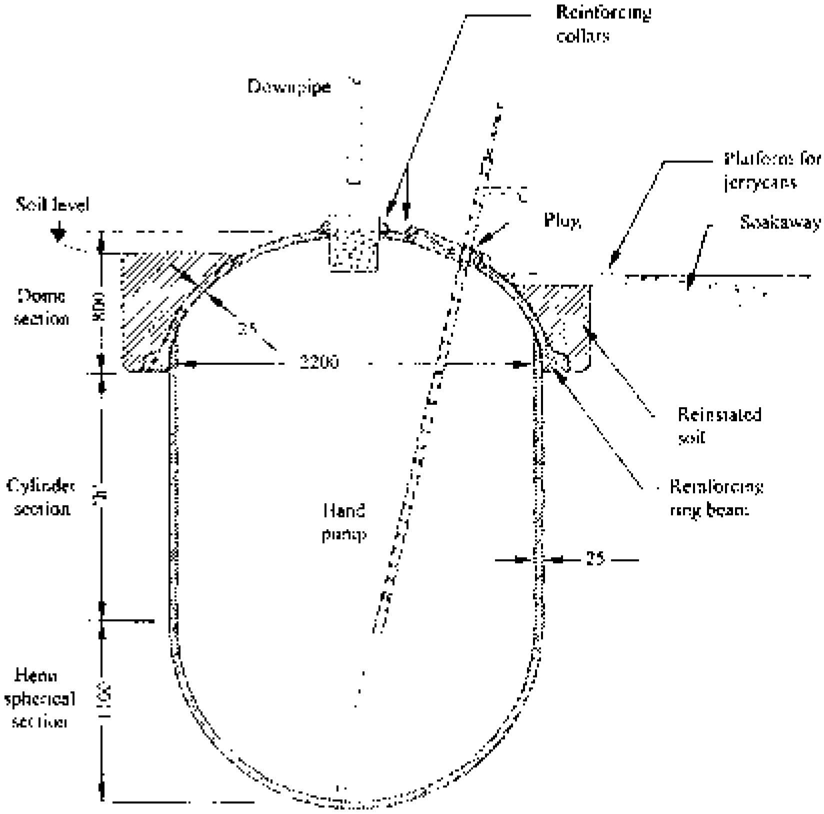


Figure 5 – DTU fully below-ground tank design

The tank is suitable only under certain soil conditions, namely where the soil is sufficiently stable to not cause any concern regarding subsidence. This is the case in many parts of Eastern Africa where the soil is lateritic.

The partially below ground tank (see Figure 6) is raised slightly above ground level in order to eliminate the problems listed above. The parapet wall sits atop a concrete ring beam through which the tank is excavated. The tank is lined with cement render and will, when the technology is sufficiently developed, be fitted with a continuous plastic lining. Water enters the tank through a pipe which directs it to the base of the tank which promotes rapid settling of sediment. The tank is fitted with an improved handpump and floating off-take. The floating off-take takes water always from slightly below the surface level of the water. These two innovations help maximise residence time in the tank, allowing full sedimentation to take place even when water is drawn off as fresh water is entering (except under conditions where the tank is almost empty).

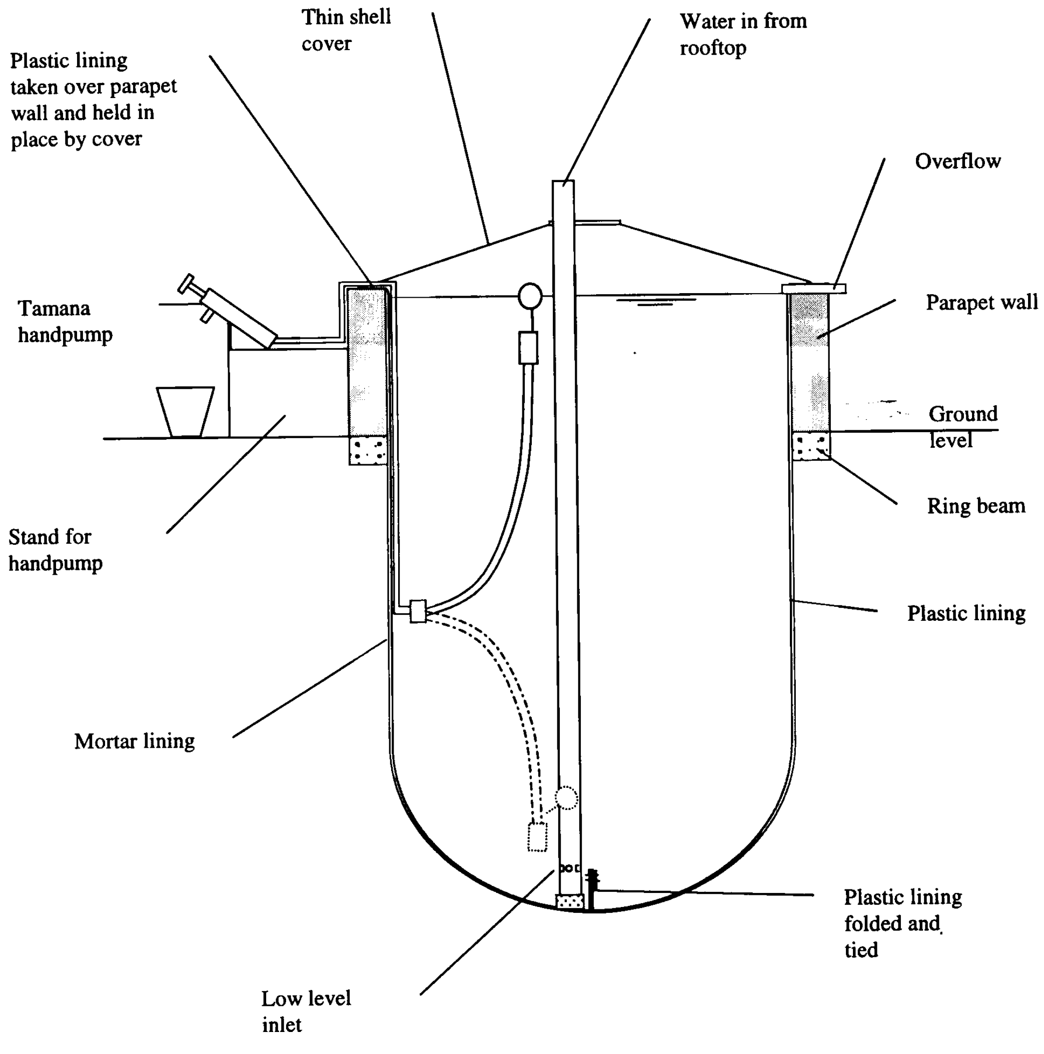


Figure 6 – Schematic representation of the cylindrical, partially below-ground tank

One partially below ground tank has been built in western Uganda. It was hoped that a number of tests would have been carried out by the time of writing this report, but due to logistical problems (funds transfer to Uganda which took 3 months), the tank has not yet been tested.

Figures 7, and 8 below, show the partially below ground tank under construction, while Figure 9 shows the thin shell ferrocement cover being manufactured.

Figure 7 – Bird's eye view of the partially below-ground tank under construction.

Figure 8 – Partially below ground tank under construction in western Uganda, showing parapet wall.

Figure 9 – Thin-shell ferrocement cover being manufacture for partially below-ground tank in western Uganda

Further information and test results form this work will be included in Milestone A5 report.

3. Thin-shell ferrocement tank covers

Milestone A2 report included documentation regarding the instructions for manufacturing a thin-shell ferrocement cover (TSF cover).

The thin-shell ferrocement tank cover is designed in such a way that it can be manufactured without the use of a mould or shuttering. It can also be manufactured remote from the tank to which it is to be fitted and moved into place once complete. The aim is to reduce the cost of the tank (cover) by eliminating costly shuttering or moulds and by reducing the quantity of material used to manufacture the cover. It also means that the cover can be removed at a later date for maintenance, refurbishment or cleaning. The cover can be manufactured by two persons (one skilled and one unskilled) in a single day (with some time required after that for curing) using tools required for the construction of a simple cylindrical ferrocement tank.

The design is based on a frame known as a reciprocal frame, that has spokes that, when loaded, put little radial loading onto the structure on which it sits. The frame is covered with a wire mesh that is then rendered with a sand cement mix.

Details of the construction process are given in Milestone A2 for a 2.0m diameter cover that has an inspection chamber opening of 0.5m.

Benefits of the thin-shell ferrocement tank

- low cost – reduced use of materials
- no shuttering or mould required
- strong and lightweight – the tank cover is designed to be strong (through good quality control) and light at the same time
- good quality control can be achieved through easy working environment
- can be manufactured by two people in a single day (one skilled and one unskilled)
- no clambering on top of tanks required during construction
- can be cured easily – in the shade and at ground level
- can be batch produced at one site

Five of these covers have been produced to date, 3 for testing purposes and two that have been fitted to tanks. One has been made in Uganda by local masons (see Figure 9 above and Figures 10 and 11 below) with no complications.

Figure 10 – 6mm steel framework being formed by local masons in western Uganda for the TSF tank cover.

Figure 11 – TSF cover being completed by local mason in western Uganda. This cover was fitted to a partially below-ground tank.

Tests have been carried out on the cover to determine its strength and the results are shown in appendix 2.

4. Appendices

Appendix 1 – Schematic representations of RWH system at field station in Herefordshire. (2 pages)

Appendix 2 – Test results for Thin-shell ferrocement tank cover. (2 pages)

Milestone A4

Completion of three partially below ground tanks, 2 rammed earth tanks, eight cement jars and two stabilised soil block tanks.

(Originally titled 'Completion of 3 sets of 4 tanks and associated instrumentation')*

Development Technology Unit (DTU)

University of Warwick

April 2000

Milestone A4 - Table of Contents

Funded by the European Commission

*Please see the introductory section for an explanation of the change in the title of this report.

1. Introduction

As mentioned in the Milestone A3 Report, early in the programme, after the initial desk study had identified the areas where further research is required, it was decided to broaden the original scope of the water storage (technology) component of the programme to include above ground, free-standing water storage tanks as well as underground tanks. The focus of the work was, initially, broadened to look at tank covers and reinforced brick tanks. We have since moved forward with more radical cost reduction ideas and have been experimenting with two very low cost water storage ideas: the partially below ground tank and the rammed earth tank. We also carried out some water quality experiments in Uganda to test the acceptance of water from cement lined storage vessels. For these experiments we constructed eight 400litre cement jars. Finally, through our connections with Uganda we were asked to carry out some collaborative research with Mr Moses Musaazi of Makerere University into the use of cement stabilised soil blocks for the construction of above-ground water storage tanks.

The title of this Milestone has therefore been altered to reflect the practical research work that has actually been carried out under the programme.

A brief overview of the work that has been completed for this Milestone is given in this report. A fuller report of the work that has been carried out, including the experiments carried out and results obtained for each of the pieces of work mentioned above, will be presented in Milestone A5 (Report A3 '*Performance of Tanks for DRWH'*). This report will be submitted in the near future.

2. Construction of three partially below ground tanks

As reported in Milestone A3, work was started on the design of the Partially Below Ground (PBG) tank in Uganda last year. The work has still, to date, been primarily field based and three tanks have been built until now, in the vicinity of Kyenjojo, Kabarole District in Western Uganda. The tanks have been built by our partner organisation, ARUCED. ARUCED has also received orders for a number of these tanks to be built privately and to date eight tanks in all have been built.

The aim of the design of the PBG is to minimise the amount of material used by partially submerging the tank below ground – the major benefit of sub-surface tanks. The fact that approximately 1metre of the tank protrudes above the ground helps overcome some of the drawbacks normally associated with below ground tanks (see Table 1 below). The ground conditions in this part of SW Uganda are ideally suited to this kind of tank, being lateritic and highly stable.

	Above ground tank	Below ground tank
Pros	• Above ground structure allows for easy inspection for cracks or leakage • Many existing designs to choose from • Can be easily purchased ‘off-the-shelf’ in most market centres • Can be manufactured from a wide variety of materials • Easy to construct for traditional materials • Water extraction can be by gravity in many cases • Can be raised above ground level to increase water pressure	• Generally cheaper due to lower material requirements • More difficult to empty by leaving tap on • Require little or no space above ground • Unobtrusive • Surrounding ground gives support allowing lower wall thickness and thus lower costs
Cons	• Require space • Generally more expensive • More easily damaged • Prone to attack from weather • Failure can be dangerous	• Water extraction is more problematic – often requiring a pump • Leaks or failures are more difficult to detect • Contamination of the tank from groundwater is more common • Tree roots can damage the structure • There is danger to children and small animals if tank is left uncovered • Flotation of the cistern may occur if groundwater level is high and cistern is empty heavy vehicles driving over a cistern can cause damage

Table 1. Pros and Cons of Tanks and Cisterns

The design also tries to minimise the amount of cement used by employing a plastic liner to form the water proof lining for the tank. Until now the plastic liner has not, however, been fitted and the water proofing has been achieved by means of cement render. Further work in Uganda in the coming months will include the manufacture of plastic liners and these will be fitted to subsequent tanks.

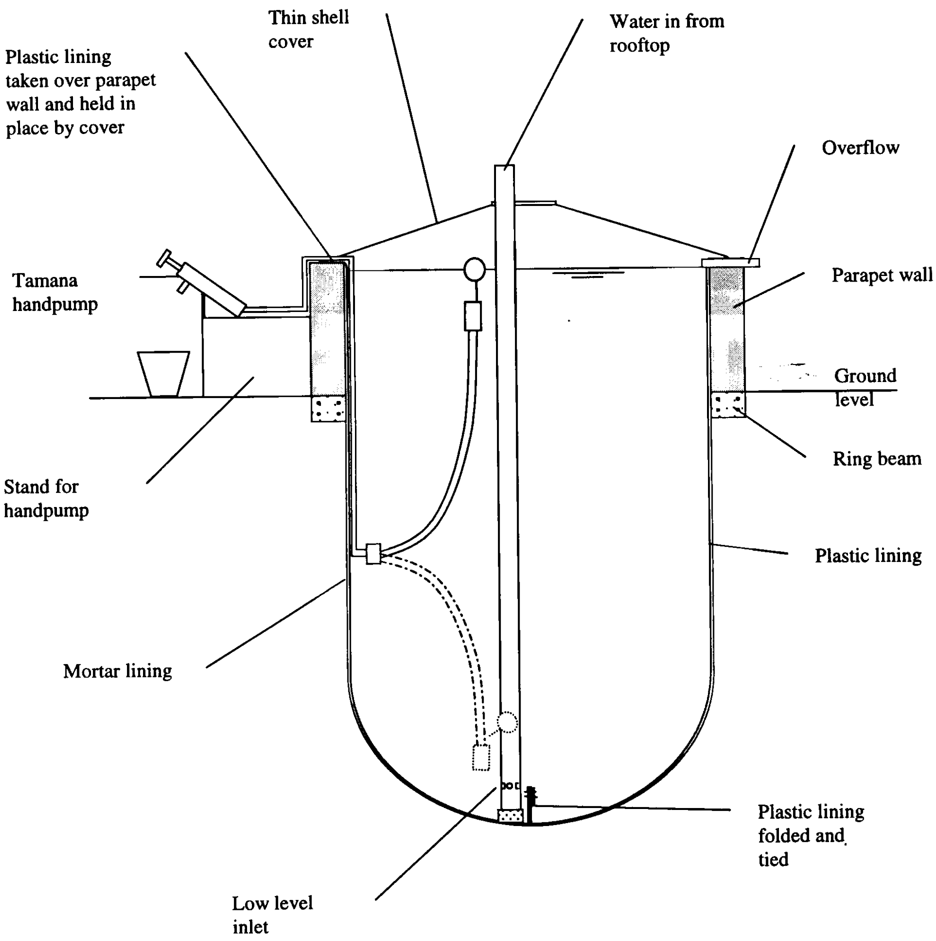


Figure 1 – Schematic drawing of the Partially Below Ground Tank

A costing exercise has been carried out to estimate the benefits of using the PBG tank against the traditional ferrocement tank (assumed to be amongst the cheapest options) and the rammed earth tank (described below). A brief summary is given Table 2.

	Ferro-cement tank	***Rammed earth tank***	***Partially below ground tank***
Material	394	179	107
Labour	81	98	90
Total cost	475	277	197

Table 2 - Comparison of costs between a ferrocement, rammed earth and PBG tank – all 11 cubic metres

Each of the PBG tanks was fitted with a thin shell ferrocement cover, described in a previous report and developed under this programme.

Figure 2 – A thin shell ferrocement cover near completion and ready for fitting to a PBG tank

Figure 3 - A completed PBG tank showing inlet pipe and handpump for extracting water

Initial leakage tests were carried out on a number of tanks using a device developed at Warwick (see Figure 4). These tests are continuing at present and will be reported later.

Figure 4 – Leakage test apparatus being tested in the field on a PBG tank

3. Construction of two rammed earth tanks

The direction of the experimental work at the University has shifted from reinforced brick tanks (reported in Milestone A3) to a more radical design of tank – the rammed earth tank. The aim is to adopt a low-cost technique commonly used for house building in areas of the world such as North Africa, North and South America and the Middle East. The technique uses a mixture of sand and clay, sometimes with a small amount of stabiliser such as cement, which is rammed, either manually or pneumatically, between wooden shutters. To use rammed earth as the structural material in a water storage vessel a waterproof liner is required. This can be provided by employing one of a number of possible options, such as cement render or a plastic sheet liner (as mentioned earlier, this latter is being developed for this application at the University). It is a technique that is relatively simple in essence and requires little in the way of imported material.

Figure 5 – A section of wall rammed between the wooden shuttering shown after removal of the shuttering

So far, the work that has been done on rammed earth tanks has been experimental, and mainly focused on adapting the basic technique (generally used for construction of straight walls in the building sector), to the construction of circular walls for water storage tanks. A 0.7m inner radius quadrant shuttering (see Figure 5) was manufactured and a number of test sections were rammed using a locally excavated soil (local to the University). The soil had to be modified somewhat to make it suitable in terms of cohesion. Wall thickness was increased gradually from a starting minimum of 60mm. We eventually opted for a 100mm wall thickness although even this is unsuitable for manufacture in the field. It is ideal, however, for testing purposes.

A tank, of radius 0.7m and depth 0.7m, has been constructed in a test pit at the University, and fitted with a plastic liner. The tank has been fitted with steel straps to give added hoop strength, but these will be removed after initial tests. The straps will be fitted with strain gauges and will be linked to computer data logging equipment. Measurements of strain, water depth, and temperature will be taken during tests. It is hoped that full-scale testing will take place within the coming month (May 2000). The aim is to seal the top of the tank with a large wooden disc, which will be pierced with a header pipe. The water pressure head will be gradually increased to a maximum of 5m while logging the variation in strain and water depth. The tank has been designed to fail at about 4m, so the test should be destructive, giving valuable information on the strength of rammed earth as a possible alternative to conventional building materials.

Figure 6 – Bird's eye view of the rammed earth tank under construction in the test pit at the University

Figure 7 – Revised shuttering arrangement shown during construction of the experimental tank

4. Construction of eight cement jars for water quality experiments.

There has been much discussion about the acceptability of water that is stored in vessels lined with cement mortar. Many users complain about the taste of the water, especially in the early days when the cement is still leaching calcium. An experiment was set up in Uganda, and carried out by the DTU's partner organisation, ARUCED. Eight small jars of 400 litres each were build and cured using different methods. The aim these experiments was to try to quantify the acceptability of water from cement lined vessels that were cured in a number of different ways. The experiment was designed to determine the efficacy of 8 different curing regimes in the search to find a regime that would minimise the taste problem. The experiment was also used to look at some technical aspects of small cement jars, as well as the user aspects of small rainwater jars and their benefits to users.

The outcome of the taste experiment has been unsatisfactory. There were a number of problems with the experiment, many of which were outside the control of the people involved, but some due to poor experimental procedure. They included:

- Key staff leaving the organisation during the experiments
- Failure of equipment that had been taken to Uganda from the UK
- Curing water being taken from an unknown source
- Poor siting of tanks which meant that sampling was difficult on a daily basis

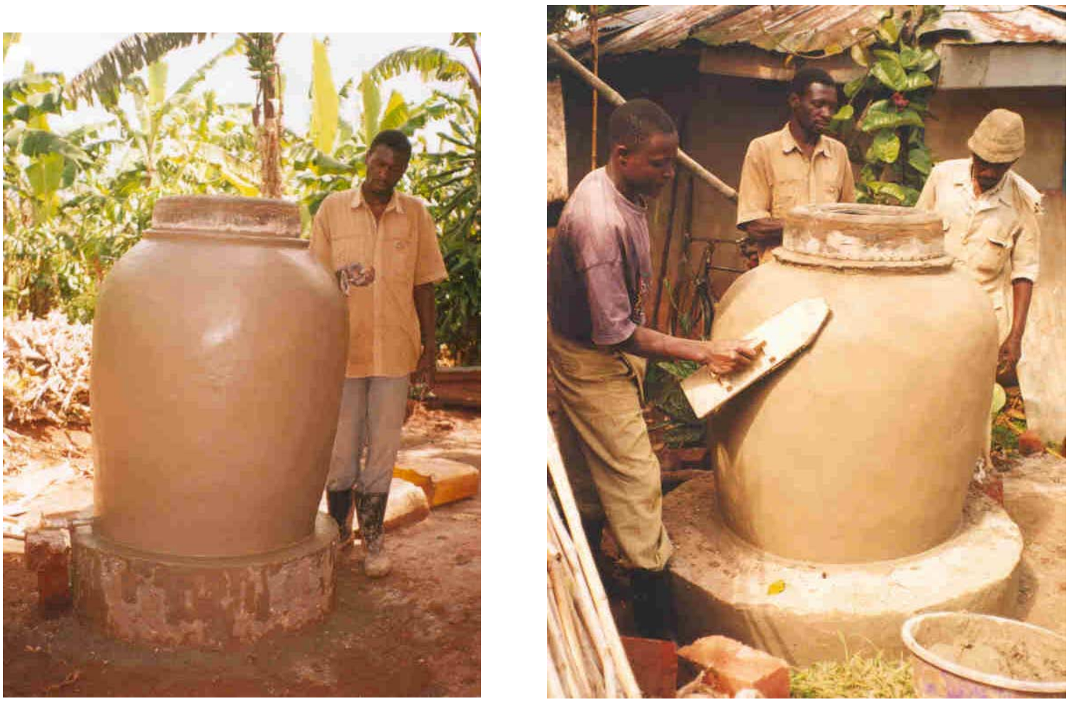


Figure 9 - 400 litre water storage jars being constructed as part of the water quality testing experiments in Uganda

A full report on the taste experiments will be presented later. The user studies are still underway. On a technical level, a number of findings have been made:

- Significant training is required to teach local masons the art of jar construction using this technique (the technique was taken directly from a classic ferrocement construction manual by S. B . Watt titled ‘Ferrocement Water Tanks’ [It Publications 1978]).
- Often the jars failed because the walls were of insufficient thickness – some method is needed of

5. Construction of two stabilised soil block tanks in Kampala, Uganda

In March 2000, two experimental cylindrical water tanks were built at Kawempwe, Kampala in collaboration with Dr Muses Musaazi, a lecturer at Makerere University. Both were built above ground of curved stabilised-soil blocks with end interlocking, 280mm x 140mm x 110mm high, made with an Approtec (Kenyan) manual block press. The soil used was a red somewhat pozzolanic local soil previously known to make strong blocks. The tanks were built on concrete plinths, lined with ‘waterproofed’ mortar (3 parts sand, 1 part cement and .02 parts ‘Leak Seal’ waterproofing compound). There was no metal reinforcing.

Figure 10 – Showing one of the stabilised soil block tanks under construction

Tank 1 is 2050mm high, has internal diameter 1300mm, wall thickness 140mm (+ 15mm render) and used 15 x 15 = 225 blocks incorporating 6% cement (100 blocks per 50kg bag). It has been filled with water and therefore has withstood a maximum head of 2.05m at the wall bottom. Volume = 2720 litres, max hoop stress = 0.19 MPa

Tank 2 is 1880mm high, has internal diameter 1000mm and the same wall thickness, but used 12 x 14 = 168 blocks with only 3% cement (180 blocks per 50 kg bag). It has been filled with water and therefore withstood a head of 1.88m at the wall bottom. Construction is continuing to extend its height up to 4m, testing its ability to resist pressure forces at both 3m and 4m head. Volume = 1476 litres, maximum hoop stress (so far) = 0.13 MPa.

Materials use included 1 packet (50kg costing $\$_{US}11$) of cement for the render, 1 packet for a conical (reinforced) lid, 1 packet for mortar between the blocks and ½ packet in the foundation. Thus only $^1/_5$ to ¼ of the cement is in the blocks themselves. Experiments to achieve curved blocks with *vertical* interlocking, if successful, will significantly reduce the quantity of mortar needed for block-laying. The lid may well be made more cheaply, as that employed was designed to carry certain testing devices.

Figure 11 – Showing the interlocking blocks used for the tanks

Report A3

Stabilised Soil Tanks for Rainwater Storage

Development Technology Unit

University of Warwick

Prepared by Mr. D.G. Rees

September 2000

This report is submitted as Milestone A5 under EU Contract ERB IC18 CT98 0276 : "Domestic Roofwater Harvesting in the Humid Tropics". That Milestone, under Task A – Technology, was originally to have been a report on the performance of underground tanks. Due to a re-sequencing of the research programme this Report A3 "Stabilised Soil Tanks for Rainwater Storage" has been substituted as the Milestone.

Contents

1. Introduction

The present cost of roofwater storage tanks is too high for many potential users. Warwick University, under an EU contract with three other partners, is investigating ways of reducing it within the specific context of the contract title above.

One cost-reduction strategy is to employ cheaper materials than hitherto. Soil ‘as dug’ is certainly cheaper than the metal, mortar or plastic commonly used for tank building. Stabilised soil *may* be cheaper provided not much stabilising additive is used. Although soil-based walling is widely used in housing, water tank walls pose special problems. They require of their materials two extra attributes, namely waterproofness and *tensile* strength that are only of minor importance in housing. Since soil is not impermeable and wet soil has no tensile strength, a process of material adaptation is required before soil can be recommended for tank construction.

Sections 2 to 4 of this paper review the various techniques of soil selection, stabilisation and construction. Section 4 addresses tank design using this material. Two promising technologies are identified, namely construction using stabilised soil blocks (SSB) and construction using stabilised or even unstabilised rammed earth (RE). Section 6 describes the theory, design, prototype manufacture and Ugandan field testing of SSB rainwater tanks. Section 7 covers the same sequence for RE tanks, but also includes the results of laboratory trials.

The paper finishes with conclusions and the identification of further work required to confirm the initial promise of SSB and RE construction. Four appendices cover cost comparisons, test data and detailed construction guidelines. (The guidelines are in the form of a free-standing Technical Release designed for use in mason training.)

Note on units: Both imperial (foot=0.3 m and inch=25 mm) and metric units are used in this report, reflecting the predominance of the former amongst builders in Equatorial countries and the greater ease of the latter for calculations.

2. Soil/earth building technology

Almost a third of the world's population lives in unbaked earth housing. The technology used varies from country to country and region to region, and sometimes even from house to house. A wide variety of earth construction technologies are known to exist and a few are listed below.

- Adobe – sun-baked earth bricks
- Wattle and daub – a wooden lattice with daubed earth in-fill
- Compressed earth blocks – using a ram (of which there are many designs)
- Direct shaping – hand shaped earth
- Cob – coarse fibre reinforced balls of earth stacked and compacted lightly
- Dug-out – dwellings excavated from earth
- Rammed earth – earth compacted between shutters with a tamper

Of these seven variants, only two appear at all suitable for constructing (above-ground) tanks, namely stabilised soil blocks and rammed earth.

Figure 2.1 Buildings (a clinic and a latrine under construction) made from stabilised soil blocks in Tanzania

All the technologies mentioned above are ancient techniques that have been passed on from generation to generation. Many have only lost favour within the last century with the advent of modern building materials, particularly brick, cement and steel. They are still used widely in many developing countries where cement is prohibitively expensive for the poorer sections of society (see the map in Figure 2.2). In some cases cement is used in small quantities to 'stabilise' the earth, giving extra strength and impermeability. Earth building technology is seeing something of a revival in the West amongst groups keen to maintain traditional techniques and those who appreciate the superior properties of earth as a building material e.g. its thermal, aesthetic, environmental and cost advantages. Improved techniques have been developed by architects and engineers over the years.

Stabilised soil block (SSB) technology that has received a great deal of attention over the last few decades and is now seen as a mature technology with a good future in the building industry world-wide. It is a technology particularly suited to drier climates, although it is practised in many humid areas. Suitable earth is mixed with a small percentage (typically 5 – 10%) of cement and is compacted using a manual or hydraulically assisted ram or press (Figure 2.3). The compaction process can be static (slow squeezing) or dynamic (impactive), but the static process is more common.

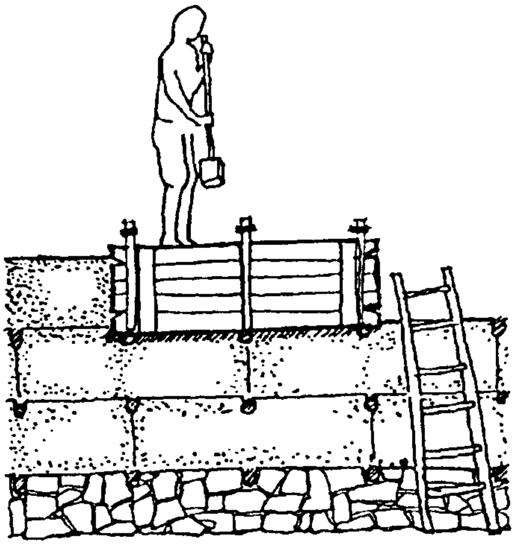
Static compaction pressure ranges from 2 MPa in manual lever machines up to 10MPa or more in machines with hydraulic assistance.

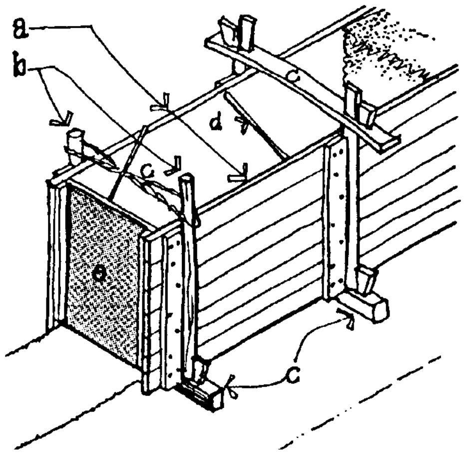
Figure 2.2 Map of the world showing areas where earth construction technologies are, or have been, widely used


Figure 2.3 A CinvaRam press being used to produce stabilised soil blocks in Africa

Rammed earth (RE) is a technique whereby earth is rammed, using a rammer or tamp, between two shutters. The shuttering is removed to reveal the wall, usually constructed in sections of a few feet long by a foot or two deep. The shuttering is then moved along and the next section of wall is rammed to form a continuous wall. The shuttering is then raised and placed on top of the first 'lift' to construct the subsequent 'lifts' (see Figure 2.4). Unlike SSB production, the RE wall is built in situ. Wall thickness for a typical two-storey house is in the region of 12 to 24 inches. Curved walls of rammed earth are not common, but are found occasionally where the technique is more developed. The curved sections are usually for decorative purposes.

Typically rammed earth has been used for the construction of housing. The technique has been used to successfully construct buildings of several stories that have lasted for centuries.





a = shuttering
b = retaining stakes

Figure 2.4 (i) rammed earth as practised in Morocco and (ii) basic elements of formwork or shuttering (from Norton[1997])

3. Soils - identification, classification and testing (field and laboratory methods)

3.1. Characteristics of soils

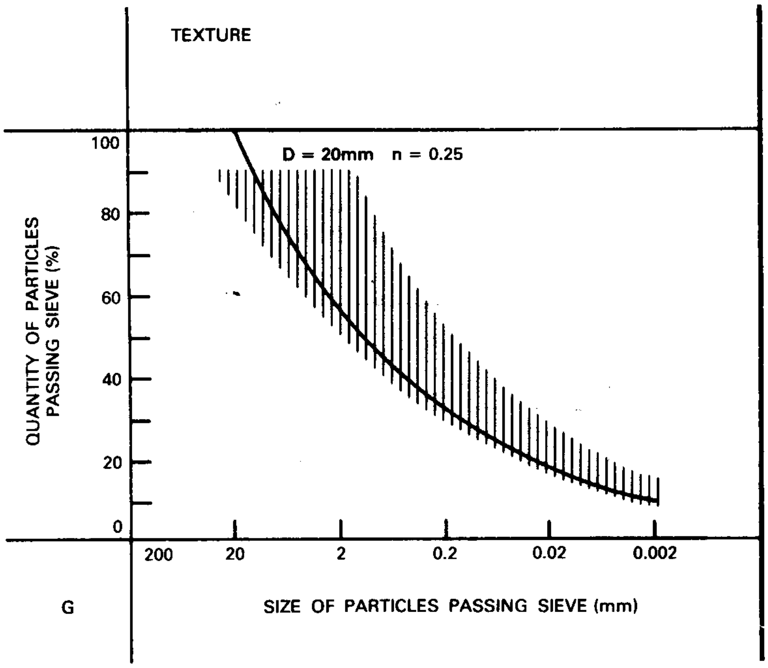
Not all soils are suitable for construction and methods have been developed for identifying those that are. For rammed earth (RE) construction a soil should be a mix of fine gravel, sand and silt with a small clay content. There should be no organic material present. Soil for stabilised soil block (SSB) construction needs to be of a higher fines content. However the actual soil used for either technique varies widely. Norton [1997] suggests the figures shown in Table 1 as suitable for rammed earth construction. The clay content should be sufficient to allow the soil to bind without causing excessive shrinkage. Soil varies widely in quality and content and so experimentation is required to find a suitable soil.

Table 3.1 – showing suitable values for soil particle distribution for rammed earth structures (Norton[1997])

Sand / fine gravel	45 – 75%
Fine sand / silt	15 – 30%
Clay	10 – 25%

Soil is generally characterised by 4 fundamental properties: texture, plasticity, compactibility and cohesion. These properties are described briefly below.

Determining the soil texture involves passing the material through a series of standard sieves and observing the fraction retained by each sieve, thus determining the grain size distribution. Further analysis is usually required to determine the fines content i.e. the make-up of the silt and clay passing through the finest practical (0.063 mm) sieve. Graph 3.1 shows the acceptable range for soil that is to be used for rammed earth structures. The ASTM-AFNOR standards and the decimal system standard for grain size distribution can be found on page 32 of Houben [1989].



Graph 3.1 Showing the acceptable particle distribution for a soil used for Rammed Earth construction (Houben [1989])

The Plasticity Index (PI) is an indicator of the plasticity of the soil. The PI is a function of the Liquid Limit (LL) and the Plastic Limit (PL) of the soil (together known as the Atterburg limits) and is a measure of the likelihood of the material to deform. LL is the % of water in a soil when it is changing from being ‘plastic’ to being ‘liquid’. PL is the water % at the boundary between solid and plastic behaviour. Numerically PI = LL – PL. There are agreed definitions of these transition points. Figure 3.1 shows on an Atterburg limits chart the type of stabiliser to be used with any particular soil.

The **compactability** of a soil defines its ability to be compacted to a maximum for a given compaction energy and degree of humidity {Houben 1994}. The compactability of a soil is measured by the Proctor compaction test (see Section 3.3).

Cohesion is a measure of the ability of a rammed soil to remain together when under tensile load. Cohesion is a function of the moisture content and the clay content (or other cementitious material) of the soil. Cohesion is higher when the moisture content is less than the PL. Cohesion increases with clay content, but so unfortunately does shrinkage.

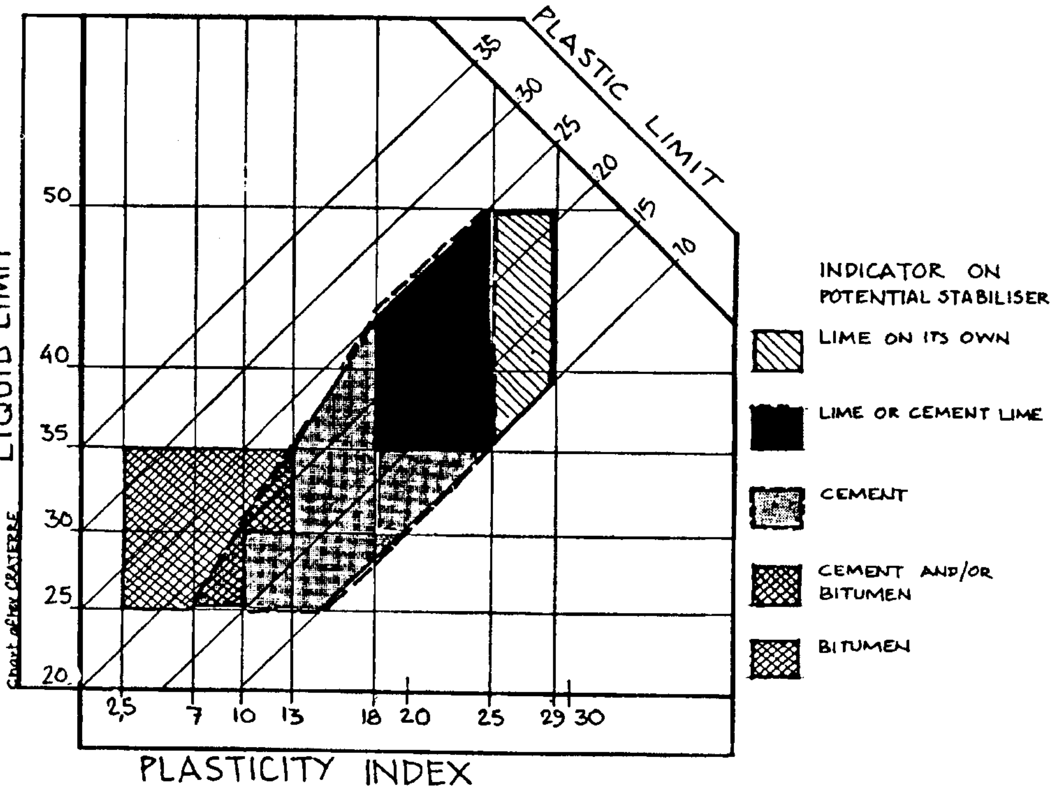


Figure 3.1 – The Atterburg limits chart (Norton[1997])

3.2. Soil suitable for rammed earth structures

A sieve analysis should indicate the ranges shown in Table 3.1 if a soil is to be used for earth construction.

Soil for use with rammed earth structures should be humid, i.e. not too dry and not too plastic, say in the region of 4% to 15% moisture content. The optimum moisture content (OMC) is defined later in this document and a method shown for determination of OMC.

3.3. Soil identification and classification

The suitability of a soil for building with is often established via three sorts of test – field tests, laboratory tests and construction trials.

Field tests

These are cheapest and come first. There are numerous initial sensory observations that can be made to help classify the soil in the field. These include:

- visual and tactile observation to analyse particle distribution
- sedimentation test to give more detailed particle distribution
- there are a number of tests to gauge (very roughly) clay content, including a simple test whereby a roll of clay is pushed over the edge of a table until it breaks - the length of the broken part gives an indication of the clay content
- drop test to determine optimum water content
- the smell of soil can sometimes give an indication of the presence of organic matter

More detail on these tests can be found in the relevant literature, especially Keable[1996], (Rammed Earth Structures – A Code of Practice).

Laboratory tests.
British Standard BS 1377 and corresponding standards in other countries cover the tests suitable for soils used in construction work. The tests are used to determine the main characteristics and suitability of the soil in question, as well as to give an initial idea of the performance of the material. The main tests that are used are described very briefly below:

A *'classification test'* is performed to determine the particle distribution of the soil. The test is carried out by passing the soil through a set of standard sieves (Figure 3.2). Material passing through the smallest sieve in the set is deemed to be a mix of clay and silt: these two components cannot be separated by further sieving.

Figure 3.2 – Wet sieving is the standard soil classification method

To determine the *'clay to silt ratio'*, the fraction of the soil passing through the last (0.063 mm) sieve is analysed using a hydrometer. The specific gravity of the liquid with its suspended particles, indicates its clay content. An alternative simpler test takes advantage of their different sedimentation rates to distinguish between clay and silt.

The compaction of a soil is dependent upon its moisture content. A *'compaction test'* is used to determine the maximum density of the material and the moisture content at which it occurs – called the optimum moisture content OMC. At maximum density the compressive strength of the soil will be greatest. To find the OMC a test is carried out using a simple compaction apparatus (see Proctor compaction test in Box 3.1) to compact a number of samples with different moisture contents. The samples are weighed and measured. The dry weight is then found after drying the compacted sample in an oven and the OMC is that moisture content which has produced the sample of greatest density.

Box 3.1 BS Ordinary Test (or the Proctor test) for compaction

This test uses a 2.5kg metal rammer with a 50mm diameter face that falls into a cylindrical mould of 105mm diameter. The drop height is kept at a constant 300mm to ensure consistent energy transfer between blows. The blows follow a pattern over the face of the sample to ensure repeatability and consistent compaction of the entire sample. Each sample made up of three layers of soil that has passed through a 20mm sieve and each layer is given 27 blows of the rammer. After compaction the sample is trimmed off to a set height that gives a constant volume of 1000cm³. This is then weighed and the density can be calculated.

In the previous section we touched briefly on the consequences of shrinkage. The *'shrinkage box test'* measures this property and is simple to carry out. It involves measuring the shrinkage of a sample that is allowed to dry naturally over a period of 14 days. In practice, if the walls of a tank are constrained, say at the base, then cracking will take place if shrinkage is significant. Conversely, where a tank's cylindrical wall is free to shrink without constraint no cracking will take place. If shrinkage is found to be too great, due to an excess of clay, the soil will have to be either modified or rejected.

'Liquid and plastic limit tests' can be carried out either in the field, although it is preferably to perform them in a laboratory. Some equipment is needed. See section 3.1 for more detail.
A *'normal moisture content test'* is used to determine the normal moisture content of the material to be used. An oven and accurate scales is required.
To measure the rate at which water passes through a material requires a *'permeability test'*. Sophisticated equipment is required.
For many applications of rammed earth walls (i.e. housing and larger buildings), it is the compressive strength that should be maximised, so *'compressive strength tests'* are commonly applied to the material. *Wet* compressive strength, which is invariably less than *dry* compressive strength, is most commonly measured. Unfortunately little consideration is given to the *tensile* strength of the material which concerns us more in water tank design. Confusingly we have four possible strength measures: *dry compressive*, *wet compressive*, *dry tensile* and *wet tensile*.

Although Houben and Guillaud [1989] indicate reasonably good values for dry tensile strength of rammed earth (0.5 –1 MPa), little work has been directed at further increasing its value. For the application being considered here, the tensile strength of the material is of paramount importance. The OMC considered earlier is the moisture content that will optimise dry density (we could call this the DOMC , after Montgomery [1999]), and hence compressive strength. It is unclear to the author if this DOMC can be used to optimise dry tensile strength. Further investigation is required to find the water content to maximise dry tensile strength, which we could call the Tensile OMC (or TOMC). We then need to consider the wet tensile strength of the materials as this has great implications on the design of tanks to hold water.

Performance tests.
Even where we have applied field and laboratory tests to the raw material (soil) or to specimens prepared from it, it is still desirable to test samples of the walling during construction. In this way we can pick up not only soil variations but also imperfections in the construction process (such a deviations from intended proportions or procedures). These performance tests are usually applied either to samples drawn from batches in the construction process (e.g. every 50th block made) or to specimens sawn out of larger components.

Some of the tests that may be applied are:

- Wet and dry compressive strength
- Wet and dry tensile strength (modulus of rupture)
- Wet and dry bending strength
- Permeability
- Adhesion of render to walls

More details is available in the literature – see Houben

3.4. Calculating the quantity of soil required for a tank

The quantity of soil required for tank construction is calculated below. The quantity of soil required for tank construction is based on tank size, wall thickness and the density of the compacted material.

Weight of soil required $W = \pi (r_o^2 - r_i^2) H \times \rho_c$ (equation 2.1)

where
r_o = external radius of tank
r_I = internal radius of tank
H = height of tank
ρ_c = density of compacted material

For laboratory and development tests approximately 150 kg of soil is required.

4. Stabilisation of soils – methods for improving soil characteristics

Frequently one finds that the most readily available soil is not suitable for construction purposes. In such cases the main options are to

- bring in a suitable soil from elsewhere
- blend together different local soils
- add some sort of stabiliser.

Which option is chosen will usually depend upon their relative costs. However if wet strength is required, no natural soil is adequate and stabilisation (e.g. addition of cement) is essential. The physical characteristics of soil can be improved in a number of ways. Usually, the main reason to improve soil is either to obtain a suitable physical grading for a poor soil or to improve some other physical characteristic such as the strength, stability or water resistance of the soil.

Soil suitable for soil construction should be well graded with a suitable content of fines and larger particles (see Table 3.1). For raw soil, this is often not the case and soils have to be modified such that their grading is suitable for use. This will involve adding a material that is lacking in the raw material, removing unwanted particles by sieving or mixing a number of soils to obtain a suitable blend.

4.1. Additives and composite materials

A large number of additives and composites are available for adding to rammed earth. Their purpose is improve the properties of the material in one of a number of ways: *Chemical stabilisers* are added to the soil to bind or alter the grain distribution characteristics of the soil and hence improve its cohesion and stability. Common stabilisers include cement and lime, which are added in small quantities, say up to 10%, and can improve material strength and stability several fold. There is a wide range of synthetic additives available from specialist suppliers in some countries (Texas, USA is a good example, where earth building is a commonly practised building technique). They are not, however, generally available in developing countries and so we will not consider them in this work.

Waterproofing agents are available that reduce the permeability of soil structures. A commonly used water repellent is bitumen, which is mixed with the soil in an emulsion form. The emulsion is made using a solvent such as gasoline or kerosene diluted sufficiently to be mixed with the soil (Norton[1997]). Many synthetic waterproofing additives are available. Most earth walls, bearing in mind that such walls are used for buildings, are given a waterproof render, or a sacrificial coating. Renders and linings will be discussed later in the document.

Fibres can be added to increase the tensile strength of the material and help prevent cracking during the curing process. Straw is a common fibre additive and it also helps to reduce the weight of the material. Unfortunately it is only durable in permanently dry conditions.

Reinforcement is widely used to create a composite in which the matrix (e.g. soil) provides some properties and the reinforcement (e.g. steel wire or polypropylene or hessian rope) provides much of the tensile strength.

Box 4.1 Discussion of the effects of soil shrinkage on tank design, and design implications.

Upon drying a rammed earth wall will shrink as the moisture is drawn out of the structure and the clay, which is expansive (different clays having differing degrees of expansivity), shrinks. If the structure is constrained in any way, say for example, at the base of the tank, then this could result in cracking. For an unreinforced tank such cracking would seriously reduce tensile strength and could cause sudden and catastrophic failure of the structure. This needs to be considered when determining the OMC (or TOMC) and the author suggests that this area of rammed earth tank design needs further work at present. It is for this reason that some form of reinforcing could be used to give additional tensile (hoop) strength to the structure. Possible forms of reinforcement include:

- Externally applied steel packaging strap (as demonstrated by the author {Rees, 1999} on single skin brick tanks)
- Hoop wire rammed into the structure – ideally barbed wire would be used as it is cheap, strong and the barbs offer resistance to 'pull-through'.
- Fibre such as straw or short lengths of polypropylene rope can offer localised strengthening

Box 4.2 Stabilisation of soils (Montgomery[1999])

Stabilisation techniques can be broken down into three categories, Houben (1994): Mechanical, Physical and Chemical. Mechanical stabilisation compacts the soil, changing its density, mechanical strength, compressibility, permeability and porosity. Physical stabilisation changes the properties of the soil by acting on its texture, this can be done by: controlling the mixture of different grain fractions, heat treatment, drying or freezing and electrical treatment. Chemical stabilisation changes the properties of the soil by adding other materials or chemicals. This happens either by a physico-chemical reaction between the grains and the materials or added product, or by creating a matrix which binds or coats the grains.

Stabilisation fulfils a number of objectives that are necessary to achieve a lasting structure from locally available soil. Some of these are: better mechanical characteristics (leading to better wet and dry compressive strength), better cohesion between particles (reducing porosity which reduces changes in volume due to moisture fluctuations), and improved resistance to wind and rain erosion. Using one or more of the stabilisation techniques listed above, many of these objectives may be fulfilled. Optimum methods depend greatly on the type of soil, and a careful study of the local soil is necessary to suggest an effective method of stabilisation. In the case of mechanical stabilisation, the soil is compacted to a greater density, and there will always be an improvement in its mechanical properties with virtually any soil type. This is not true however with other forms of stabilisation, where different soil mixtures can lead to better or worse properties using the same technique. In the majority of cases mechanical stabilisation is used in conjunction with a common chemical stabiliser, such as cement. If the stabiliser and the soil are mixed together thoroughly and there is a suitable clay fraction in the soil, the compaction process reduces the quantity of chemical stabiliser required in the block. The increased density also increases the effectiveness of the cement matrix, given that the cement is left in a moist environment (the hydration period to let the cement cure) for at least 7-14 days. For details on selection of soils for cement stabilisation see Gooding (1993 - B).More details on the process of cement stabilisation can be found in Houben & Guillaud (1994) and Spence (1983).
(Source: Montgomery, David, http//:www.eng.warwick.ac.uk/DTU/buildingmaterials/index.html, 1999)

5. Tank design using stabilised soils

It is certainly inconvenient, and it can be dangerous, for a water tank to fail (because at some point in it the local stresses exceed that which the tank material can bear). We therefore need to be able to calculate the size and location of the peak stresses. Unlike other areas of structural design, stretching does not need to worry us much unless it is severe enough to cause the cracking of a superficial waterproof coat.

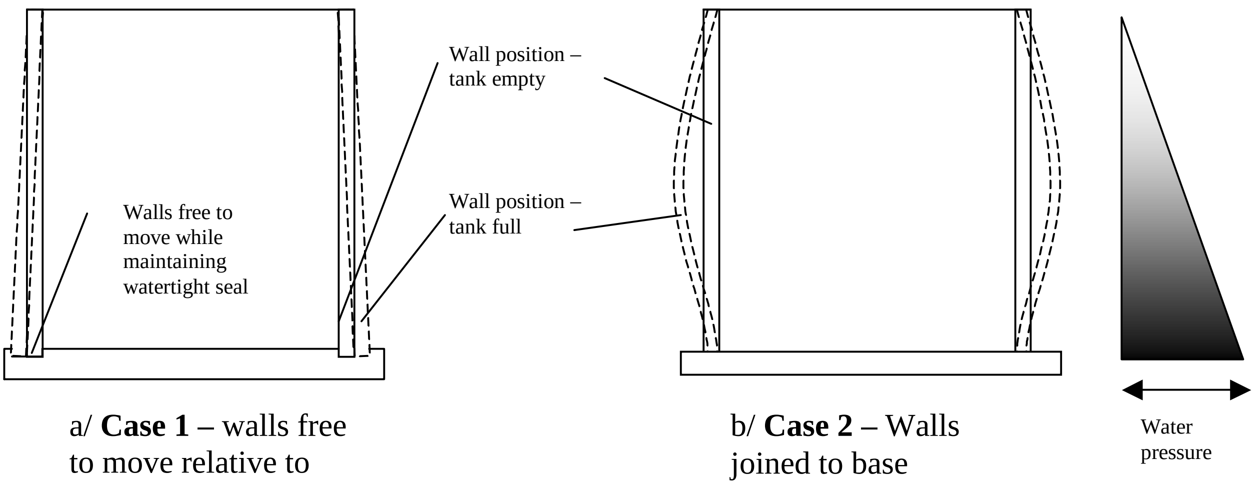
Our general design objective is to produce a safe watertight tank. This we can do either by using a single walling material that is both strong (in tension) and waterproof or by combining a strong material with a waterproof one. A wall of unstabilised soil is not waterproof and (when wet) doesn't have any tensile strength either. We therefore have either to stabilise it or to keep it dry by applying a waterproof skin to its inside face. In the latter case we must allow for the possibility that the skin will be punctured during the life of the tank – including some sort of reinforcing that may not save the tank but will prevent it bursting suddenly and dangerously.

5.1. The theory of stresses in cylindrical tanks

Tall cylindrical tank walls experience a horizontal tensile 'hoop stress' which is proportional to the diameter *D* of the tank, the local pressure *p* on the walls of the tank and the thickness of the tank wall *t* (Equation 5.1). As *p* increases towards the bottom of the walls, so too does the hoop stress.

$$\sigma_t = \frac{pD}{2t} \qquad \text{(Equation 5.1)}$$

However the stress in a cylindrical tank wall near to its base is also be affected by the type of joint between the tank base and tank wall. There are two obvious cases to consider as illustrated in Figure 5.1 below. In Case 1, the tank wall is made so that is free to move slightly at its base and yet still maintain a watertight seal. The water pressure against the wall will cause the diameter of the tank to increase until the hoop stress is wholly taken up by tensile stretching of the wall. (In the figure the increase in diameter is exaggerated here for clarity: in practice it might be less than 1 mm). The maximum hoop stress will be experienced at the base of the wall and will decrease linearly with height to zero at the top of the wall.



a/ **Case 1** – walls free to move relative to base

b/ **Case 2** – Walls joined to base

Figure 5.1 – The two cases for wall-to-base union in cylindrical tanks

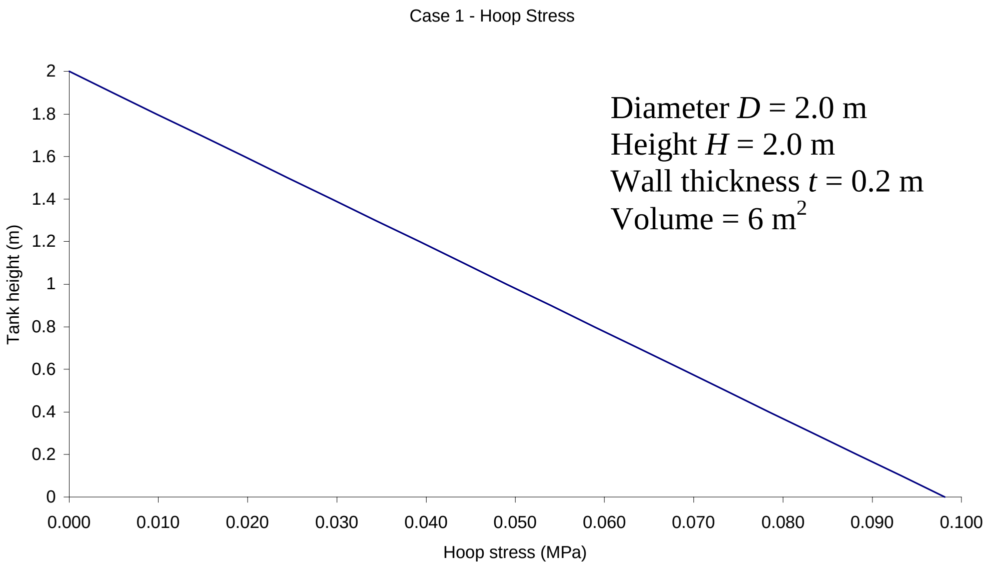


Figure 5.2 – Hoop stress in the tank wall for Case 1 (i.e. wall and base separate)

In Case 2, where the wall and base are monolithic i.e. the wall and base are continuous (and the base is assumed to be rigid), the situation becomes more complex. Bending and shear stresses are set up in the wall as a result of the restraining effect of the base slab. There now exists a combination of bending, shear and hoop stresses. The lower part of the wall is now constrained and is not free to move as in Case 1. The result is that a bending moment is generated in the wall that is a maximum at the joint of the wall and the base. This bending moment sets up vertical tensile stresses on the inner face of the wall of the tank that can be more than twice the magnitude of the horizontal hoop forces. Shear forces are also generated although these can generally be neglected as they are small in comparison the other induced forces. This is illustrated graphically in Figure 5.2

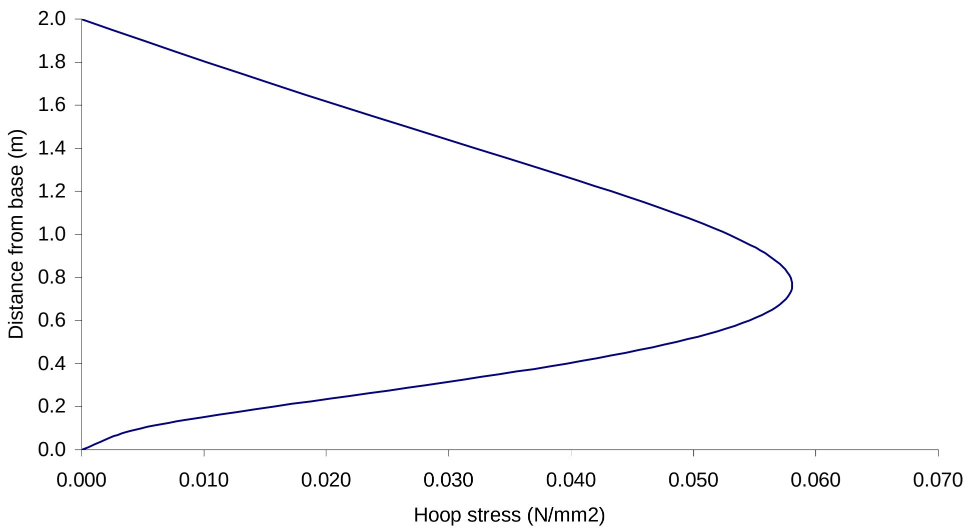


Figure 5.3 – Hoop stress for Case 2 (i.e. wall and base monolithic)
(*D*, *H* and *t* as in Figure 5.2)

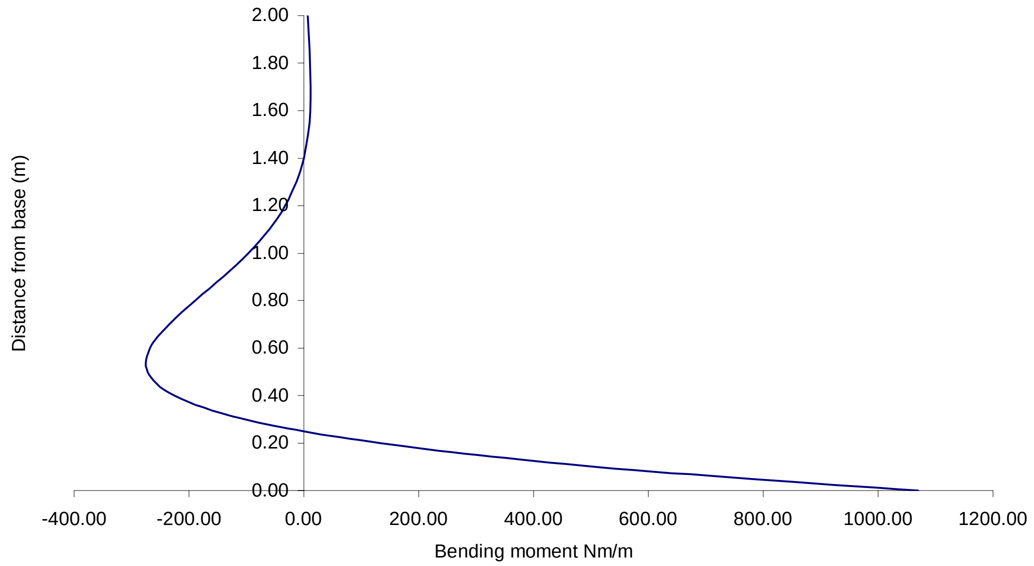


Figure 5.4 – Bending moment induced in tank wall for Case 2 (*D*, *H* and *t* as in Figure 5.2)

In Figure 5.2 we see that for Case 1 the hoop stress increases linearly from zero at the top of the tank wall to a maximum of 0.098 MPa at the base/ wall joint

In Figure 5.3 we see that for that Case 2 the hoop stress is a maximum of 0.058 MPa – less than in Case 1 – and occurs at 0.8m from the base (i.e. 0.4*H*). The hoop stress then drops off to zero at the base where the strain is zero. In Figure 5.4 we see the effect of the rigid constraint on the bending moment that is now induced in the wall. The maximum vertical tensile stress in the bottom of the inner face of the tank wall in Case 2 is 0.16 MPa, considerably more than the maximum hoop stress in either Case 1 or Case 2. So the tank is now vulnerable to cracking at the wall-base joint.

5.2. Conclusions from Section 5.1 and implied design considerations when using soil as a building material

Note:
Case 1 - wall is free to move at base (flexible joint)
Case 2 - wall and base are continuous (monolithic joint)
(assuming rigid base in both cases)

Table 5.1 Summary of conclusions from Section 5.1 and resulting design implications

Conclusions from Section 5.1	Design Implication
In Case 2 a bending moment is set up in the wall due to the constraint at the base of the tank. This is a maximum at the joint of the wall and the base and causes vertical stresses whose size is more than twice the maximum hoop stress. No bending moment is induced in Case 1.	Sliding base / wall joint are preferable to avoid complex stress regimes in the tank walls (see Box 4.1 below). These are difficult to achieve in practice.

The vertical stress cause by bending is very sensitive to changes in wall thickness as it is proportional to $1/t^{2}$; thus halving wall thickness will multiply stress by four. The hoop stress is only proportional to $1/t$.

Where a monolithic base / wall joint is used it is wise to use a good safety factor when considering wall thickness or to thicken the wall near the joint.

The maximum hoop stress for Case 2 is always less than that of Case 1 (for an identical tank profile) and is experienced at some point above the joint of the base and wall, typically 0.3H to 0.6H, but lower for very thin walled tanks.

Do not design the wall thickness to decrease with height in the case of monolithic tanks.

For a tank of identical profile, the maximum bending moment set up in Case 2 will be of greater magnitude than the hoop stress set up in Case 1 or Case 2 (see Figure 5.5).

A greater wall thickness is required for Case 2 than for Case 1. This implies more material usage and hence higher cost. On this point, it is worth bearing in mind that many materials will have a greater tensile strength than bending strength and so the problem will be compounded.

Deflection is greater for Case 1, as there is no constraint at the base to restrict movement of the wall.

Strengthening at the base can help reduce deflection for Case 1. This is important where renders are used to help prevent cracking due to excessive strain.

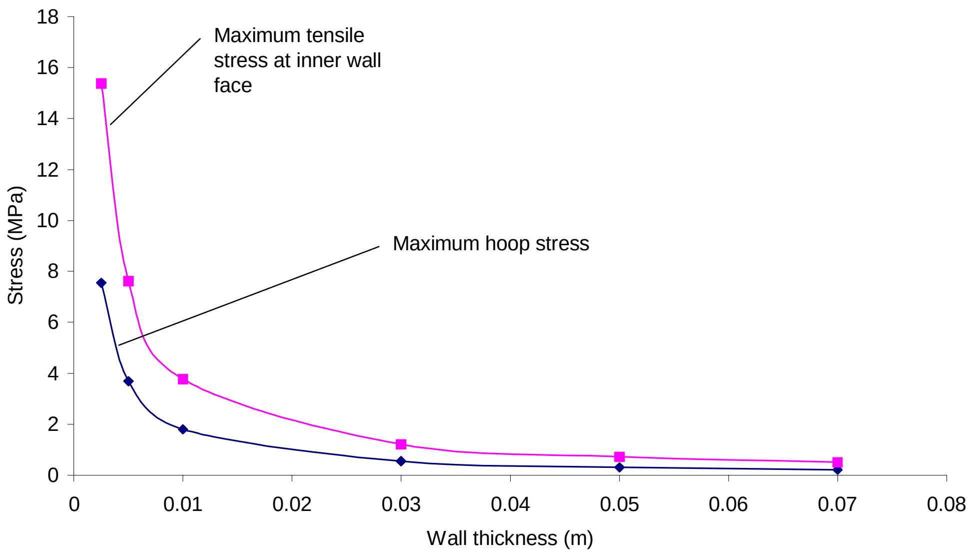


Figure 5.5 – The graph shows the relationship between hoop stress, maximum tensile stress due to bending and wall thickness. (*H* and *D* as for Figure 5.2) Note the significantly higher values for bending-induced tensile stress.

Box 5.1 Sliding / flexible base wall joints

Sliding of flexible joints between the wall and the base of a tank are common for large concrete storage. The reduction in stresses by using such a joint is well documented and the subsequent savings in materials well recognised. Such joints, in concrete structures, are usually effected using a bitumen layer between the base and the wall. The degree of movement in the wall is very small and the flexibility of the bitumen allows sufficient movement while maintaining impermeability.
The technique developed at the University of Warwick for work with soil tanks has been to lay two layers of polythene sheet beneath the wall of the tank. The sheets can have a smear of grease to help them slide across on another. The effect is to produce a 'sliding' joint that allows the wall and the base to act independently which, as pointed out in earlier, reduces stresses, simplifies tank design and leads to lower costs.

5.3. Spreadsheet design for rammed earth tanks

A spreadsheet has been developed at Warwick (Turner 1999) for aiding the design of cylindrical tanks. The spreadsheet allows the main variables to be entered and gives output in terms of design parameters. The main variables are tank diameter, tank height, material properties (or relative volumes in the case of composites), required safety factor and wall thickness. The outputs are given for both a tank with monolithic base and wall, and for a tank with separate base and wall. The outputs include maximum hoop stress, maximum bending moment, maximum deflection, maximum tensile stress on inside face of tank wall, maximum shear stress, tank volume and overall tank diameter. The spreadsheet allows the designer to play with the parameters until a satisfactory design solution is reached. Table 5.2 shows the general layout of the spreadsheet. The spreadsheet was used to design both the laboratory experimental tank and the field experimental tanks described and discussed in Section 7.

Table 5.2 Showing the input and output cells of the spreadsheet developed at Warwick to aid design of cylindrical tanks.

Data input section - click on boxes for data input instructions			
Parameter		**Value**	**Unit**
E (matrix)	e.g. soil	700	MPa
E (fibre)	e.g. steel	2.1E+5	MPa
po(matrix)		0.5	Poisson's ratio
po(fibre)		0.1	Poisson's ratio
thickness		0.20	m
tank rad		1	m
height		2	m
Vol. Fraction of fibre		0	0 – 1
po(total)		0.5	
E(total)		700	MPa
Volume		6.283	cubic metres

Output cells –for a Rammed Earth tank The output here is for a tank monolithic with base		
Tensile strength (of soil)	0.50	MPa
Safety factor required	4.00	
So design stress is	0.1250	MPa
Maximum hoop stress generated	0.0579	MPa
Position of max hoop force		m from base
Maximum bending moment	1069	Nm (always at base)
Maximum deflection	0.0033	mm
Position of maximum deflection		m from base
Maximum tensile stress on inner face of wall	0.16037	MPa (always at base)
Maximum shear stress	0.00021	MPa
Overall tank diameter	2.4	m

Output here is for tank wall unconstrained at base		
Tensile strength	0.50	MPa
Safety factor required	4.00	
Design stress	0.1250	MPa
Maximum hoop stress generated	0.098	MPa
Maximum deflection	0.1401	mm
No bending moment generated		

5.4. Construction principles when building with earth

There are several basic rules to follow when building with earth:

- The wall should be well protected from damp or wetness. Wet earth has less strength than dry and unstabilised earth will quickly become a mess of mud should it become saturated.
- Good foundations are used to protect the base of the wall from rising damp and often the first foot or two of wall above the foundation will be from stone or other impermeable material.
- Where a roof is fitted, the wall can be protected by using large overhanging eaves to prevent rain from hitting the wall directly.
- Renders or other coatings help to protect the wall from rain also. Practitioners colloquially use the phrase “good hat and boots” to describe the protection required – an overcoat doesn’t go amiss either!

The benefits of using earth as the construction material are numerous:

- low material cost (see cost comparison with ferrocement in Section 8)
- suitable material readily accessible locally in many parts of the world
- a well-known and widely-used technology in many parts of the world
- a simple technology that is easily taught to semi-skilled people

The drawbacks of using earth for tank construction are:

- not suitable for below-ground tanks or cisterns
- in the case of leaks serious problems can develop, especially if unstabilised earth is used
- high labour input – a problem where labour costs are higher

6. Stabilised soil block (SSB) tanks

6.1. Introduction

The work carried out on Stabilised Soil Block (SSB) tanks has been done in conjunction with Dr Moses Musaazi, a lecturer at Makerere University, Kampala and private entrepreneur working in the construction industry. Dr Musaazi also directs the Gatsby Trust in Uganda whose aim is to promote small-scale private enterprise through training. He has been working with SSB's for some time, both for low-cost housing and also for the construction of small too medium sized domestic rainwater storage tanks. Warwick University approached Mr Musaazi with the aim of working together to test the strength of the SSB tanks through practical experimentation.

A full report of the work that has been completed by Dr Musaazi is found in Appendix II.

6.2. The basic principles of SSB manufacture and construction

Stabilised, compacted, soil block technology is mature and widely used throughout the world. It involves compacting a suitable soil, which is often mixed with a small percentage (typically 5 – 10%) of cement, using a manual or hydraulically assisted ram or press. The compaction process can be static or dynamic, but the static process is most common. Static compaction pressure ranges from 2MPa (manual) up to 10MPa or more (hydraulic). This compaction reduces the voids in the material and hence it susceptibility to attack from water. Figure 2.3 shows a CinvaRam press being used on a building programme in Tanzania. Figure 6.1 shows some common press types.

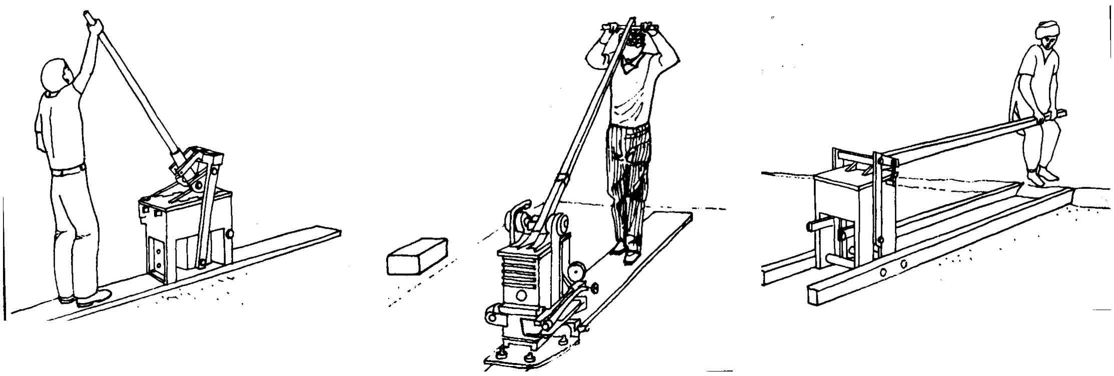


Figure 6.1 – A small selection of the many press types available for purchase (Houben[1994])

Special moulds can be manufactured to produce blocks required for special purposes. In the case of the cylindrical tanks manufactured in Uganda, curved blocks were produced using the mould shown in Figure 6.2.


(**Figure 6.2** – Mould for producing curved blocks)


(**Figure 6.3** – Curved blocks after curing)

For information about tank linings and water extraction for SSB tanks see Section 7.2. These principles are the same for RE and SSB tanks.

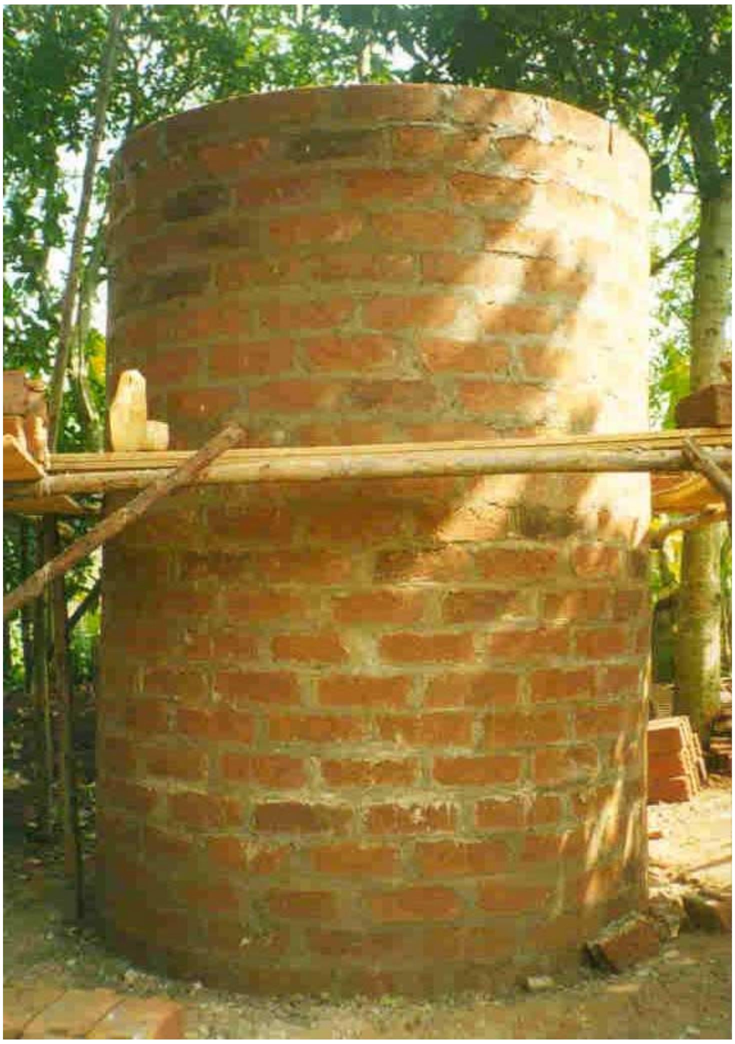
6.3. Field testing of SSB tanks in Uganda

Tests on SSB tanks started in March 2000 in Kampala. A small cylindrical tank (1.0m internal diameter, 2.1m height) was built and an attempt made to fit sealed covers to it. The tank was to have been pressurised by means of a header pipe fitted to the cover, however this proved impractical so it was simply subjected to the pressure (21 kPa) derived from its own height. The materials used are specified in the report in Appendix II. The soil used was stabilised with 5.25% OPC.

The experimental method was changed for subsequent tests and two tanks were built of the same curved end-interlocked SSBs. No reinforcing was included, but the tanks were rendered inside with a waterproof mortar. Details are:

Tank diameter (internal) in m	Tank Height in m	Code	% cement
1.67	3.8	SST1	5.25
1.00	5.2	SST2	3.27

SST1 was tested in July 2000 and failed dramatically when only part full. SST2 is still under test.



(**Figure 6.4** – A SSB tank of 1.67m diameter)

SST1, according to theory and using the *dry* tensile strength for the blocks from the literature, should have been able to withstand about 5 times the stress induced by the water pressure at which it burst (at 2.5m water pressure). There were, however, some irregularities:

- Some of the blocks analysed after the experiment seemed to have no wet strength and disintegrated completely in water.
- The tank was filled from a large water tanker. The waterproofing agent that was used in the cement render lining takes time to act and so water would have been passing through the tank lining and into the soil matrix. This would have reduced the strength to somewhere below the dry strength quoted in the literature.

SST2 by contrast was filled slowly by rainwater from a gutter and has sustained a pressure, 53 kPa corresponding to its full height, for many weeks despite being built with considerably less cement in the blocks than SST1. The peak stress in SST2 is 50% greater than that at which SST1 failed.

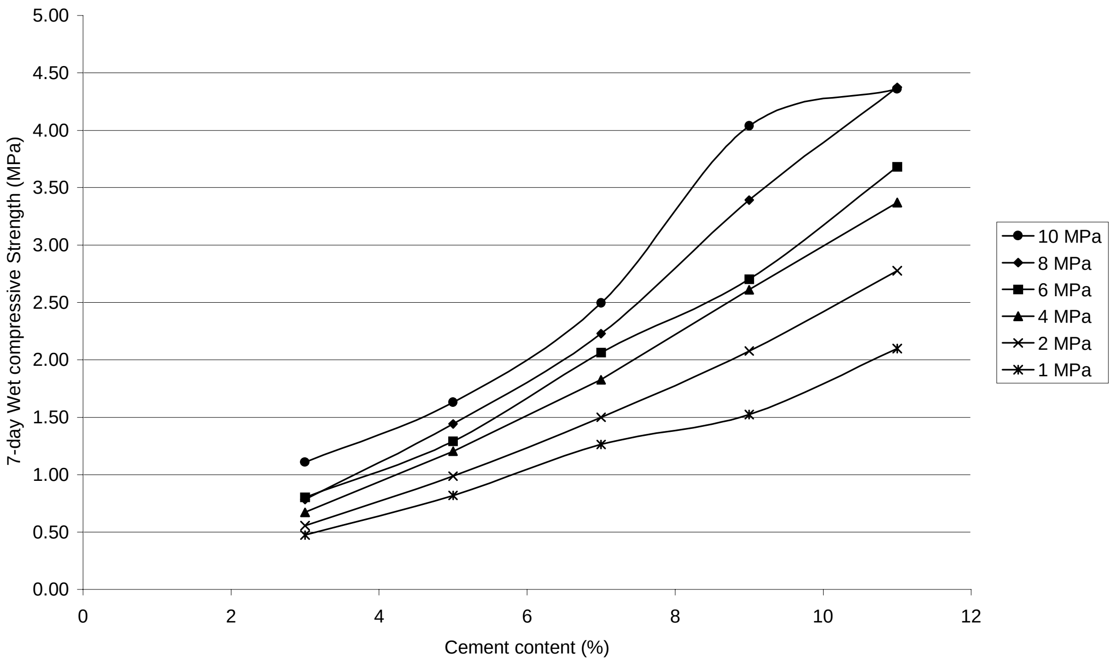
A further complication in the analysis of any tank made of blocks or bricks is uncertainty about how tensile forces travel from one block to the next. One route is via the mortar that separates blocks of the same course. Mortar does not have very high adhesion to blocks, although in this case the loose interlock between block ends would have provided some shear connection. The other route is via shear forces into, and then back out of, the overlapping blocks in the courses above and below the course of interest.

Three further but smaller SSB tanks have been built and sold.

6.4. Conclusions concerning SSB tanks

It is unclear exactly what was the failure mode of SST1. It appears that the lining was not sufficiently waterproof to prevent water passing to the stabilised soil matrix, causing the soil to rely solely on its wet strength. The maximum stress induced in the wall of the tank at 2.5m water pressure is 0.234 MPa. This is the maximum resultant tensile stress on the inner face of the tank wall due to both bending and hoop stresses (as calculated using spreadsheet by Turner [1999]).

If we look at Graph 6.1 we see that the 7-day wet compressive strength for the material (which has a 5.25% cement content and was compacted at about 2MPa, is about 1MPa. Similar figures are found in Houben for 28 day wet compressive strength – see Table 6.1. The usual rule of thumb for dry to wet strength ratio for soil is 5:1. So we see that the wet strength of the soil blocks would be in the region of 0.2 MPa, which is lower than the induced stress prior to failure. It could therefore be said that the tank performed better than might be expected.



Graph 6.1 Graph showing variation of 7-day wet compressive strength with cement content for a number of compaction pressures (Gooding [1993]).
Cement content = 5.25% in our case.

So for this diameter tank (1.67m), using the construction method outlined in Appendix V, and with the standard 2.2m height, the dry strength would give a safety factor of approximately 5. However, if the tank lining should fail, or if the tank should become saturated for any other reason, then the tank would be on the border of failure as the wet strength (~0.2MPa) is approximately equal to the induced stress (0.2026MPa).

Table 6.1 Compressive and tensile strengths for SSB's. All the above assume 2MPa compaction pressure an assume tensile strength to 0.2 x compressive strength

Source	Compressive strength	Estimated tensile strength
Gooding (see Graph 6.1)	1MPa (7 day wet)	0.2MPa
Houben	1 – 2MPa (28 day wet)	0.2 – 0.4MPa (estimated by author)
Houben	5 – 12 MPa (28 day dry)	1.0 – 2.4MPa (estimated by author)

The analysis above would hold if the subsequent tests on the SSB's from the tank debris had not shown that some of the blocks had NO wet strength. The only explanation for this is that these blocks failed before their minimum strength was reached.

6.5. Further work with SSB tanks

Further suggested work on this area of work includes the following:
full laboratory analysis of the soil (the soil used in Kampala was somewhat pozzolanic and was assumed to be superior to standard lateritic soil there)

- development tests to determine the soil / block performance (however there is some suspicion that the blocks in SST1 had been starved of their proper cement allocation)
- further full-scale tank tests with some modifications to improve tank performance
- investigate methods for improving tensile strength e.g. reinforcing with barbed wire
- further work to investigate the relationship between wet and dry tensile strength of stabilised soils

7. Rammed earth (RE) tanks

7.1. Introduction to RE tank development

The general aims of the work carried out on rammed earth tanks (RE) are listed below:

- to set out the theory of RE tank design and to investigate the options available for RE construction
- to develop the specification for an experimental laboratory tank and two experimental field tanks
- to develop the skills required (within the research team) to analyse soils for building, in both the laboratory and in the field
- to investigate soil modification techniques and methods to develop a soil suitable for tank construction
- to develop the tools and equipment required for RE tank construction

- to develop a technique suitable for rammed earth tank construction
- to develop a technique for tank lining using plastic sheet
- to test a rammed earth tank under laboratory conditions to verify the theory
- to build a number of tanks in the field to test the feasibility of the technology and its suitability to LDC skills
- to test a tank under field conditions, again to verify the theory
- to carry out a cost analysis of a RE tank to allow comparison with other tank types
- to develop guidelines for the manufacture of RE tanks

Three (3) RE tanks have, to date, been designed and constructed:

- A tank of 1.4m diameter and 1m height was built at the University (RE-UK*). The tank was from unstabilised soil. The aim was to test the general principle of construction and to allow for initial tests to be carried out on earth tanks. The tank was completed in May 2000 and simple tests carried out, but full tests are yet be to be carried out on this tank. The simple tests include; fitting experimental linings, applying steel strapping and filling the tank a number of times to test strength. The full tests are to include a pressure test to ascertain the actual strength of the tank and to allow comparison with the theory.
- Two tanks of stabilised soil of 2m diameter and 2m height were constructed at Kyera Farm, Mbarara in June and July 2000. The first (RE1*) was constructed to a high specification (see Section 6.4.2) with concrete base and masonry wall section to 0.35m high. The second (RE2*) was designed to be very low cost using predominantly stabilised earth. Both were stabilised with 4% cement and reinforced with barbed wire hoops at 75mm intervals (see Appendix IV for Rammed Earth Tank Construction Guidelines)

** Note:*
For easy identification of the three tanks, they have been labelled as follows

RE-UK	- UK built tank
RE1	- Uganda high specification tank
RE2	- Uganda low specification tank

7.2. The basic principles of rammed earth construction

The general principles for building with earth are outlined in Section 5.4. and a full account of the rammed earth construction process used for the experimental tanks in Uganda is given in Appendix IV.

Some other points regarding RE construction are listed below:

- Availability of suitable materials is a key factor. If significant work is required in modifying the soil once it has been excavated, then the process can quickly become too costly.
- Wall thickness tends to be high, compared with modern materials. This is due to the relatively low compressive strength of the material and the variability in material quality. Compressive strength can be improved by using a small percentage (say 5% – 8% by weight) of cement mixed with the earth.

- Tensile strength is fairly low – in the region of 0.5 – 2.0 Mpa. The tensile strength can be improved by adding fibre or composite materials as well as by stabilisation with cement.
- The technique is characterised by high labour input and low material costs. This is well suited to many developing countries where labour is cheap and manufactured materials are costly.

The tools required for rammed earth construction can be few and of relatively low cost. In the West, sophisticated tooling has been developed, and sometimes costly pneumatic rams are used with steel shuttering, but in the less developed countries (LDCs) the tools have remained unchanged for centuries. Many designs of wood shuttering have been developed to meet the needs of the builder, but the principle actually varies little world-wide.

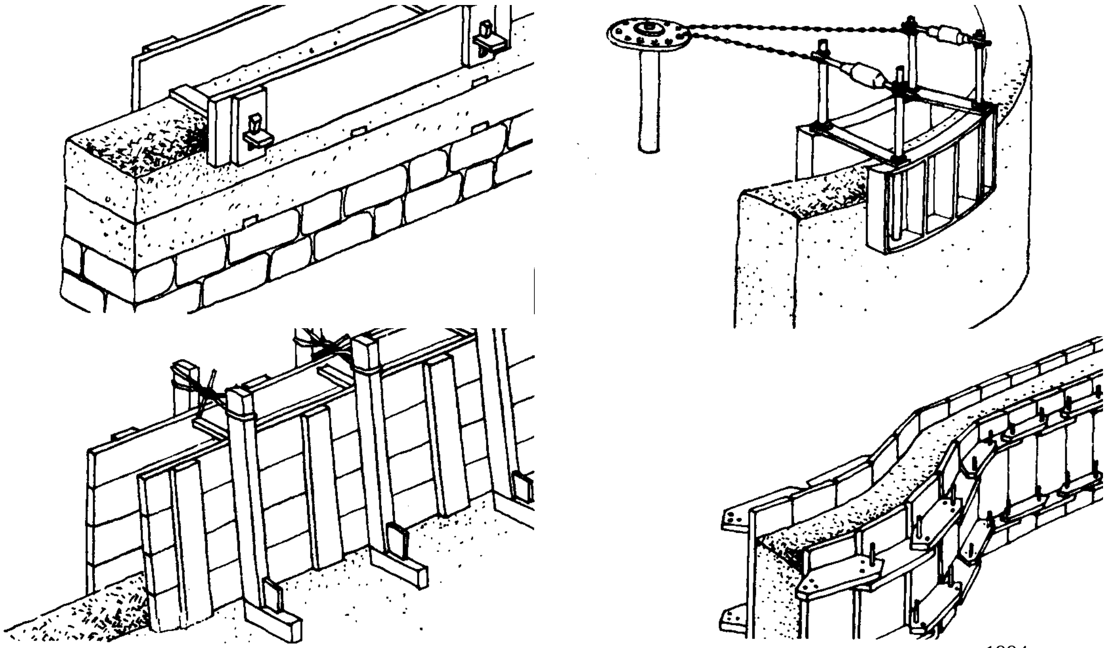


Figure 7.1 – Common types of shuttering used for RE construction (Houben[1994])

7.3. Equipment for making cylindrical RE tanks

Shuttering Most shuttering used for RE construction is designed to be used for the construction of straight walls. There are few examples of shuttering for curved walls, and those shown in the literature seemed to be unsuitable for cylindrical tank construction. The shuttering was, therefore, designed by the author and manufactured at the University workshops (RE-UK) and by local carpenters (RE1 and RE2) - see Figures 7.2 and 7.3. The UK shuttering required some subsequent strengthening to prevent deflection when compaction was taking place. The second set of shuttering was suitably strengthened during manufacture.

- *Shuttering fabrication methods used in UK*

The materials and techniques used for the shuttering construction varied slightly between the UK and Uganda. In the West, plywood is readily available, whereas in Uganda it is not. Plywood is well suited to forming curved surfaces through lamination and so 4mm plywood sheets were used to form the curved faces of the mould – 3 layers glued together. Plywood (12mm – doubled up) was also used to form the main strengthening spines of the shuttering. The tie rods were from 12mm steel rod with T's welded on one end and threaded to sufficient length on the other. The nuts used were fitted with wings to make shuttering assembly easier. The end-stops were made for 12mm ply. The shuttering is shown in Figure 7.3.

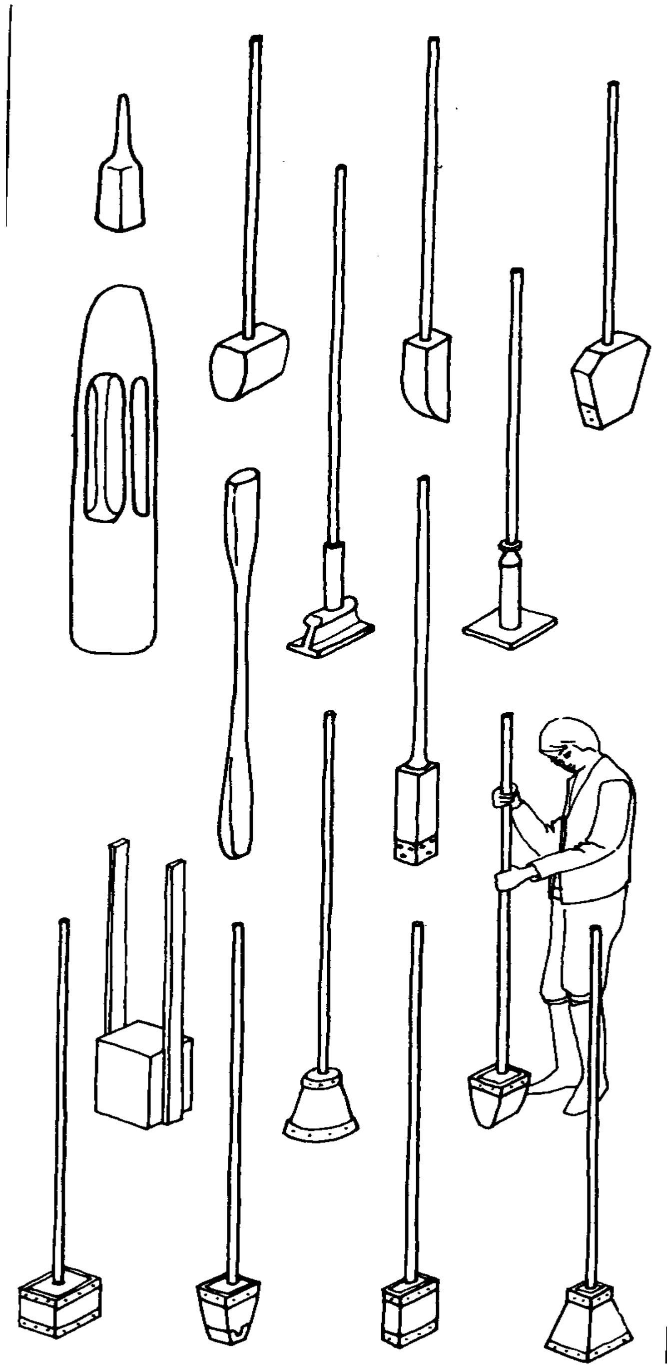


Figure 7.2 – Common types of rammers used for RE construction (Houben[1994])

- *Shuttering fabrication method used in Uganda*

In Uganda sawn hardwood timber is more commonly available and this was used for the construction of the shuttering. The curved faces of the mould were made using 25mm x 25mm strips of timber butted to one another and glued and nailed carefully to each other and to the main spines. The spines and end-stops were from hardwood. The work was carried out at a local workshop and needed constant supervision to ensure that the work was done in accordance with the drawings. The workshop had few power tools and most of the work was done by hand. Again, the tie rods were of 12mm steel rod with T's and threaded section. The shuttering is shown in Figure 7.4. Design drawings for the shuttering are shown in Appendix V.

Figure 7.3 – Shuttering manufactured in the UK

Figure 7.4 Shuttering manufactured in Uganda

- *Test for shuttering strength*

The standard test for strength of shuttering is that the mid point deflection of the shuttering, when loaded with a 150 kg weight (e.g. 3 bags of cement) should less than 3mm. Loading is mid point between the vertical stays.

- *Expected useful life of Shuttering*

Based on the experience in Uganda, where the shuttering was used to make two tanks, it is estimated that well-made shuttering should be good for the construction of between 15 and 20 tanks, with some maintenance required to repair any damage caused during ramming.

Rammers / tampers The tampers were manufactured at the University workshops and at a local workshop in Uganda. Drawings are shown in Appendix V. The handles were of hollow round section steel in order that the weight of the rammer can be adjusted by partially filling the handle with sand. The profiled 'V' rammer, used for creating a joint that helps prevent shear, was made using a piece of 50mm angle iron welded to the flat base of the rammer.

Covers The DTU ferrocement thin-shell cover was used on both RE1 and RE2 and a special sealed (pressure-resistant) cover is to be developed for RE-UK.

Tank linings
Experiments have been carried out with two types of tank lining:

- *Plastic lining*. Work on plastic linings has been underway for some time at the University by an MSc student. A technique has been developed for welding 250 micron construction or damp proof membrane (DPM) plastic sheet to make 'bags' (similar to large bin liners) that fit inside the tank structure to form a waterproof lining. The welding technique has been successfully developed but there are still problems to be overcome in relation to the quality of 'off-the-shelf' plastic sheet and failure of the lining due to abrasion.
- *Cement render lining with waterproofing agent*. This is a more traditional form of waterproofing for water storage tanks and was eventually used for both the field experimental tanks. Further investigation into the nature of render linings, with particular respect to permeability and the effects of waterproofing agents, is underway at the University.

7.4. Laboratory work on RE tanks

Aims of the laboratory experiments Between November 1999 and May 2000 laboratory tests were carried out at the University of Warwick. The work was carried out to allow the author to gain experience with earth building technology and to develop a technique suitable for building tanks from soil. The full aims of the work was as follows:

- to set out the theory of RE tank design and to investigate the options available for rammed earth tank construction
- to develop the skills required (within the research team) to analyse soils for building, in both the laboratory and in the field
- to investigate soil modification techniques and methods to develop a soil suitable for tank construction
- to develop the tools and equipment required for RE tank construction
- to develop a technique suitable for rammed earth tank construction
- to develop a technique for tank lining using plastic sheet
- to test a rammed earth tank under laboratory conditions to verify the theory with regard to:
    - hoop strength (reinforced and un-reinforced)
    - the effect of rain on stabilised soil
    - performance tests for soil:

- wet and dry tensile strength
- wet and dry bending strength
- wet and dry shear strength (all three to be carried out at experimental and applied levels)
- swell and shrinkage
- erosion
- abrasion
- passage of water
- compatibility (adhesion) with renders and mortars

Preparation for the laboratory experiments The test site was established in a disused water turbine testing sump in the university laboratory. The set up for the experiments was time consuming and the preparation of the site took several weeks, mainly fitting lighting and safety equipment to conform with university safety regulations

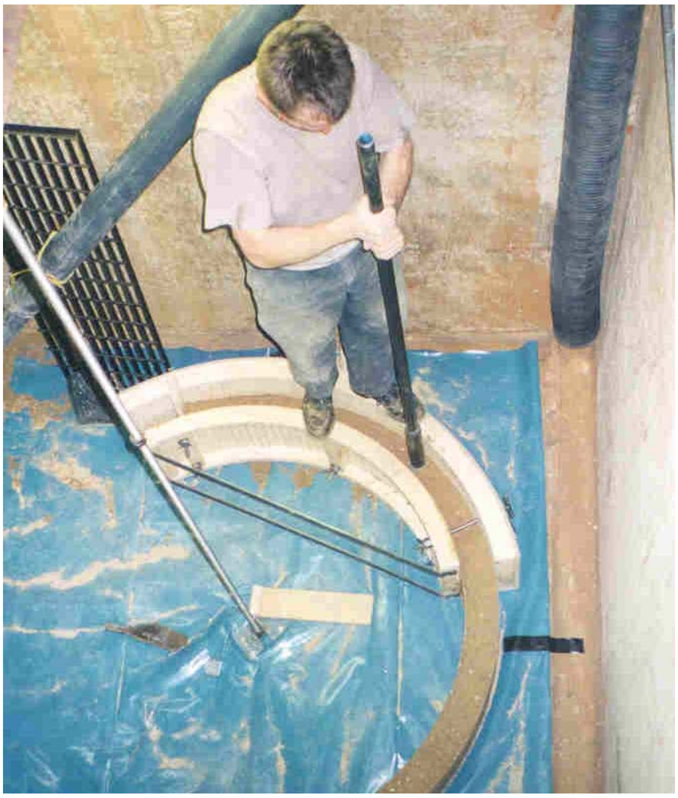


Figure 7.5 Showing tank test area at the university laboratory

Desk work to develop experimental tank specification
Sufficient deskwork was undertaken to formulate a specification for the RE tank. The theory of stresses in tanks is given in Section 4 along with detail of the spreadsheet used to analyse stresses and to develop the specification for the experimental tank. The specification is as given below:

Table 7.1 Specification for laboratory experimental tank

Tank internal diameter	1.40m
Depth	1.0m
Wall thickness	0.1m

Wall / base joint	‘slippery’ joint using plastic sheet
Section bonding	Polypropylene knotted rope to give tensile bond for vertical joints V channel ramming for better horizontal joints
Drying time	2 weeks in dry environment

Work carried out in the laboratory

Soil analysis What was thought to be a suitable soil was purchased from a local quarry and an analysis was carried out. The soil is known locally as ‘Hoggin’The soil was analysed using the standard wet sieving technique and the following results were obtained:

Table 7.2 Soil classification for Hoggin from Husbands Bosworth, UK, December 1999. Sample size 1500g, oven dried for 24hrs, wet sieved

Sieve Size (mm)	Weight retained	% retained	Description
20.000	220.6	14.87	Pebbles
6.300	394.6	26.59	Gravel
2.000	379.8	25.60	Gravel
0.420	222.5	15.00	Coarse sand
0.063	146.3	9.86	Fine sand
<0.063	120.0	8.09	Silt, fine silt and clay

Table 7.3 Results of a sedimentation test carried out on particles passing through 0.063mm sieve (from Table 7.2 above).

Time elapsed since agitation	Settled depth (mm)	% of total settled depth	Description	%age of total soil sample
1min	18	56.25	Sand	18.46
30 min	25	21.875	Silt	7.18
24 hrs	32	21.875	Clay	7.18

The soil was assessed and it was noted that:

a) there were many large stones >30mm
b) there was an excessive amount of material over 6.3mm in size – about 40% of total
c) there was insufficient clay for binding the material (7.18% of total material content)
d) there was insufficient sand in the soil

Soil modification It was decided that the soil should be modified. Several experiments were carried out to formulate a suitable soil and the following procedure was developed to prepare the required soil:

a) all large stones >30mm were removed
b) the soil was sieved to remove all particles above 10mm
c) the soil was then sieved again to isolate all particles between 5mm and 10mm
d) kaolin (china clay) was purchased and used to make up the deficiency in clay content

e) concrete sand was acquired to make up the deficiency in sand in the original soil

The final mix was as follows:

Hoggin <5mm	- 30%
Hoggin 5mm – 10mm	- 20%
Concrete (builders) sand	- 35%
Kaolin	- 15%

No stabilisation was carried out at this point and the material could therefore be recycled during the early tests.

Table 7.4 The sieve analysis for the modified soil (now known as Soil 4)

Sieve Size (mm)	Weight retained	%age retained	Description
6.300	83.80	9.16	Coarse gravel
2.000	232.10	25.37	Gravel
0.420	274.45	30.00	Coarse Sand
0.063	160.70	17.57	Fine sand
<0.063	163.80	17.90	Silt and clay

Soil 4 was used for all subsequent construction work in the laboratory.

Compaction tests and Optimum Moisture Content Compaction tests were carried out (as described in Section 3.3.2) on Soil 4 to determine the Optimum Moisture Content (OMC). The results show a moisture content of 9% to be optimum. However, in practice a figure of 8% was used as soil with 9% moisture content was too 'sticky' and caused the soil to stick to the rammers.

Liquid limit and plastic limit tests – the Atterburg chart Tests to determine the liquid and plastic limits were carried out and the results shown below:

Table 7.5 – LL, PL and PI Figures

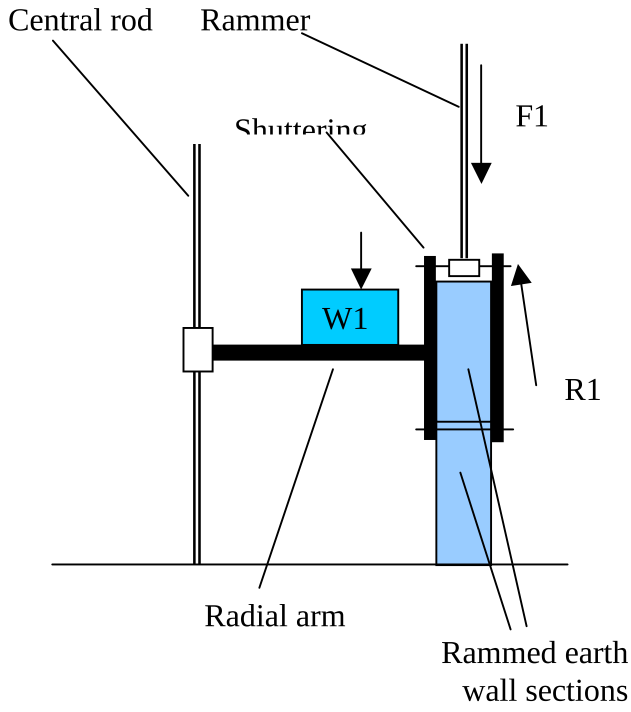
Liquid limit	14.80
Plastic limit	9.59
Plasticity Index	5.21

Figure 7.6 – Early experimentation with ramming wall sections

Initial ramming tests and developing the technique for RE tank construction The shuttering was developed in such a way that 3 sizes of wall thickness could be tried; 60mm, 80mm and 100mm. The reason for this was that it was doubtful if the smaller wall thickness (the lower lifts) would withstand the compaction blows of subsequent lifts.

Early tests showed a number of problems with the ramming technique:
- the shuttering was deflecting and causing the wall section geometry to loose its true curvature
- the 60mm thick wall showed signs of cracking when the shuttering was clamped to it for the subsequent section
- when ramming, a reaction caused the shuttering to bounce or lift slightly (see Figure 7.7)
- the geometry of the shuttering was crucial and problems were encountered due to slight irregularities in the geometry

These problems were overcome by some adjustments to the shuttering:
- strengthening was added to prevent deflection
- the wider setting was used to produce a wall of 100mm thickness
- radial arms were fitted to the shuttering to maintain centrality and to allow weights to be added to prevent 'bouncing' (see Figure 7.7)
- the geometry of the shuttering was corrected slightly as no allowance had been made for the width of the end stops



The ramming force, F1, causes a reaction in the shuttering, R1, which causes the shuttering to lift and hence cause deformation. The weight, W1 (two x 25kg bags of sand were used), counteracts R1 and prevents the deformation.

Figure 7.7 Showing the forces created through ramming and the measures taken to counteract these forces

Manufacture of experimental tank A tank of 1.4m diameter and 0.7m height was built in a disused turbine testing sump (see specification given in Section 6.3.3). The tank was built from Soil 4. The aim was to test the general principle of construction and to allow for initial tests to be carried out on earth tanks. The tank was completed in May 2000 and simple tests carried out, but full tests are yet be to be carried out on this tank The simple tests include fitting experimental linings, applying steel strapping and filling the tank a number of times to test strength. The fuller tests are to include pressure tests to ascertain the actual strength of the tank to allow comparison with the theory.


Figure 7.8 Showing the experimental tank in the sump at the university

Composite materials for added tensile strength Composite materials and reinforcement can be used to improve the strength of compacted soil. For improved tensile strength the added material should have good tensile properties and be malleable so that it can be rammed into the soil matrix. Some ideal materials include barbed wire, polypropylene fibre and straw.

External reinforcement for added tensile strength External reinforcement such as steel packaging strap, barbed wire, or plain fencing wire can be wrapped around the finished tank walls for added tensile strength.

Laboratory tests Due to time constraints and unforeseen problems with developing the technique for RE tanks, other planned laboratory tests have not yet been carried out. Tests are being undertaken at present to verify the theory with regard to:
hoop strength (reinforced and un-reinforced)
the effect of rain on stabilised soil

- performance tests for soil:
- wet and dry tensile strength
- wet and dry bending strength
- wet and dry shear strength (all three to be carried out at experimental and applied levels)
- swell and shrinkage
- erosion
- abrasion

- passage of water
- compatibility (adhesion) with renders and mortars

This work will be reported at a later date.

Conclusions and discussion concerning laboratory work on RE tanks Soil analysis was carried out for a locally sourced soil. The soil was modified to create a suitable soil for RE tank construction. The tools and techniques for RE tank construction have been developed.

Developing the technique for RE tanks proved to be more problematic and time-consuming than was originally thought, with much work needed to overcome geometry problems. The level of accuracy required, both in shutter manufacture and in the construction process, is higher than expected. Early tests show that the RE tank design has good potential.

Soil preparation time is high and so a suitable soil should be sought in the field that requires little preparation.

The problems encountered during the construction process meant that few further tests have yet been carried out.

Further laboratory work
Recommended further work is listed below:

- Manufacture of sealed cover and tank pressurisation equipment
- Pressure testing of the existing experimental tank
- Data logging to allow analysis of the stresses in the tank during pressurisation
- Performance tests to analyse the soils characteristics in use
- Experiments to determine the effects of cement content on stabilisation
- Experiments to determine the effect of cyclic loading on rammed earth tank walls (cracking, joint failure, etc.)
- Waterproof lining (resistance to penetration, fixing methods, liner penetration for off-take, etc)
- Wet strength tests of tanks

7.5. Field work

The principal aims of the field tests were:

- to test the technique developed in the laboratory for rammed earth tank construction
- to test the technique for tank lining using plastic sheet
- to build a number of tanks to test the feasibility of the technology and its suitability to LDC skills
- to test a tank under field conditions to verify the theory
- to carry out a full cost analysis of a RE tank to allow comparison with other tank types

Specification for 2 experimental field tanks

The specification for the two rammed earth tanks is given in the tables below. The specifications were drawn up using the results of the spread sheet analysis and with the specific aim of testing a) a tank using standard building guidelines for rammed earth construction (RE1) and b) a low-cost tank using predominantly stabilised earth and minimising the cement content (RE2).

High Specification RE Tank (RE1)

Tank internal diameter	2.0m
Tank external diameter	2.4m
Tank height	2.0m
Tank capacity	approximately 6 cubic metres
Concrete base thickness	100mm un-reinforced (on 100mm hardcore where ground is soft)
Concrete base diameter	2.8m
Soil wall thickness	0.2m
Soil make-up	10% clay, 15 – 30% silt, 50 – 70% sand, 10 – 20% garvel, 4% cement stabilisation
Reinforcement	barbed wire hoops at 50-60mm spacing in rammed earth sections steel reinforced cement hoops at spacing shown in drawing
Lower wall dimensions	350mm stone masonry, 0.2m thickness
Tank lining	waterproof render approx. 15mm thick (or plastic liner)
Cover	thin shell ferrocement cover
Water extraction	By gravity – washout also by gravity

Low Specification RE Tank (RE2)

Tank internal diameter	2.0m
Tank external diameter	2.4m
Tank height	2.0m
Tank capacity	approximately 6 cubic metres
Base	100mm stone with 50mm compacted stabilised soil, 2.8m diameter
Wall	Stabilised soil, 0.2m thickness
Reinforcement	barbed wire hoops at 50-60mm spacing in rammed earth sections
Soil make-up	10% clay, 15 – 30% silt, 50 – 70% sand, 10 – 20% garvel, 4% cement stabilisation
Tank lining	plastic liner
Cover	thin shell ferrocement cover
Water extraction	by handpump where plastic liner is used

Some of the design features and design considerations that were incorporated are outlined below:

- The wall sections were given a ‘V’ profile to prevent shear. The aim was to encourage better bonding and so reduce the likelihood of shear. The profile was achieved using a special ram with a V attached. See Figure 7.9.

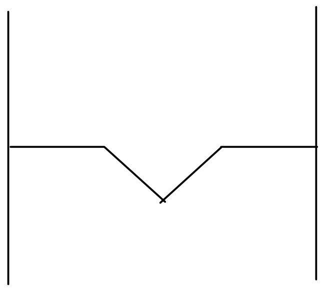


Figure 7.9 – Profile of the joint between 'lifts'

- The principle adopted for house construction of good protection at the base and overhanging eaves for protection from above were adopted, especially in the case of RE1
- The tank size is based on useful capacity and available catchment area
- Minimal cost – the general aim of this task is to reduce the cost of storage for rainwater
- The use of local materials as far as possible
- Durability – a minimum of 10 - 15 years life expectancy
- Easily constructed by local artisans in developing countries
- Foundations – these should be:
    - should be laid in a good sound soil
    - be protected against ingress of moisture
    - be protected against frost (not an issue in the Trpoics0
    - be protected from wind erosion – sand blasting in severe storms
    - be protected against animals, rodents, insects, etc
- Protection from surface water through good drainage (Houben pg 252) – ground to slope away from tank, ground gutters, moisture barriers,
- Foundations in unstable soils need to be considered – ground stabilisation may be needed

Work carried out in the field

The two tanks specified in the previous section were constructed at Kyera Farm. The procedure for their construction is outlined in Appendix IV. Each step of the work was carefully observed and monitored against the aims set out in Section 6.4.1. An initial field soil analysis was carried out to determine a suitable soil. Some modification was needed and then a full laboratory analysis was carried out on the modified soil.

Tests carried out in the field

Soil analysis – field tests

Three soils were analysed crudely in the field. The following tests were performed:

Sensory tests

The soil was analysed by feel, look and smell to test for initial suitability

Sedimentation test. The results are shown below:

Table 7.6 Results of the sedimentation test carried out in the field

Soil source	Soil code	T1	T2	T3	% sand	% silt	% clay
Murram from farm <4mm	S1	40	42.5	42.5	94	6	0
Anthill P <4mm	S2	22	53	53	37.7	62.3	0
RE2 excavation site <4mm	S3	53	59	59	89.8	10.2	0

Notes:
T1 = reading taken in mm, start time plus 1minute
T2 = reading taken in mm, start time plus 30 minutes
T3 = reading taken in mm, start time plus 24 hours
The murram has some organic material; good distinction between layers.
Anthill has little organic matter; poor distinction due to uniform colour.
RE2 excavation site high organic content; good definition.
Clay content is shown as 0% in each of the above. This is because further settling of the sand and silt after initial measurement caused final reading to be low.

A further sedimentation test was carried out to estimate the clay content. Tests were carried out on the murram and the anthill soil. Soil passing through a 0.075mm sieve was sedimented and readings taken at 20mins, 24hrs and 48hrs. From the readings obtained, the clay content was estimated to be:

Table 7.7 Results of tests to determine clay content of soils

Soil	**Clay content**	**Comment**
Murram from farm <0.075mm	4%	Again definition was difficult and the reading was not fully trustworthy
Anthill P <0.075mm	0%	Settling took place rapidly indicating little or no clay content

Soil modification

None of the three soils tested was fully suited to RE construction. However it was found that by mixing the larger particles of the murram (those retained on a 4mm sieve) with the crushed anthill soil, a suitable soil was developed. Stabilisation was provided by adding 4% cement. The cement percentage was kept deliberately low to save on cost. The final mix is shown in Table 6.8.

Table 7.8 Showing mix used for modified soil for RE tank construction

Material	**Quantity**
Crushed anthill soil	80%
Murram >4mm	16%
Cement	4%
Water	Added in sufficient quantity and checked using field test

Soil analysis – laboratory tests

An analysis of the modified soil was carried out at the Central Materials Laboratory, Ministry of Works and Housing, Kampala. The analysis is summarised below.

- *Sieve analysis*
  The full analysis is shown in Appendix III but a summary is given in Table 6.9

Table 6.9 Summary of sieve analysis for RE1 and RE2 tank soil

Classification	
Pebbles	0%
Gravel	21%
Coarse sand	17%
Fine sand	17%
Silts	10%
Fine silts	19%
Clays	16%

- *Other tests*

Table 7.10 Results from laboratory tests for construction soil

Test name	Testing for	Result
Hydrometer analysis	Specific gravity	Gs 2.65 (measured)
Atterburg limits(1)	Liquid limit	34
	Plastic limit	16
	Plasticity index	18
Normal moisture content	Normal moisture content	2%
Compaction and OMC(2)	Material density	1,930Kg/m^3 (Maximum dry density – MDD)
	Optimum moisture content	12%
Linear shrinkage	Linear shrinkage	10%
Permeability	Permeability	4.09 x 10^{-7} m/s
Unconfined compressive strength	Unconfined compressive strength	0.2MPa @ 95% MDD

Notes:
[1] Referring to the Atterburg Limits Chart in Figure 2.1, we can see that the ideal stabiliser for this soil is cement.
[2] A heavy compaction test was carried out to determine density and OMC:
The results are discussed in Section 6.4.7.

Water bearing test on finished tanks

RE2 was filled using a bowser. It was filled quickly using a motorised pump. The tank held water for about 8 hrs but damp patches appeared at the base of the tank. 'Piping*' caused loss of water which became critical after about 12hrs and there was a total loss of water after approximately 14hrs. As the wall became damper, 3 major cracks appeared in the tank wall. The possible reasons for the failure are discussed in Sections 6.4.7 and 6.4.8.

*Piping is a term used in the dam industry and is a mode of failure whereby water finds a path from one side of the dam wall to the other and the subsequent erosion of the wall causes failure.

Figure 7.10 The experimental rammed earth tank under construction at Kyera Farm, Mbarara, Uganda.

As a result of the failure of RE2, RE1 was relined and is awaiting slow filling by rain. RE1 has not yet be tested at full working water pressure.

7.6. Results of field and laboratory tests

Some observations regarding the laboratory tests:

- The results of the sieve analysis shows that the soil is suitable for RE construction.
- Referring to the Atterburg Limits Chart in Figure 2.2, we can see that the ideal stabiliser for this soil is cement.
- Compaction density (MDD) seems rather low – normally we would expect a MDD of at least 2000Kg/m^3.
- OMC seems rather high – this is normally in the range of 8 – 10%
- Permeability is high at 4.09 x 10^{-7}m/s. The norm for stabilised soil is 1 x 10^{-8}m/s.
- Unconfined compressive strength seems very low and this figure is not trusted.
- Linear shrinkage is also high.

The result of the water bearing test described in Section 7.4. was discouraging as far as the overall viability of RE tank technology is concerned. The failure highlighted a number of points:

- Soil tanks should have sufficient wet tensile strength to cope with total saturation in water. This has a strong impact on the design of such tanks and could result in excessive wall thickness.
- A tensile reinforcement should be included within the soil matrix to prevent catastrophic failure in the event of saturation.
- A good tank lining is critical. A poor leakage of the lining will cause rapid degradation of the wall.
- Where a waterproofing agent is used in a render lining it should be given adequate time to take effect. Follow the manufacturers instructions and where they are

lacking, fill the tank slowly, at no more than 200mm per day after 2 weeks of curing in a saturated environment.

7.7. Observations made regarding the RE tank construction technique and field tests

The construction technique developed in the laboratory was further refined in the field. The following observations were made:

- Soil preparation was initially very time consuming, until a groundnut sheller was acquired and modified to mill soil through a 4mm sieve (see Appendix IV). This reduced soil preparation time to about one tenth.
- RE construction is a time consuming process. The cost analysis in Section 6.5 shows how labour intensive the process can be.
- The shuttering used in the field had no radial arms and so the reaction discussed in Section 7.3 was dealt with by standing on the shuttering whilst ramming.
- The level of accuracy attained in the field was lower than in the laboratory. This was not a problem, however, as soil was cut away with a machete if the geometry was not perfect. This was more feasible in the field as the wall thickness of these tanks was higher.
- Some work was carried out to assess the feasibility of plastic liners for use with soil tanks. There were problems in a number of areas: finding good quality plastic sheet; finding plastic sheet of a suitable size; abrasion of liners when in use causing small puncture holes. The technique is not recommended for use at present.

Limitations of the field tests Time constraints were tight due the limited amount of time in-country. It was difficult to set up and execute proper tests during the limited time available, the construction process itself being very time consuming. The tests that were carried out were done so hurriedly and give only an indicative feel for the behaviour of the tank.

7.8. Conclusions, discussion and planned further work concerning RE tanks

Discussion The probable mode of failure of RE2 is outlined in this Section. It appears that water passed through the lining of the tank almost immediately the water was placed in the tank. It seems probable that the waterproofing agent had had insufficient time to act and that water was passing through the render under full water pressure. Alternatively, the lining was of poor quality and water passed through cracks in the lining. The water forced its way through the soil matrix under piping action until it emerged at the outer face of the wall, and the resultant erosion increased the ‘pipe’ area until the water could flow freely from the tank (see Figure 7.11).

The fact that cracks appeared in the tank wall mean that the wet strength of the soil was insufficient to withstand the forces exerted through normal working water pressure, which is not tolerable.

Figure 7.11 Failure of RE2 through 'piping' – this figure shows the leakage at the base of the tank and the crack running through the wall

Further work on RE Tanks A similar water pressure test will be carried out on RE1, as was carried out on RE2. This tank has been relined and will be filled with rain during the coming wet season (starting September 2000). Staff at Kyera Farm have been asked to observe the tank as it slowly fills.

The following is a list of further work that is recommended to clarify uncertainties regarding the technique:

- Performance tests on soil samples taken from field
- Further investigations into suitable levels of soil stabilisation
- Investigation of techniques for improving wet strength of stabilised soil
- Investigation of techniques for decreasing permeability of stabilised soil (e.g. inclusion of bitumen emulsion)
- Further field tests including destructive tank pressurisation tests
- Further work to develop suitable plastic or other flexible linings

8. Cost analysis

A cost analysis for an 11 cubic metre ferrocement tank and an 11 cubic metre rammed earth tank has been carried out

The bill of quantities for the ferrocement tank was taken from Gould and Nissen-Peterson, 1999. The rammed earth tank is an externally rendered tank fitted with a thin-shell ferrocement cover. It is also fitted with a plastic 'sock' lining as described later in this document. Hoop strength is augmented using barbed wire hoops spaced every 0.1m for the entire height of the tank. Cement content is 5% and moisture content of the soil is 8%. We have used material prices based on costs in Uganda as of August 1999. This allows us to make a realistic analysis using costs from a single location and an identical size of tank. We are therefore making a direct cost comparison. The cost comparison and relative benefits may change if the costing is repeated for a different location (with different prices). India, for example, has lower cement prices, which would impact greatly on this comparison. One pound sterling is roughly equivalent to 2440 Ugandan shillings (UGX) as of January 2000. Transportation costs for materials (other than sand) are not included.

The costing figures are shown in the table in APPENDIX I and the table is self-explanatory. Some points to consider are:

- Cement is the major cost in the ferrocement tank. There is little means of reducing the cost without reducing wall thickness. Transportation of cement can be costly if the site is remote.
- The cost, including labour, of the thinner walled rammed earth tank is about 80% that of the ferrocement tank. Material costs are about 72%. This reflects the higher labour input required.
- If suitable soil is available on site then the material costs of the RET can be reduced significantly.
- The RET wall sits on a concrete ring rather than a concrete disc. The base of the tank is of compacted earth. This helps reduce costs as concrete raft bases consume significant quantities of material and are therefore expensive.
- The RET wall is stabilised. The wall would have sufficient strength without stabilisation and the cost could be reduced further. Stabilisation is desired however in case of wetting of the wall.
- If stabilisation and reinforcing is omitted, the cost of the (thinner-walled) RET drops to 697,000 UGX or £278.28, 58.5% that of the ferrocement tank cost (including labour).
- Tooling costs, shuttering and moulds in particular, tend to be higher for the ferrocement tank, although for both tanks the moulds / shuttering can be reused many times.

9. General conclusions, discussion and plans for further work

9.1. Discussion

A number of stabilised soil tanks have been constructed and tested, using two different soil construction techniques. Two tanks have failed under test and the failures have been analysed in the relevant sections of the report. Two further tanks are still under test. Some important lessons have been learned from the initial tests and these lessons will be used to guide further research. The main lessons learned are listed here:

- When designing stabilised soil tanks one should take into consideration wet tensile strength of the material. It is inappropriate to use the dry tensile strength as the design strength value unless it can be guaranteed that water will never reach the soil matrix.
- A composite tensile member is recommended to prevent sudden catastrophic failure of a tank.
- Tank linings should be fully impermeable to prevent water reaching the soil matrix.
- Plastic linings

9.2. Recommended further work

Stabilised soil block tanks

- Full laboratory analysis of the soil
- Performance tests to determine the soil / block characteristics
- Further full scale tank tests with some modifications to improve tank performance
- Investigate methods for improving tensile strength e.g. reinforcing with barbed wire

Rammed earth tanks - Laboratory work

- Manufacture of sealed cover and tank pressurisation equipment
- Pressure testing of the existing experimental tank
- Data logging to allow analysis of the stresses in the tank during pressurisation
- Performance tests to analyse the soils characteristics in use
- Experiments to determine the effects of cement content on stabilisation
- Experiments to determine the effect of cyclic loading on rammed earth tank walls (cracking, joint failure, etc.)
- Waterproof lining (cyclic loading with a variety of ‘sharp’ objects to test resistance to penetration, fixing methods, liner penetration for off-take, etc)
- Wet strength tests of tanks

Rammed Earth Tanks - Field work

- Performance tests on soil samples taken from field
- Further investigations into suitable levels of soil stabilisation
- Investigation of techniques for improving wet strength of stabilised soil
- Investigation of techniques for decreasing permeability of stabilised soil (e.g. inclusion of bitumen emulsion)
- Further field tests including destructive tank pressurisation tests
- Further work to develop suitable plastic or other flexible linings

Other work

- Render linings for tanks – permeability tests and effects of waterproofing agent on render (both in early stages and longer term)
- Methods for reducing permeability of compacted soils (e.g. treatment with bitumen emulsion)
- Further experiments on the relationship between wet and dry strength in tension
- Tank linings from plastics
- Termite attack on soil tanks
- Sliding joint at base of tank

10. References and bibliography

1. DTU Roofwater Harvesting Programme, http://www.eng.warwick.ac.uk/DTU/rainwaterharvesting/index.html, DTU, School of Engineering, University of Warwick, UK, 2000
2. Gooding D., *Quasi-Static Compression Forming of Stabilised Soil-Cement Building Blocks*, DTU WP 40, Warwick University, 1993
3. Gould, John & Nissen-Petersen, Erik, *Rainwater catchment systems for domestic supply*, IT Publications, London, 1999.

4. Gray, W. S., *Reinforced concrete reservoirs and tanks*, Concrete Publications Limited, London, 1960.
5. Houben, H. & Guillaud, H., *Earth Construction - a comprehensive guide*, CRATerre-EAG, Marseille, 1989.
6. Keable, Julian, *Rammed earth structures: a code of practice*, IT Publications, London, 1996.
7. Montgomery D., *Dynamic Compaction of Soil for Low-cost Building Blocks*, DTU Working Paper WP53, Warwick University, 1999
8. Neville, A. M., *Properties of Concrete*, Addison Wesley, UK, 1997
9. Norton, John, *Building with earth - a handbook*, IT Publications, London, 1997.
10. Turner, Stephen, J., *Design of storage tanks for rainwater harvesting*, (dissertation), Department of applied mechanics, Indian Institute of Technology, Delhi, India, May 1999.
11. Watt, Simon B., Ferrocement water tanks and their construction, IT Publications, London, 1978.
12. *BS 1377, Soils for Civil Engineering Purposes*, BSI, UK, 1990.

APPENDIX I - Cost comparison between 11 cubic metre ferrocement tank and two rammed earth tanks (of wall thickness 0.2m and 0.3m).

					11 cub m ferrocement (Nissen-Peterson, 6 cm wall thickness)		11 cub m rammed earth tank (wall thickness 0.2m)		11 cub m rammed earth tank (wall thickness 0.3m)	
Item	**Specification**	**Unit**	**Unit cost**		**Quantity**	**Cost**	**Quantity**	**Cost**		
Cement	50 kg	bag	15,500		22	341000	11	170500	15	232500
Lime	25 kg	bag	10000	*	1	10000				
Sand	coarse and clean	tonne	30000		5	150000	7	210000	11	330000
Crushed stones	10 to 20mm	tonne	40000		2	80000	2	80000	2	80000
Rubble stones	100 to 500mm	tonne	30000		1	30000				
Bricks	variable	number	70		50	3500	100	7000	100	7000
Water	200 litre	oil drum			15	0	5		7	
BRC mesh	No 65	m	6000		24	144000				
Chicken mesh	25mm, 0.9m	m	3000		38	114000				
Twisted iron	12mm	m	2000	*	3	6000				
GI wire	3mm	kg	1,250		10	12500				
GI Pipe	38mm	m	7000	*	0.9	6300	1	7000	1	7000
GI Pipe	18mm	m	5000	*	0.9	4500	0.3	1500	0.4	2000
Tap, elbow nipple and socket	18mm	unit	15000		1	15000	1	15000	1	15000
PVC pipe	100mm	m	5000		2.2	11000				
PVC pipe	50mm	m	3000		3	9000	2	6000	2	6000
Coffee mesh	galvanised	m	3000	*	1	3000	1	3000	1	3000
Mosquito mesh	plastic	m	2000	*	0.5	1000				
Lockable door	steel	0.9 x 1.5	20000	*	1	20000	1	20000	1	20000
Reinforcing steel	8mm	10m lngth	10000				3	30000	3	30000
Plastic liner	custom made	unit	20000	*			1	20000	1	20000
Barbed wire	galvanised	roll 50m	50000	*			2.5	125000	2.5	125000
Skilled labour	mason, supervisor	day	12,000		10	120000	12	144000	14	168000
Unskilled labour	assistants	day	4000		20	80000	24	96000	28	112000
Totals		Total minus labour			UGX	960,800		695,000		877,500
					£	393.77		284.83		359.63
* estimated cost		Total with labour			UGX	1,160,800		935,000		1,157,500
					£	475.73		383.19		474.38

APPENDIX II - REPORT ON WATER TANKS FROM STABILISED SOIL BLOCKS

(by Engineer Dr. M. K. Musaazi PhD DIC, August 2000)

Machinery

Manual Block Press (by ApproTec Kenya): Specially modified by authorized manufacture (Makiga Engineering Services, Kenya) to make curved interlocking blocks. There are two Block Presses designed so that 13 blocks interlock to make a 1.0m internal diameter circle or 17 blocks make a 1.5m internal diameter circle.

Material

Murrum (red) soil or volcanic soil and Ordinary Portland Cement (OPC)

Material Preparation

Sieve soil with a 5mm wire mesh. Mix while dry with OPC in the ratio.
Cement : Soil = 1: 19

i.e. 50kg(l bag) of OPC makes 110 SSB blocks each of about 9kg. A little water is added to make the mixer just damp.

Block Making

Prepared material compressed by 40% to make one block at a time.
Two men make 330 blocks/day.

Curing and Drying of Blocks

Blocks are cured under a black Polythene sheeting for five days if made from murrum and for 2 days if from volcanic soil. Dried for 5 days under direct sunshine, after curing, before used to make tanks.

Tank Building

A 150 mm concrete base is made and blocks laid the following day. Blocks interlock to form a circle desired internal diameter.

Experience has shown that the internal diameter can be increased from design by as much as 0.5m without any noticeable distortion of the circle.

The blocks are laid using a 1:4 cement to sand ratio. One bag (50kg) of OPC lays about 200 blocks. The inside of the tank is plastered with fine sand, OPC and water proof cement (I kg added to 50kg of OPC).

Block Size:

Length = 280 mm
Width = 140 mm
Height = 110mm

Hence the tank wall is 140 mm thick plus 10 mm of plaster.

Tank Sizes

The size of course depends on the internal diameter and height. The tanks that have NOT failed number five as tabled below:

TANK NO.	DIMENSIONS	CAPACITY (Litres)	WHEN MADE MONTH, 2000	REMARKS
1	Int diam – 1.0m Height - 5.2m Block layers – 13 Total blocks – 507	5,800	March	Full up to 3.5m as of 1st August 2000
2	Int diam – 1.24m Height - 2.12m Block layers – 15 Total blocks – 240	2,500	March	Full to capacity as of 1st August 2000
3	Int diam – 1.20m Height - 2.12m Block layers – 14 Total blocks – 210	2,400	April	Full to capacity as of 1st August 2000
4	Int diam – 1.20m Height - 2.12m Block layers – 14 Total blocks – 210	2,400	April	Full to capacity as of 1st August 2000
5	Int diam – 1.20m Height - 2.12m Block layers – 14 Total blocks – 210	2,400	May	Full to capacity as of 1st August 2000

One tank that ruptured when trying to fill it with water had the following dimensions:

Int. Dia = 1.67 m
Height = 3.80 m
Blocks/layer = 19
No. of layers = 28
Capacity = 8.324 litres

Tank ruptured when about 2.5m full.

Possible causes of tank failure:

(i) Diameter too large, hence insufficient hoop strength
(ii) Filled too quickly with a tanker – this does not give the waterproofing agent time to work properly

(iii) Blocks from volcanic soil seem not to have been stabilised – broken blocks disintegrated completely when soaked in water. This could be due to poor workmanship, although the building team are known and trusted.

(Figure AII.1 – Tank built for experimental tests in Uganda)

(Figure AII.2 – failure of the SSB tank was catastrophic – the debris remaining after failure is shown here)

APPENDIX III - Sieve analysis for soil used at Kyera Farm

mm	% passing	%retained	Classification
50.0000	100	0	Pebbles 0%
37.5000	100	0	
20.0000	97	3	Gravel 21%
10.0000	94	3	
6.3000	87	7	
5.0000	84	3	
2.0000	79	5	
0.6000	75	4	Coarse sand 17%
0.4250	73	2	
0.3000	68	5	
0.2120	62	6	
0.1500	52	10	Fine sand 17%
0.0630	45	7	
0.0600	43	2	Silts 10%
0.0579	42	1	
0.0411	40	2	
0.0258	38	2	
0.0202	35	3	
0.0148	25	10	Fine silts 19%
0.0107	23	2	
0.0076	22	1	
0.0054	22	0	
0.0044	20	2	
0.0038	20	0	
0.0034	19	1	
0.0031	19	0	
0.0028	18	1	
0.0027	16	2	
0.0016	15	1	Clays 16%
<0.0016		15	

APPENDIX IV – Rammed Earth Tank Construction Guidelines

Including Shuttering and Rammer Drawings

Domestic Roofwater Harvesting
Research Programme

Roofwater
Harvesting
Research
Group

RECOMMENDATIONS FOR DESIGNING RAINWATER HARVESTING SYSTEM TANKS

O-DEV Contract No. ERB IC18 CT98 027
Milestone A6: Report A4

January 2001

THE UNIVERSITY OF WARWICK

Development Technology Unit

School of Engineering, University of Warwick

CONTENTS

1. INTRODUCTION

The water store (the 'tank') accounts for a large fraction of the cost of any roofwater harvesting system. Most poor households cannot afford to buy as large a tank as their roof catchment area might justify. There is therefore a strong incentive to seek cheaper yet adequate forms of tank.

The cost of a tank depends upon its size, the type and quantity of materials used in its construction, the labour needed to build it and in some areas the 'hire' of special equipment. The tank itself might be regarded as having two main parts; a water store and a set of ancillaries to lead water into and out of it (taps, an overflow, a filter, a level indicator etc.). In this paper, we restrict ourselves to considering just the water store.

Rainwater collection is in no way a new technology and is described in many publications (Gould & Nissen-Petersen, 1999; Lee & Visscher, 1992; Pacey & Cullis, 1996)In the last decade, there have been many attempts to identify better tank designs. Therefore this report starts with an analysis of needs and constraints and follows with an analysis of good existing designs. Lastly comes a description of our own work towards making tanks cheaper.

2 TANK REQUIREMENTS & CONSTRAINTS

2.1 Needs and specification

Tanks need to be watertight although some leakage (such as <5% of daily abstraction) might be tolerable if it does not weaken the structure or cause puddles. They also need to hold the required volume and to be adequately durable (say 25 years before they become unserviceable). Beyond these basic requirements we can list many further specific requirements. Tanks should:

- have a means of being charged with water without unduly disturbing tank-bottom sediments and if possible maintaining stratified flow (the bacterial quality of outlet water is maximised if the flow through the tank resembles 'pipe flow', namely 'last in is last out'
- be able to handle excess input by overflowing in a convenient and safe manner - preferably without leading water unnecessarily via the tank (such water may drop unwanted sediment in the tank)
- have a means by which the water can be extracted which is convenient for the user and which does not pollute the water left behind (as dipped buckets may)
- exclude vermin and as far as possible mosquitoes
- exclude light (so that algae do not grow and larval growth is inhibited)
- have some form of ventilation, especially if there is not an efficient filter to prevent organic material from entering the tank and decaying there
- be easy to access for cleaning (where cleaning is needed) and be unlikely to be damaged during cleaning
- have a sufficient structural safety factor to withstand wear and tear, some impacts and occasional large natural forces caused by winds and (in places) earthquakes

- not present hazards to passers-by or small children and (in some societies) offer some protection against deliberate poisoning
- not give the water a bad taste

2.2 Local constraints

There are often also local constraints upon the construction process, such as:

- absence or excessive expense of particular materials (e.g. cement, sharp clean sand)
- constraints upon the plan-area space, height or depth of a tank
- tank location; some designs are easier to locate than others

3 ROUTES TO CHEAPER TANKS

3.1 Changing To cheaper materials

Surface tanks for roofwater storage are commonly made of brick, ferrocement, concrete blocks, plastics and galvanised iron. Some of these materials are themselves variable in make-up (the cement or steel fraction within ferrocement can be varied) and all can be varied in thickness, as is discussed in the next section. Requiring a material to be both strong and waterproof considerably constrains its choice, Once however one accepts that waterproofing and structural strength can be separated and accommodated by different materials – a number of new materials options appear.

Bricks

Burned bricks are often made locally and are available much more cheaply than materials which have to be imported into an area such as plastics or cement. Tanks made from bricks can therefore be cheaper than those of "imported" materials and will also keep more of the money spent on the tank within the community. The challenge when building such tanks is to absorb the tensile stresses typical of water tank with a structure best suited to compressive forces. The Rainwater Harvesting Research Group has experimented with a number of alternatives such as external reinforcing and shaped bricks. This work is detailed in Section 7.

Stabilised soil

Another "earth technology" is to use a small amount of cement to hold together (stabilise) an otherwise soil based block. This technique can dramatically reduce the quantity of cement needed to make a tank of equivalent strength. The wet strength of stabilised soil is considerably lower than cement so the designer must either balance the cement content to ensure the wet strength is sufficient or employ some waterproof barrier to prevent water soaking into the blocks.

Rammed earth

Ramming earth between to walls ("shutters") compacting it, gives the wall a stiffness that simple soil building does not have. The technique uses only local materials and can be achieved without particularly specialised labour, it does however require some specialised tooling such as the shutters and a tamper to ram the earth into place but these can be made locally and used to make several tanks spreading their cost. The tank wall, however, is not waterproof with this technique so a means of waterproofing the inside surface must be employed.

Wattle and daub

A traditional technique for house building in many marts of the world, wattle and daub used mud to fill in a structure made from collected roundwood. The technique is well known and practised at a village level and requires no specialised tooling or knowledge to implement. All materials are available locally and usually need only be collected, thus the only capital requirement is labour making it extremely suitable for “self help” projects.

Like all “raw” earth technologies, the technique, results in a structure that is not waterproof and so a method of holding the water must be employed. One excellent example of this is the Rwandan “tarpaulin tank” which utilises an UNHCR tarpaulin in an excavation to hold the water with a wattle and daub enclosure. This tank is further described in Section 6.

Plastics sheeting

Plastic sheeting is becoming available in many parts of the world and can be used for lining otherwise permeable tanks to render them waterproof. At a basic level this could be simple polythene sheet but this tends to have a short lifespan. There are also several fibre reinforced plastic sheets such as those used for tarpaulins becoming available in centres and also in areas of specialised demand. At present it is unlikely that these materials will shortly become widely available due to a lack of demand but appropriate promotion/dissemination could change this situation.

Waterproof coatings

Waterproof paints are quite common in the developed world where they are used to seal ponds, swimming pools etc. these paints are available in some LDC centres and local variants may be developed. Quality control will become a major issue for these to be used as any uncoated sections could result in dangerous catastrophic failure of the tank.

3.2 Minimising the volume of materials used

A second method of cost reduction is by reducing material quantities. In general we can save materials by four approaches, namely:

- removing material where it is not needed
- reducing overall material use by reducing safety factors if they are unnecessarily high
- making use of some existing structure (e.g. the wall of a house or the ground itself)
- adopting a more efficient shape whereby overall tank wall thickness can be reduced

Removing material

The first of these approaches is sometimes prevented by practicalities. Wall thickness is often dictated not by strength but by buildability. For example the upper part of the walls of a cylindrical tank are subject to lower pressure than the bottom and so theoretically could be thinner. Unfortunately it is impractical to taper bricks or even ferrocement as one builds up the wall. the. It is, however possible to go some way toward this ideal.

- If coiled wire reinforcing is used, the spacing can be varied from close-spaced at the bottom of a wall to wide-spaced near the top.
- Materials can be concentrated so that extra strength is provided only where actually needed; for example cement content of ferrocement or stabilised soil blocks can be varied with height above the base (using least at the top).
- Material thickness can sometimes be reduced step-wise by, for example using a double run of bricks at the bottom and reducing to a single run further up the tank.

Material savings should be balanced against the extra complexity of manufacture. All of these techniques have quality control implications and should be used only when workers are familiar with the techniques or are well supervised.

Reducing safety factors

The stresses in water tanks can be calculated and then the expected stress compared to measurable properties such as maximum tensile strength, compressive strength, change of shape under load, etc. So long as the expected stress is lower than the chosen maximum material stress (usually the tensile yield stress in the case of tanks), then the structure will not fail. In practical problems, the expected stress is multiplied by a safety factor (F_S) for a number of reasons.

- The local load may be larger than we realised (indeed our method of calculation may itself contain serious inaccuracies)
- The material may be weaker than it should have been or some of the original strength may have been lost through wear and tear
- The material will almost certainly not be homogeneous, that is it will be stronger in some places than others (this is especially true of building materials)

A safety factor of 5 is typical for a water tank made of building materials. If the safety factor is very large (say $F_S = 15$) then material is being wasted and savings can be made.

Few practitioners of DRWH in developing countries include a well-considered safety factor in tank design calculations. The safety factor is usually applied to tanks in one of two ways:

- by arbitrary application during design, usually leading to excessive wall thickness as the engineer errs on the side of caution
- by trial and error leading to many trials and many errors

Arbitrary application can be expensive in terms of materials, while trial and error is expensive in terms of broken tanks (and even downright dangerous if field trials use consumer tanks).

Defining a sensible safety factor can be difficult given the extreme variations in quality of materials and workmanship in developing countries. The normal engineering approach to safety factor application is to use standard engineering materials (of more or less known strength) and to look up the appropriate safety factor in an engineering data book or approved ‘code of practice’. The nature of tank construction in LDC’s is however such that ‘standard’ (well-quantified) civil engineering materials are rarely available. Sand and aggregates as found in the local village and cement itself is often of dubious quality. Reinforcement may be of poor quality and the strength of bricks fired in a clamp kiln will vary from one to another. Safety factors will therefore vary depending on the type and variability of the material used and the level of skill available to build the tank.. Some suggested Factors of Safety are in Table 1

Table 1: Factors of Safety

Material	Skill level	Factor of safety
Ferrocement	High	2-3
	Medium	4-5
	Low	7-8
Burned Brick	High	3-5
	Medium	6-8
	Low	9-12
Galvanised Iron	High	1.2-1.4
	Medium	1.5-1.8
	Low	1.9-2.3

Architectural integration

The third approach, of saving costs by integrating a rainwater tank with a house structure, has been discussed from time to time but rarely found to yield decisive economies. For space or aesthetic reasons, tanks have often been located within a house's structure (especially where the house is multi-storey) but it is hard to show that any significant material saving has been so obtained. A shallow tank with a large area may substitute for part of a roof, however the requirement that the tank's top be *lower* than the bulk of the roof from which it is supplied restricts this substitution to say veranda roofing or between stories of a multi-story dwelling. The volume of the tank is also limited by the structural integrity of the roof supports as $1m^2$ of water weighs in at one tonne!

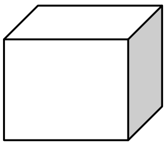
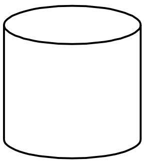
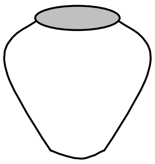
Conversely a very tall tank might substitute for walling. This has been proposed by ??? for Australian houses (ref?) but the architectural detail of moving from normal to tank walling and back again is complex and the long, flat wall will be subject to large bending stresses. Moreover a deep, thin tank has a poor ratio of volume to wall area and may also be difficult to clean. A tank could be made as a core round which a house could be built, in the same way as some traditional village houses in India were built around grain stores. All in-house tanks, however have the problem of ensuring that tank overflow will never inundate the house – as often happens with roofs having 'internal' gullies. Any leakage from these tanks will also enter the house.

Finally tanks can be built into the basements of houses saving space and integrating the tank and house foundations. Laurie Baker has used this technique extensively in his design of The Centre for Development Studies in Karalla India albeit for non-drinking water purposes. If this strategy is to be used, account must be taken of the fact that even ground floor floors will have to be suspended and water will need to be pumped from the storage.

Optimal tank shape

Since the shape of a tank affects both the volume-to-surface ratio and the distribution of forces, it is worth examining the effect of tank shape upon material requirement. As always these material savings must be balanced against the additional complexity in manufacture, straight sides are much easier to form than curves and can be produced with a larger selection of materials. Table 2 shows a summary of tank shape from the viewpoint of induced stress, material use and construction. Three common tank shapes are considered: the cuboid, the cylinder and the doubly curved 'Thai' water jar shape.

Table 2: Relative merits of some common tank shapes

Tank shape or type		Stresses	Material usage	Construction
Cuboid		Stresses are unevenly distributed around the structure. Bending stresses are especially high near the edges.	The ratio between material usage and storage capacity is higher than for a cylindrical or doubly curved tank.	Construction is quite simple using most materials
Cylindrical		Stresses are more evenly distributed with bending stresses only near the bottom	There is an improvement in the material use to storage capacity ratio (a saving of 7.5% over a similarly proportioned cuboid)	Construction becomes more difficult with some materials e.g. bricks, but the shape is well suited to construction with materials that can be bent e.g. ferrocement and GI sheeting or built in sections
Doubly curved tanks (e.g. Thai Jar)		Stresses are well distributed. The base of the tank is of smaller diameter, reducing both hoop stresses and bending stresses there.	Material usage to capacity ratio is very good (savings of up to 20% over a cuboid)	Construction can be difficult, often relying on specialised moulds. Materials must be pliable and able to curve in two directions e.g. ferrocement and clay

The table shows that the cuboid shape fares relatively badly in terms of material use versus storage capacity and it is also associated with high peak stresses. The cylindrical shape deals quite well with stresses in comparison, and it has a lower (better) ratio of walling material to storage volume. It is still easy to manufacture, a technique well suited to circular or irregular forms. The ‘Thai’ style tank has the ideal shape to cope with the main induced stresses but requires greater skill and tooling to make.

Underground tanks

Significant material savings can be made if the tank is built underground. If the soil is suitable it can take the weight of the water and the walls can be made considerably thinner as they will simply be used as a waterproof layer stopping the water seeping into the soil. The geometry can also be very efficient (hemispherical) as the ground will act as a former for construction and the tank needn’t stand up on its base like an above ground tank. This material advantage should be balanced against the additional cost of digging a hole (which can be significant if the ground is particularly hard) and the possibility of the tank becoming contaminated by leaks or a rising water table floating the tank out of the ground.

3.3 Minimising labour and equipment costs

Generally labour costs rise as equipment costs are reduced and *vice versa*, so one should seek to get the best balance. What that balance is clearly depends upon location. In rural Africa digging deep pits by hand is cheapest, in urban Asia it would often be possible and cheaper to hire a back-hoe for the job. Similarly the transport of 'centrally' produced tanks or jars by motor vehicle might incur only modest costs in an urban area but quite excessive ones in a hilly rural area with few roads.

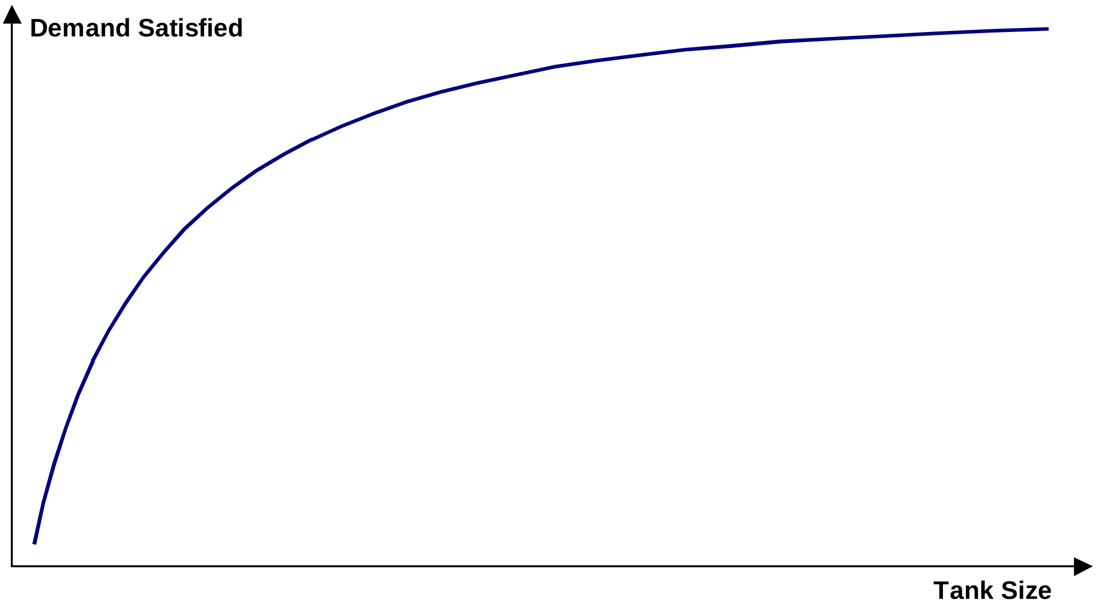
Employing moulds to build tanks improves their accuracy (and thereby may save materials) and also saves labour by reducing 'setting out' times. Moulds themselves become cheaper (per tank) if they are durable and can be used many times. Recycling moulds however requires suitable organisation – such as their attachment to a multi-system programme or availability for hire – and easy transportability.

In many cases, organisations assisting the construction of RWH systems have as their major objective the creation of income-earning opportunities in a specific locality. In consequence they seek to minimise the capital requirement associated with use of complex or motorised equipment, and to maximise the labour fraction of total costs. This approach may or may not minimise overall RWH tank costs.

3.4 Appropriate sizing

One of the simplest ways to make a tank cheaper is simply to make it smaller. Figure 1 shows the water demand satisfied by a tank compared to its size. As can be seen economics of tanks are such that the benefit of a tank is not strictly proportional to its size. The reason for this is that a smaller tank will be filled and emptied (cycled) often whereas a larger tank will only be cycled rarely. A fuller discussion of this is found in section 5

Figure 1: Benefits of tank sizing



Thus, while a large tank may be beyond the affordability of a household, a smaller tank will usually provide significant time-savings, particularly during the rainy season when footing can be wet and slippery. Another tank may also be added later and the total system capacity grown this way. This modular approach has can be seen in many parts of Southeast Asia.

3.5 Mass production

Tanks (particularly smaller tanks) benefit from the economies of scale that come with mass-production. In Thailand, a country that has undergone a massive rainwater harvesting promotion programme, 400 litre jars are produced for less than $US5. Another possibility is the mass production of some parts portion of a tank such a the lining and this way reducing the overall complexity of tank construction and allowing local materials and skills to fill in the balance.

4 STRUCTURAL STRENGTH & STRUCTURAL FAILURE OF TANKS

In a famous case over 150 years ago, the engineer Stephenson (who had no means of analysing structures) designed an early railway bridge by a careful series of experiments on a scale model. He built the model rather weak then loaded it until it collapsed. He then strengthened the part that broke first and repeated the procedure several times. By the end of the process the bridge had increased in weight by 50% but was 6 times stronger. We might use this technique to develop more materials-efficient tanks. Unfortunately the process would be long, expensive and perhaps unreliable (since testing long-term durability is harder for tanks than for steel bridges). Moreover today we have a much better (although incomplete) understanding of theory behind the structure of a tank.

4.1 Failure modes

There are number of possible modes of structural failure for a water jar or cistern, of which the most common are:

- Cracking and leaking (which may be temporarily repairable but often later progresses to failure)
- Leaning over, due to inadequate foundations
- Bursting (which can be dangerous, with fragments flying some distance)

It is a normal pattern that when a product like a RWH tank is first introduced to a new location a very ‘conservative’ design is used. It is in consequence expensive and may need subsidy. It should be the aim of any such programme to use the period of subsidy to simplify or cheapen the product by some degree of trial and error. Unfortunately failures resulting from a practical search for the design limits are embarrassing and can lead to mistrust of the product. For this reason it is prudent to perform such experiments in private (‘behind a hedge’) at least until the probability of failure appears low. Moreover the design should be chosen to exhibit functional failure such as leaking before any dangerous failure such as bursting.

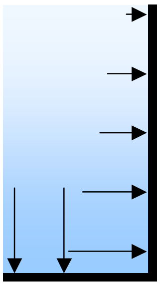
4.2 Stresses due to the stored water

Pressure forces

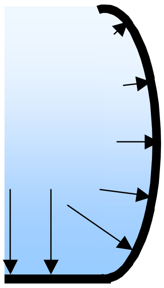
Water exerts a pressure proportional to its depth equivalent to 10 kPa. Per meter. The pressure always applies a force perpendicular to the inside surface of the tank, so at the bottom it acts downwards, over most of the walls it acts outwards and near the top of a doubly curved tank it can even act upwards (see Figure 2)

Figure 2: Action of pressure in a water tank

a. Straight sided



b. Doubly curved



Generally this pressure puts the tank walls into tension (stretch). This is unfortunate because many materials traditionally used for building and transferred to tank construction are only 10% or 20% as strong in tension as they are in compression.

Stresses in cylindrical tanks

In the case of a simple cylinder, the tensile stress acts around the cylinder and is called “hoop stress”. This stress can be found using the equation:

$$\sigma_h = \frac{p\,r}{t} \qquad (1)$$

Where:

σ_h is the hoop stress

p is the water pressure

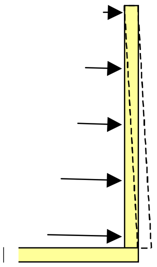
r is the tank radius

t is the wall thickness

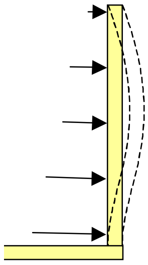
This simple result however is only true when the walls of the tank are free to move as shown in Figure 3a. The movement is only very small and can be achieved by using a flexible material between floor and wall such as bitumen or by allowing the wall to slide along the floor. Where the walls are fixed, such as at the base of a tank, they will tend to bow out as shown in Figure 3b

Figure 3: Movement of tank walls due to pressure

a. Unconstrained



a. Constrained



This will change the hoop stress and also cause two other stresses acting in different directions as are shown in Figure 4.

Figure 4: Stresses caused by constrained walls

a. Shear Stress

b. Bending stress

The wall will be stressed in shear at its edge where the water pressure forces it outwards but the base opposes this: the shear stress acts through the wall in a horizontal plane (Figure 4a). Another stress is due to bending of the tank walls as they bow outwards. This is especially high near the joint and will cause vertical compression of its outside face and tension on the inside face of a tank (Figure 4b) both acting vertically up the wall which can cause cracking of the inside face leading to failure.

Quantifying this situation is rather more complex and uses the technique of shell theory where the tank walls are idealised as being very thin (like egg shells). The theory also breaks the problem down into two parts. The first part considers the wall to be very flexible and therefore incapable of being stressed in bending or shear. The second part looks at the bending only and is confined to the boundaries between the wall and the floor where these forces are most prevalent. Furthermore the tank is considered to be made of a material whose properties are constant throughout and which will deform in direct proportion to the forces acting on it (Hooke's law). The relevant equations (Flugge, 1967) are:

$$N_\theta = \gamma r\left(h - x - he^{\frac{-\lambda x}{r}}\cos\frac{\lambda x}{r} + \left(\frac{r}{\lambda} - h\right)e^{\frac{-\lambda x}{r}}\sin\frac{\lambda x}{r}\right) \tag{2}$$

$$M_x = -\frac{\gamma r t}{\sqrt{12\left(1-v^2\right)}}\left(\left(\frac{r}{\lambda} - h\right)e^{\frac{-\lambda x}{r}}\cos\frac{\lambda x}{r} + he^{\frac{-\lambda x}{r}}\sin\frac{\lambda x}{r}\right) \tag{3}$$

$$Q_x = \frac{\gamma t\lambda}{\sqrt{12\left(1-v^2\right)}}\left(\left(\frac{r}{\lambda} - 2h\right)e^{\frac{-\lambda x}{r}}\cos\frac{\lambda x}{r} + \frac{r}{\lambda}e^{\frac{-\lambda x}{r}}\sin\frac{\lambda x}{r}\right) \tag{4}$$

where:

N_θ is the radial hoop force

M_x is the bending moment

Q_x is the shear force

γ is the specific weight of water (density times gravity)

r is the radius

h is the height water height

x is the height of the stress to be calculated

v is poisons ratio (the ratio of a materials change in shape in the direction of a stress to the change in shape perpendicular to the stress – as a rubber band is stretched it gets thinner)

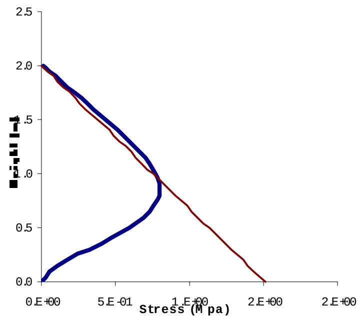
λ is given by

$$\sqrt[4]{3\left(1-v^2\right)\left(\frac{r}{t}\right)^2} \qquad (5)$$

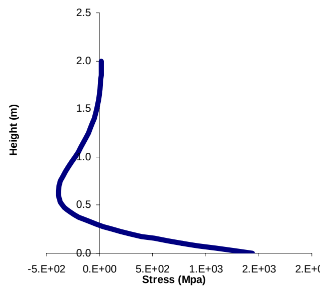
These fairly daunting equations can be easily coded into a spreadsheet and used to provide useful curves for designing tanks. Typical output is shown in Figure 5

Figure 5: Stress curves for cylindrical tank with fixed base

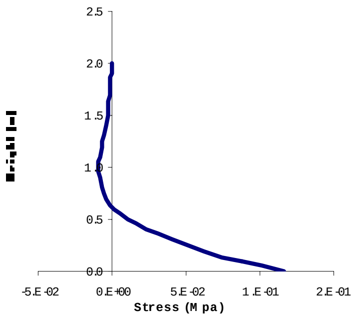
a. Hoop stress



b. Bending stress



c. Shear stress



Design considerations

Design to resist tensile stresses, whether vertical due to bending, radial hoop stresses and shear stresses acting through the wall, obviously includes the use of adequate wall thickness and adequate material tensile strength. It is sometimes assumed that brick and concrete cannot carry *any* tensile stresses without failing, but this is not so. Although it may be prudent to include steel reinforcing in a tank wall to prevent dangerous bursting, it is uneconomic to include so much that the mortar or brick carries negligible tension. If one relied solely on the stiffness of the reinforcing, its movement under water-pressure would almost certainly result in cracking of the mortar or brick matrix, causing perhaps leakage and almost certainly rusting of the reinforcing. Reinforcement that passes from a base into the bottom of a tank's walls will help resist both shear and bending stresses. Extra wall thickness, or filleting at the joint between wall and base is also a useful strategy.

4.3 Stresses due to materials shrinkage

As well as the cyclic stresses due to the water rising and falling inside the tank, there are other stresses due to the material *shrinking* during construction. Cement mortar, concrete and stabilised soil all shrink slightly (about 1 part in 1000) as they cure or dry out. If mortar or other such material is restrained during curing - for example by a metal mould or by attachment to a base - there will be a struggle between the mortar trying to shrink and the constraint trying to stop it. In consequence large stresses can develop in the mortar causing it to crack. To reduce such shrinkage cracking, we could:

- Remove stiff constraints (e.g. putting lime mortar or a rubber strip between the wall bottom and the base plate or using flexible moulds);
- Reduce mortar shrinkage by using a very dry mortar mix or a low cement content: unfortunately the former makes the mix difficult to work and the latter reduces strength;
- In the case of soil, use a low clay content;
- Spread such cracks that form (so that there are many tiny cracks rather than one big one) by putting a metal or fibre mesh inside the mortar: this is particularly helpful in controlling leakage since splitting one wide crack into two narrow ones will reduce leakage flow about 4-fold;

- Choose a shape (e.g. a sphere) where there are no hard attachment constraints;
- Modify the material by the addition of a slightly expansive component that counteracts the normal shrinkage;
- In the case of a render applied only to achieve water-tightness, it is often possible to apply it in two layers and place a sealant such as cement-plus-water between the layers.

5 ECONOMICS OF TANK SIZING

5.1 Economic overview

All households already have some access to water from point sources. For some days per year, many also employ ‘informal’ rainwater harvesting, placing bowls and jugs under eaves or even trees during rainfall.

The introduction of more formal (and productive) RWH will normally be accompanied by three benefits. The most obvious is a reduction in the time spent carrying water from point sources – a reduction more or less proportional to the volume of water no longer carried. The second is an increase in household water consumption wherever it was previously constrained by the effort of collection. The third is a common, although not invariable, increase in water quality. All these benefits rise with DRWH storage capacity, albeit in a way showing diminishing returns.

The increase in water consumption with RWH has not been widely measured. Generally any increase is restricted to the wet seasons. DRWH is not generally capable in the dry seasons of supplying quantities larger than already obtained from point sources: this means that it will be used to supplement, but not to substitute point-source water.

The costs of DRWH are overwhelmingly capital costs, as neither operation nor maintenance usually involves significant expenditure. These capital costs are subject to economies of scale. The sensitivity (elasticity) of tank cost to storage capacity is about 0.6

5.2 Value of water

As with many other goods, water has a declining value with quantity. The first litre per day is worth more than the tenth. By examining the limited data available that relates household consumption per day to the effective unit cost of water (i.e. cost per litre), we might construct a curve such as shown in Figure 6. Each socio-economic group would have its own curve.

The cost line on 6 is horizontal, which reasonably represents the situation where water is fetched, each successive litre requiring the same input of labour. Such a line does not fairly represent harvested roofwater, where the effective cost general rises with daily consumption despite the economies of scale in tank construction. A typical cost v volume characteristic for RWH supply is shown in Figure 7.

Figure 6: value vs. quantity

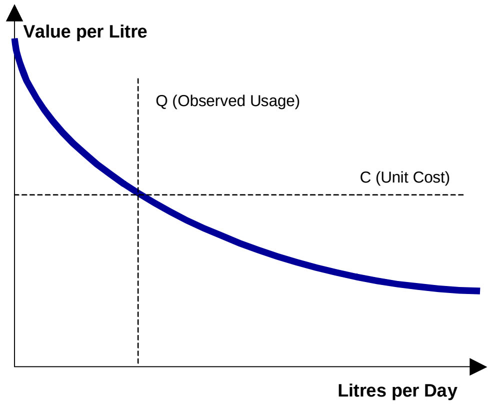
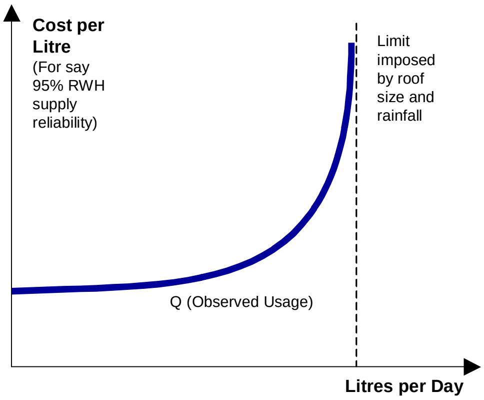


Figure 7: cost vs. volume



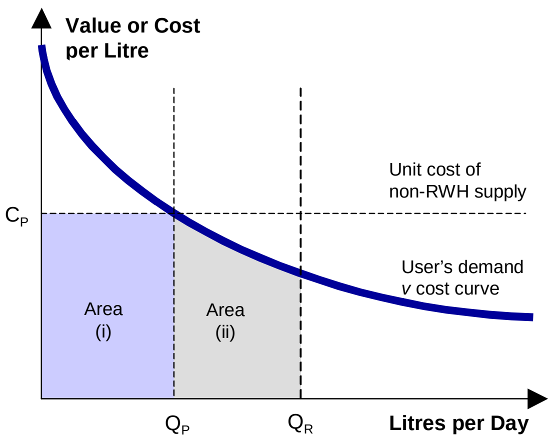
Sometimes we can find examples of water purchase and use them to infer the value of water. Richer households, or those experiencing illness, may pay for water to be brought to the house. More usually we have to infer costs indirectly through conversion of fetching distance/height into time and then time into money. Such costs, like the value of water discussed above, will be lower for poorer households than for richer ones.

5.3 Combining RWH with other water sources

For a given size and location of RWH system and for a given operating strategy, there will be a limit on the water it can supply per day, per week or per year. The maximum per year, corresponding to zero tank overflow, in litres will be the product of roof area (m^2), the annual rainfall (mm) and a run-off / capture factor (typically 0.85).

Consider first the situation where we can disregard seasonal factors, and assume that before RWH arrived, daily consumption from a point source was Q_P (litres/day). Q_P is determined by the interaction of the user's demand (cost v volume) curve and the unit cost C_P of supply from the point source. The daily cost to the user was therefore Q_P x C_P.

Figure 8: Value of rainwater



If the water Q_R available per day from RWH is *less* than Q_P, then the users will draw Q_R from the RW system and the remainder Q_P-Q_R from the point source. The total consumption will not increase and the effective value of the harvested rainwater will be the saving Q_R x C_P .

If the water Q_R available per day from RWH is *more* than Q_P, then the users will increase their consumption from Q_P to Q_R and the rainwater will be worth more than the former total cost Q_P x C_P. Exactly how much more will depend on the user's demand curve. The situation is represented in the diagram below, where Area (i) is the saving (Q_P x C_P) while Area (ii) is the value of the extra water.

Note that Q_R is the daily amount *available* from RWH, whereas Q_P is determined by the price of supply (from non-RWH sources). The total value Area(i) + Area(ii)) is less than (Q_R x C_P) because the extra water is per litre less valuable to the user than the water 'replaced'.

5.4 Seasonal effects and water management strategies

In the last section we ignored seasonal effects, although one can identify the condition $Q_R<Q_P$ as representing a dry season and $Q_R>Q_P$ as representing a wet one. However seasonality is central to the operation and performance of a RWH system. A user can choose to emphasise dry season security or alternatively to emphasise roofwater capture. To some extent the dry and wet season water needs are in competition with each other. Consider the following four water management strategies for an already built RWH system.

To make the strategies easier to visualise, assume a scenario typical of a homestead in the Great Lakes region where mean daily roofwater runoff is R = 100 litres). Assume that 'dry' weeks (runoff less than 350 litres per week) comprise 1/3 of each year and that the RW storage capacity is 700 litres (7 x R or '1 week'). This storage is only modest, but corresponds to perhaps 50 days drinking water or 14 days total water under very careful management.

Strategy 1 – *High Water Capture* – Water is withdrawn at a high rate, Q = 1.5 R, (e.g. 150 litres/day under our scenario) whenever it is available. This will result in fairly low occurrence of tank overflow, but leave little reserve for dry weeks.

Strategy 2 – *High Security* – Water is withdrawn at a low rate, Q = 0.5 R, (e.g. 50 litre/day) whenever it is available. Much water will overflow the tank, so annual capture will be low.

Strategy 3 – *Adaptive* – Water is withdrawn at a rate Q determined by how much is in the tank, thus:

Q = 1.5 R (e.g. at 150 lpd) if tank > 2/3 full;

Q = R if tank < 2/3 but >1/3 full;

Q = 0.5 R, if tank < 1/3 full.

Strategy 4 – *Maximum Security* – Water is saved for the dry seasons and drawn frugally (e.g. 50 litres/day) *only* after nearby point sources have run dry or after 2 weeks without rain.

The trade-offs involved between these alternatives are summarised in the following table, in which the word 'security' is taken to mean the fraction of days the demand is met by RW (the tank does not run dry). The factor K is the dry-season value of water (valued at its cost from the nearest point source) divided by its wet season value. Thus K=1 represents places where point-source water is unvarying through the year, whereas the extreme value K=10 represents places where in the dry months all local sources dry up, so water must be queued for, then carried from, very far away. A typical value of K in the humid tropics might be 2.

Table 3 suggests how we might account for seasonal differences in our economic evaluation, namely by assigning different wet and dry season values for water and operating the system to maximise their sum.

Table 3:System Performance under Different Operating Strategies

Strategy No	Annual consumption	Relative value of annual water harvested		Wet season security	Dry season security
		if K=1	if K=10		
1	high	high	med	high	v. low
2	low	med	med	high	low
3	medium	low	v. low	high	low
4	very low	med	med	nil	med

Table 4 represent the simulation of the four strategies applied to respectively a small DRWH system (storage volume V = 7 x mean daily run-off, R), a medium size system (V/R = 21) and a large system (V/R = 63). Daily rainfall data for 10 years has been used and a roof area of 45 m^2 has been selected to give the assumed mean run-off R = 100 litres/day. For Mbarara, the town used, the dry season (defined by rain in the last fortnight being under 50% of mean fortnightly rainfall) is 36% of the year.

Table 4: Relating RWH system performance to operating strategy and storage volume

Strategy number / type	V / R	Capture Efficiency	Tank Utilisation	Mean daily consumption Q in litres		'Security' (*S*) = fraction of days demand is satisfied by roofwater		
	tank size days	*E*	*U*	Q_1 *K*=1	Q_5 *K*=5	S_w Wet	S_d Dry	All year
Small tank,								
1 High demand High capture	7	**0.70**[1]	36.5	70	**95**	0.75	**0.22**	0.56
2 Low demand High security	7	**0.41**[3]	21.4	41	**80**	na	**na**	na
3 Adaptive	7	**0.66**[2]	34.4	66	**93**	0.94	**0.38**	0.74
4 Max security in dry seas	7	**0.17**[4]	8.9	17	**84**	na	**0.52**	na
Medium size tank								
1	21	**0.91**	15.8	91	**125**	0.90	**0.25**	0.67
2	21	**0.47**	8.2	47	**107**	na	**na**	na
3	21	**0.86**	14.9	86	**138**	1.00	**0.66**	0.88
4	21	**0.26**	4.5	26	**128**	na	**0.73**	na
Large tank								
1	63	**1.00**[1=]	5.8	100	**165**	0.92	**0.38**	0.72
2	63	**0.51**[3]	3.0	51	**123**	na	**na**	na
3	63	**0.99**[1=]	5.7	99	**203**	1.00	**0.98**	0.99
4	63	**0.37**[4]	2.1	37	**182**	na	**1.00**	na

Notes:
1. Data is for Mbarara, Uganda
2. Annual run-off = annual demand
3. na indicates strategy does not allow demand to be met.
4. Highlighted cells indicate best strategy or within 3% of best
5. Strategy 1 gives best $\mathbf{Q_1}$ (highest water capture)
6. Strategy 3 gives best $\mathbf{Q_5}$ (highest benefit if K = 5)
7. Strategy 4 gives best $\mathbf{S_d}$ (highest dry season security)
8. Strategy 3 is always best or second best by all measures.

‘Value’ is calculated assuming first litre per day is worth 1.5 falling via 0.5 at the 100^{th} litre to zero at the 150^{th} litre

Strategy 1 is to withdraw 1.5 times base demand when available (and otherwise what is available)

Strategy 2 is to withdraw 0.5 times base demand when available (and otherwise what is available)

Strategy 3 is to withdraw 1.5, 1 or 0.5 times base demand, according to amount in tank

Strategy 4 is to withdraw nothing in wet season and in dry season base demand when available (and otherwise what is available).As well as water supplied (column 5), a ‘weighted’ water supplied column is shown alongside in which effectively K = 5. This yields the weighting (a ‘wet season litre’ is a cost-equivalent volume): 1.0 dry season litre is deemed to be worth 5.0 ‘wet season litres’

The **bold** columns in the table contain the performance measures of most interest.

Column 3 shows ‘Capture efficiency’, (E) – a high value indicates that most of the roof run-off is being consumed.

Column 8 shows ‘Dry season water security’, (S_d) – the fraction of dry season that tank does not run dry and so demand has been satisfied; note however that under Strategy 1 the dry season demand is maintained very high at 1.5 R, whereas the other strategies are using demand of only 0.5 R for the dry season.

Column 6 shows weighted annual water consumption, Q_5, which is a measure that attempts to combine quantity, and security measures, by valuing wet season water much more highly than dry season water.

Examination of the top part of the table – which is for a small system with V/R only equal to 7 days – indicates that Strategy 1 (in which water is drawn generously whenever available) gives the highest annual water yield E, the lowest level of dry season security S_d, yet a high value for the seasonally-weighted yield W_e .

By contrast Strategy 4 (water is drawn sparingly and only in the dry season) gives the highest dry season security at the cost of the lowest annual yield. The seasonally-weighted yield is however also low. In fact we can dismiss Strategy 4 because even here, where per litre we have valued dry season water at *five times* wet season water, it still gives the lowest output valuation.

Strategies 2 and 3 are intermediate in performance, with Strategy 3 (adaptable) generally outperforming Strategy 2 (fixed low-demand).

From this table we can conclude that unless dry season water has exceptional value – e.g. it is per litre worth more than the 5 times wet season water assumed in the table – Strategies 1 (high usage) and 3 (adaptive) are superior to the other strategies.

The bottom band of the table is for a much more expensive system with 9 times larger storage. With such a large tank, the relative superiority of Strategy 3 is increased. We also see the benefit of the larger store. Comparing say Strategy 3 for the very large tank with that for the small one, we find a 50% increase in water harvested (E), a nearly 4-fold increase in dry season security (S_d) and under the assumed value ratio (K=5) a 120% increase in water value. Table 5 shows the variation in value of water harvested for varying values of K and for various sizes of tank. It confirms that small systems (V/R < 10 days) give a generally acceptable performance unless dry season water is deemed very much more valuable (e.g. K=5) than dry season water. Note the clear ‘diminishing returns’ with increase in tank size. If water value had been plotted against tank *cost* rather than tank *size*, the same pattern of diminishing returns would appear but with a slightly reduced strength.

A small system in the humid tropics, attached to a $50m^2$ roof, might be expected to harvest around 25,000 litres of water per year (say 75% of run-off), averaging about 90 litres per day in the wettest 8 months and 30 litres per day in the driest 4 months.

Table 5**:** Performance under Strategy 3 – Table showing variation of value ratio, capture efficiency and security with tank size

	dry:wet value per litre	Normalised tank size – V/R in days								
		1	3	5	7	14	21	30	60	90
Benefit ratio = value of water harvested ÷ value water demanded	if K=1	0.29	0.49	0.60	**0.66**	0.79	0.86	0.91	0.98	1.00
	if K=2	0.24	0.40	0.49	**0.53**	0.65	0.72	0.79	0.90	0.94
	if K=5	0.18	0.30	0.35	**0.38**	0.48	0.56	0.65	0.82	0.88
Capture efficiency		0.39	0.66	0.76	**0.82**	0.90	0.93	0.95	0.99	1.00
Security		0.15	0.41	0.57	**0.67**	0.81	0.86	0.90	0.98	1.00

Notes:
1. Under this strategy the demand is varied from 0.5 to 1.5 times the mean daily runoff according to how much water remains in the tank
2. V/R is tank size (normalised to mean daily run-off); K is dry-to-wet season water value ratio; the bold column shows the performance of a typical very-low-cost RWH system

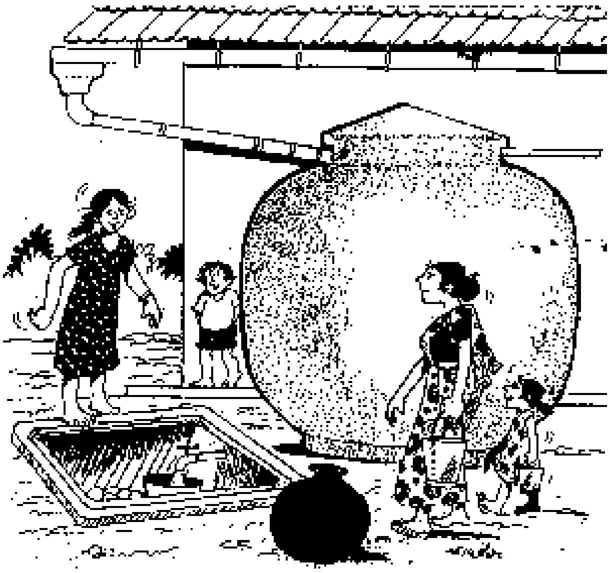
6 ANALYSIS OF EXISTING TANK DESIGNS

In the course of the project a number of tank designs have been investigated by the RHRG. These have been written up as "case studies" and are available on the project web site at:

www.eng.warwick.ac.uk/dtu/rainwaterharvesting/casestudies.htm.

A summary of these designs is set out below

6.1 The Pumpkin Tank, Sri Lanka



The Sri Lankan Pumpkin Tank, and the associated construction technique, was developed as part of a World Bank sponsored Water and Sanitation Programme that was implemented in the country between 1995 and 1998. The Community Water Supply and Sanitation Programme (CWSSP) covered 3 districts within the country – Badulla, Ratnapura and Matara Districts. Hundreds of these tanks have been built in areas where conventional supply schemes, such as piped supplies or groundwater supplies, were difficult to provide.

Catchment (typical) – $32m^2$

Storage – 5 – 7 m^3

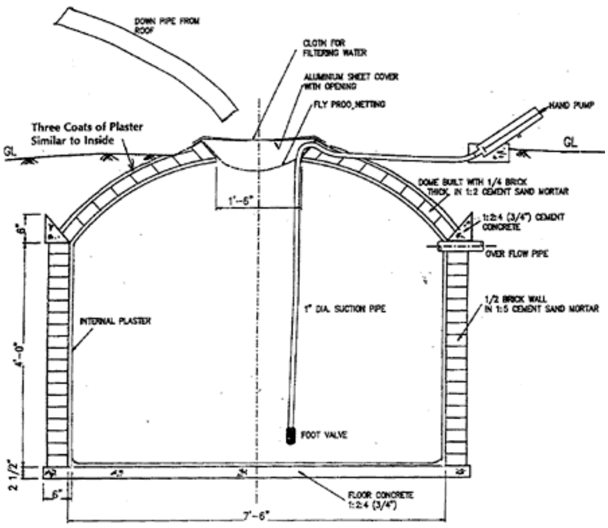
Storage cost - £112 (skilled labour - £19, Unskilled, £21 unskilled)

Material - ferrocement

Lessons

- Doubly curved structures
  - Material economies
  - Specialised techniques needed
  - Ferrocement construction
  - Use of mould for many tanks

6.2 Underground brick dome tank, Sri Lanka



This is another RWH system which was developed by the CWSSP programme in Sri Lanka. The tank, a $5m^3$ underground brick built tank.

Catchment – $28m^2$

Storage – $5m^3$

Storage cost - £125 (skilled labour - £15, Unskilled, £28 unskilled)

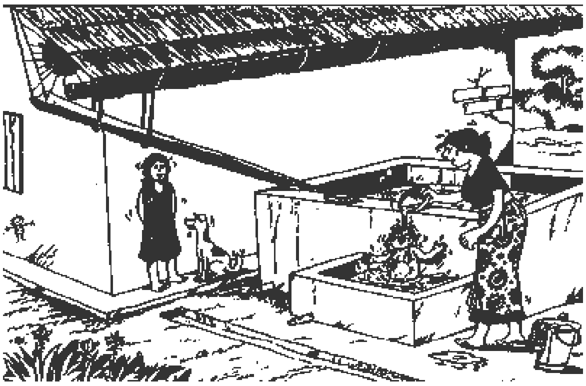
Material - Brick

Lessons

- Brick tank construction
  - Less skilled
  - *but* used as much cement as pumpkin tank

- Brick dome roof
- Low cost pumps for water extraction

6.3 Brick built storage tank, Sri Lanka



This is an example of local initiative in design and manufacture in DRWH. The tank in question was constructed in the village of Ahaspokuna, near Kandy, in the highlands area of Sri Lanka. The tank was built 10 years ago by a local mason for the Rajasomasari family and has since been copied so that there are now several of these tanks in the area.

Catchment – 90m^2

Storage – 3m^3

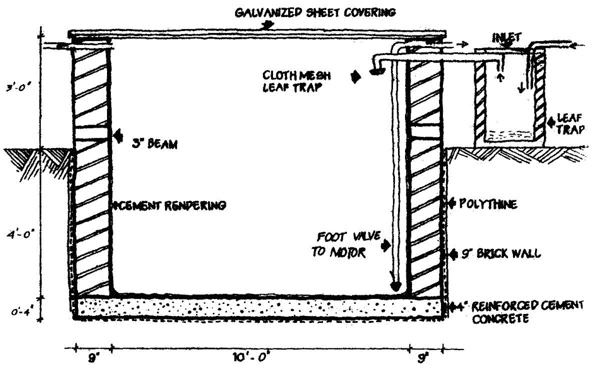
Storage cost - £80 (est.)

Material – Brick with cement render

Lessons

- Square construction
  – Good for bricks
  – Simple for local labour

6.4 Partially below ground brick built tank, Sri Lanka



This tank was built by Mr G. Victor A. Goonetilleke in the hill town of Kandy,. Mr Goonetilleke decided to build his RWH tank after experiencing difficulty in sinking a well to sufficient depth to have a reliable perennial source of groundwater at the site of his newly built home.

Storage – 10m^3

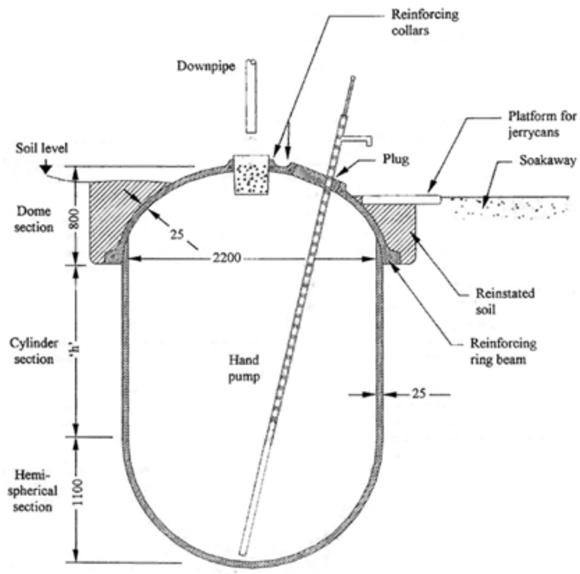
Storage cost - £550

Material – Brick with cement render

Lessons

3 Partially below ground construction – many of the advantages of below ground construction but with several of the advantages of above ground construction

6.5 Underground storage cistern – 4 to 10 cubic metres, Uganda



This tank (or cistern) was developed in Uganda by members of the Development Technology Unit, Warwick University and members of the Uganda Rural Development and Training Programme (URDT), between 1995 and 1997. Work is still continuing on the refinement of the tank. URDT is a service NGO located at Kagadi in Mid-Western Uganda. Several of these cisterns were built and tested with the aim of developing a low cost (under US$150), alley, domestic, water storage technology for the surrounding region.

Catchment – Varying

Storage – 4-10m^3

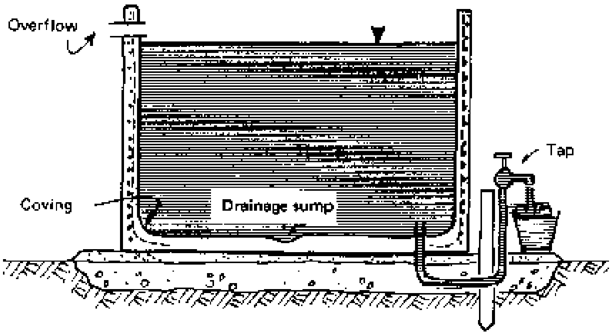
Storage cost – £90 (8m^3)

Material – Cement Mortar

Lessons

- Underground tanks – very thin walls are possible in appropriate soil
- Unreinforced mortar dome roof – lower cost due to no steel
- Ground as formwork – reduced cost amortisation of formwork

6.6 Ferrocement water tank using former



Used for many years in parts of Africa these tanks have been designed to be as simple as possible to build in self-help programmes. The users, who are at first unskilled in this sort of construction, can contribute their time and efforts in collecting sand and water, digging the foundations and preparing the mortar under the general guidance of a trained builder. With experience they quickly learn how to make the tanks without further guidance. A trained builder with 5 helpers takes approximately 3 days to complete the tank. The users often contribute some money towards the cost of the tank, which helps to cover the builders' wages, the cement, reinforcement and the hire of the corrugated iron formwork

Catchment – Varying

Storage – 10m^3

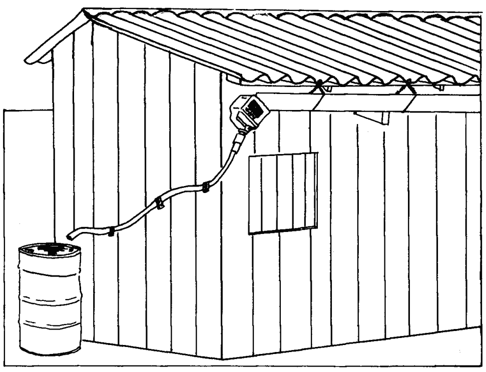
Storage cost – £90 (8m^3)

Material – Ferrocement

Lessons

- Reusable formwork with built-in depth gauge (corrugations)
- Adapted from successful commercial design from New Zealand
- Wire reinforcement graded through structure (more on bottom)
- Reinforcing from base to walls and filleted base join to avoid cracking at base

6.7 RWH in the barrios of Tegucigalpa, Honduras



Health statistics show that the residents of the barrios are suffering from a number of water related diseases that could easily be avoided with provision of a reliable, clean water supply. Unfortunately, more than 150,000 residents have to find their own water. Although technically unsophisticated and lacking some good health practice, the systems show what urban settlement have done to improve their own lot. Many of the systems make use of recycled or scavenged materials and some examples show high levels of initiative.

Storage – $0.2m^3$ used barrels (up to 3) or $1\text{-}2m^3$ open concrete tanks

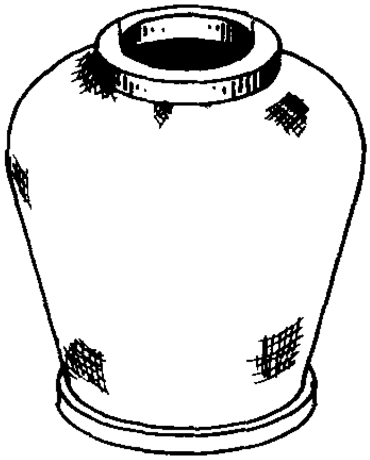
Storage cost – £10 (drum) - £18 “small tank

Material – Steel drum or plastered bricks

Lessons

- Impact of very small storage
- Use of available containers

6.8 Tai Jar



This type of water vessel was originally developed for a large country wide programme in Thailand that has installed over 10 million jars. Small jar making became a successful business with many companies producing up to 30 jars a day. The design has also been adopted in East Africa and South Asia

Storage – $0.5\text{-}2m^3$

Storage cost – £ 30 for a $2m^3$ jar

Material – Pottery or cement

Lessons

- Small tanks used modularly – several houses have more than one and up to 10 are used commercially
- Mass production – lead to much cheaper tanks
- Critical mass – the programme grew exponentially and became self sustaining within a few years

6.9 Plastic Lined tanks

Several experiments have been done using plastic sheets as a waterproof membrane in an otherwise wholly traditional structure. The bamboo tank developed by ARTI in Pune India uses a polythene sheet in a basket structure that is traditionally used for grain storage. In East Africa Rwandan refugees have used a tarpaulin distributed by UNHCR as a liner for an underground tank over which a wattle and daub enclosure is built to protect the water.

Lessons

- Small, portable imported input bolstering principally local technique – greatly reduced cost
- Some questions of durability

6.10 Summary

Table 6: Summary analysis of case study material

	Material costs	Percent Labour costs	Unit costs (per m^3)	Skills required	Equipment/ tools required	Space requirement	Suitability for incremental adoption	Reliability	Water quality, safety and health	Impact on insect breeding	Stage of maturity or experience
Pumpkin Tank	✓	✓	✓	✗	✗	O	✗	✓	✓	✓	✓
Underground brick dome tank	✓	✓	✓	O	✗	✓	✗	✓	✓	✓	✓
Brick built storage tank	O	O	O	✓	✓	O	O	✓	✗	✗	✓
Partially below ground brick built tank	✗	✗	✗	✓	✓	O	✗	✓	✓	O	O
Underground storage cistern	✓	✓	✓	✗	✓	✓	O	✓	✓	✓	O
Ferrocement water tank using former	✓	✓	✓	✓	✗	O	O	✓	O	O	✓
RWH in the barrios of Tegucigalpa	O	O	O	✓	✓	O	✓	O	✗	✗	O
Tai Jar	✓	✓	✓	O	✗	O	✓+	✓	✓	O	✓
Plastic Lined tanks	✓+	✓+	✓+	✓+	✓	O	✗	?	O	O	✗

Key ✓ Good O Medium ✗ Bad

7 RHRG RESEARCH INTO MEANS OF REDUCING TANK COSTS

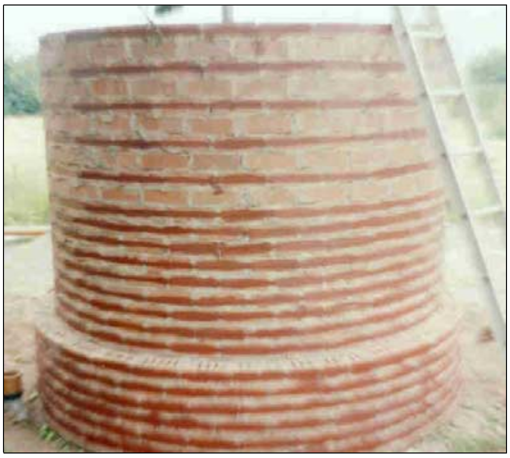
7.1 Research strategy

Having collected information on existing rainwater systems and identified probable areas for design improvement, the RHRG has developed several cheaper tanks, concentrating on materials substitution and material reduction while working mainly with smaller tanks suitable for incremental adoption. The main materials investigated have been:

- Rammed Earth
- Stabilised soil blocks
- Bricks
- Plastic linings

Various underground tanks have also been developed mainly on the partly below ground model. A lift-on tank cover has also been developed to remove the need for formwork from the cover making process.

7.2 Use of externally reinforced bricks



Brick is a material that is used widely in developing countries and is thus readily available. It is ideally suited to wall construction, but not quite so well suited to conventional larger volume tank construction. As it;

- has a poor strength in tension
- has a poor adhesion one brick to another through the mortar and so tensile forces must be spread through the brick runs which will be in sheer with one another
- can need more cement than an equivalent Ferrocement tank due to poorly fitting bricks

One way of improving the suitability of brick to low cost tank manufacture is by using external steel reinforcing to give additional hoop strength to cylindrical brick tanks. If an empty circular tank is wound with steel reinforcing wire on the outside, and the wire is then tightened, we will achieve ‘post tensioning’ whereby the masonry (brick/concrete) is initially in compression and the steel in tension. Putting the steel on the outside not only facilitates the post-tensioning but also makes it easier to protect the steel from being rusted by seepage from inside the tank. Filling the tank with water will result in a lowering of the compressive stress and strain in the mortar and an increase in the tensile stress and strain in the steel. One such tank has been built and was detailed in Milestone Report A2

The technique holds some promise as it is easy to implement, however there is some doubt over the availability of tensioning tape as described in the report. It should, however be possible to use galvanised wire for this purpose although more will be necessary due to its lower tensile strength.

Table 7: Pros and cons of brick tanks

Pros	Cons
• low material cost • suitable material readily accessible locally in many parts of the world • a well-known and widely-used technology in many parts of the world • a simple technology that is easily taught to semi-skilled people	• not ideal for round tanks as extra mortar or special angled bricks are needed • Poor in tension – Needs reinforcing or very thick walls

7.3 Use of rammed earth

The use of rammed earth has been the subject of a previous report to the EU (Milestone A5: *Stabilised Soil Tanks for Rainwater Storage*, submitted September 2000) so only a brief summary will be presented here.

Rammed earth is a technique which is experiencing a resurgence of interest, particularly in developing countries where cement is expensive and in "green" architecture where its low energy use and excellent thermal properties are particularly appreciated. The technique has been used for centuries for the construction of houses many of which are still standing, attesting to its stability and longevity.

Just as it's name suggests this technique involves earth being rammed between two shutters, using a rammer or tamp. The shuttering is removed to reveal the wall. Walls are usually constructed in sections of a few feet long by a foot or two deep with shuttering moved along to form a continuous wall. The shuttering is then raised and placed on top of the first 'lift' to construct the subsequent 'lifts'.

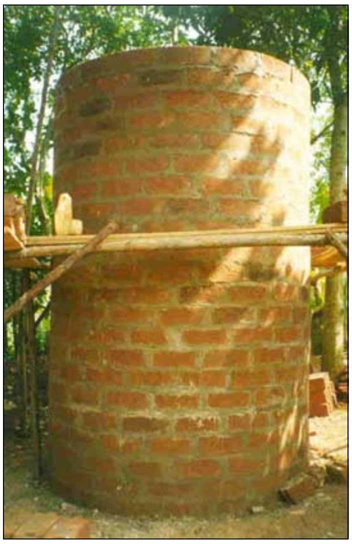
Several experiments with rammed Earth have been undertaken by the RHRG both in the lab and in the field. The material has proved capable of withstanding the forces typical of a tank however there are some practical problems. The pros and cons of Rammed earth tank construction are listed in table

Table 8: Pros and cons of rammed earth tanks

Pros	Cons
• very low material cost • suitable material readily accessible locally in many parts of the world • a well-known and widely-used technology in many parts of the world • a simple technology that is easily taught to semi-skilled people	• not suitable for below-ground tanks or cisterns • in the case of leaks serious problems can develop, especially if unstabilised earth is used • high labour input

The main problem with rammed earth is its wet strength. The tank must be fitted with a waterproof lining to hold the water. If this liner becomes damaged and *any* water leaks, then the tank will almost certainly fail as any water ingress will ultimately seep out along with some soil, this process is continued until a hole forms ultimately growing large enough to destroy the tank's structural integrity.

7.4 Use of stabilised-soil



Stabilised soil has also been covered in a previous report (Milestone A5: *Stabilised Soil Tanks for Rainwater Storage*, submitted September 2000)

Stabilised, compacted, soil block technology involves compacting a suitable soil, which is often mixed with a small percentage (typically 5 – 10%) of cement, using a manual or hydraulically assisted ram or press. This compaction reduces the voids in the material and hence it susceptibility to attack from water. Special moulds can be manufactured to produce blocks of different shapes for special purposes.

Work has been carried out on stabilised soil blocks for tank construction in two locations (Uganda and Sri Lanka) In the case of the cylindrical tanks manufactured in Uganda, curved blocks were produced. For a cylindrical above-ground tank. In Sri Lanka a square underground tank was produced.

The Uganda work involved the building of two experimental cylindrical water tanks in collaboration with Dr Moses Musaazi, a lecturer at Makerere University. Both were built above ground of curved stabilised-soil blocks with end interlocking, 280mm x 140mm x 110mm high, made with an Approtec

(Kenyan) manual block press. The soil used was a red somewhat pozzolanic local soil previously known to make strong blocks. The tanks were built on concrete plinths, lined with ‘waterproofed’ mortar (3 parts sand, 1 part cement and .02 parts ‘Leak Seal’ waterproofing compound). There was no metal reinforcing.

Tank 1 was 2m high, with an internal diameter 1.3m, wall thickness 140mm (+ 15mm render) and used blocks incorporating 6% cement. It has been filled with water and briefly withstood a maximum head of 2.m at the wall bottom before failing catastrophically (and spectacularly). 2, for test purposes, has been built to 5m high, has internal diameter 1.0m and the same wall thickness, but with only 3% cement. It has been filled with water and therefore withstood a head of 5.0m at the wall bottom. The inconsistency of the result could be attributed to an undersupply of cement to the blocks which made up tank one resulting in a low wet strength, coupled with a cracking of the mortar used to line the tank.

The pros and cons of stabilised soil construction tank construction are listed in table 7

Table 9: Pros and cons of rammed earth tanks

Pros	Cons
• Reduced cement content resulting in inexpensive blocks •	• Low wet strength • Reduced cement content must be balanced against lower strength requiring thicker walls • Needs specialised tooling for compacted blocks • Low tensile strength of block joints

7.5 Lining tanks with plastic bags

Plastic linings can considerably reduce the cost of the tank by removing the need for any building work to be watertight. Indeed they can be simply placed in a hole to form a very cheap and portable tank (although a cover should be constructed). Plastic liners also allow removal for inspection, cleaning, maintenance and occasional repair.

Work on plastic linings by the RHRG includes the development of a technique for welding 250 micron construction or damp proof membrane (DPM) plastic sheet to make ‘bags’ (similar to large bin liners) that fit inside the tank structure to form a waterproof lining. The welds themselves have proved successful with the weld both watertight and stronger than the plastic itself. The technique uses simple tools and can be taught in a couple of hours to a reasonably skilled craftsperson. One skilled person should be able to make a bag in a single day and productivity can be greatly enhanced by batch production. However, while the welding technique has been successfully developed, there are still problems to be overcome in relation to the quality of ‘off-the-shelf’ plastic sheet and failure of the lining due to abrasion. Observations in the field suggest that plastics with a woven structure may be more resilient, however these will not weld with the technique developed at Warwick.

The availability of plastic tubes from local markets is quite wide spread and a 600litre "plastic tube tank" was also developed in the course of the project. Details of this tank can be found in DTU Technical Release *TR-RWH08 Plastic Tube Tank (600 litres) – Instructions for manufacture*. (Rees & Whitehead, 2000b)The 87cms diameter tube is the largest that was found at the time but the design could be easily modified for different sizes of tube. The tank is based on the partly below ground concept and consists of a brick parapet wall, which incorporates the inlet basin and handpump and the lower section ground excavation, lined with two plastic liners one inside the other. The tanks have developed slow leaks reducing their effectiveness but are still holding water and making a significant contribution to household water where they have been installed.

Table 10: Pros and cons of plastic linings

Pros	Cons
• Greatly reduced cost • Portable • No clambering on or in the tank is required during construction. • No curing time is required. • Can be removed for cleaning./inspection • Can be batch produced	• Fragile – likely to tear, subject to pin holes • UV degradation • Joining requires specialised techniques

7.6 Simple underground tanks in stable ground

The RHRG has also experimented with creating underground tanks using stabilised soil with bamboo reinforcing and a plastic liner for waterproofing. The work (undertaken by the Open University in Sri Lanka) .The tank was designed to contain 3600 litres of water. It is trapezoidal with 100cm x 100cm square cross-section at the top and 80cm x 80cm square cross-section at the bottom of the tank. Strips of bamboo are used both as a support to the tank walls and to protect the lining material, which is 500 gauge polythene sheet. During the development soil samples were tested for strength and durability and a 1:12 cement:soil ratio was found to be optimum with the local soil. This will, however differ with location.

The problems of covering such pits can still cause cost problems absorbing more than 2/3 of the cost of such a tank. Tanks have also been broken due to raising water table, punctured by tree roots and are vulnerable to infiltration by runoff.

Table 11: Pros and cons of simple underground structures

Pros	Cons
• Greatly reduced cost as surrounding ground gives support allowing lower wall thickness. • More difficult to empty by leaving tap on • Can be made unobtrusive	• Water extraction is more problematic – often requiring a pump • Leaks or failures are more difficult to detect • Contamination of the tank from groundwater and surface runoff • Tree roots can damage the structure • There is danger to children and small animals if tank cover is left off • Flotation or breaking of the cistern can occur if groundwater level rises

7.7 Partly-below-ground tanks

Several of the problems of underground tanks can be overcome by siting the tank partly above ground and partly below ground. These tanks have been described in Milestone Report A3. Details can also be found in the DTU Technical release *TR-RWH 01 Partially Below Ground (PBG) Tank for Rainwater Storage – Instructions for Manufacture,* (Rees, 2000b) available at: the DTU website. Over 20 of these tanks have been built in East Africa and feedback suggests that the tanks are easy to construct by masons with some training, at a reasonable cost.

Table 12: Pros and cons of Partially below-ground tanks

Pros	Cons
• Lower material requirements • Difficult to empty by leaving tap on • Reasonably unobtrusive • Surrounding ground gives support allowing thinner walls and thus reduce costs	• Requires a pump • Leaks or failures are difficult to detect • Contamination of the tank from is possible • Tree roots can damage the structure • Flotation of the cistern may occur

7.8 Lift-on tank covers

The lift on tank cover developed by the RHRG has been detailed in Milestone Report A2 and in DTU Technical Release *TR-RWH 04 – Low-cost, thin-shell, 2m diameter ferrocement tank cover - Instructions for manufacture (Rees, 2000a).* The thin-shell ferrocement tank cover is designed in such a way that it can be manufactured without the use of a mould or shuttering. It can also be manufactured remote from the tank to which it is to be fitted and moved into place once complete. The aim is to reduce the cost of the tank (cover) by eliminating costly shuttering or moulds and by reducing the quantity of material used to manufacture the cover. It also means that the cover can be removed at a later date for maintenance, refurbishment or cleaning. The cover can be manufactured by two persons (one skilled and one unskilled) in a single day (with some time required after that for curing) using tools required for the construction of a simple cylindrical ferrocement tank.

The design is based on a frame known as a reciprocal frame, that has spokes that, when loaded, put little radial loading onto the structure on which it sits. The frame is covered with a wire mesh that is then rendered with a sand cement mix.

Table 13: Pros and cons of lift on lank covers tanks

Pros	Cons
• low cost – reduced use of materials • no shuttering or mould required • strong and lightweight – the tank cover is designed to be strong (through good quality control) and light at the same time • good quality control can be achieved through easy working environment • can be manufactured by two people in a single day (one skilled and one unskilled) • no clambering on top of tanks required during construction • can be cured easily – in the shade and at ground level • can be batch produced at one site	• Needs skilled craftspeople • Needs good quality control to be effective • Available in one-size-only as frame angles must be recalculated for other sizes

8 COSTS ANALYSIS OF RWH TANKS

The aim of this exercise is to compare tanks from different parts of the world and to carry out a costing exercise so as to assist those considering DRWH to make an informed choice. Such choices are usually complicated by the fact that material costs, labour costs, per capita income, currencies and exchange rates all vary from one country to another. Cost of storage per litre also varies as tank size increases. To take into account this variability an effort has been made to normalise some of the figures.

- 8 tanks have been costed for each of three countries i.e. Uganda, Sri Lanka and Brazil using bills of materials from the designs and material cost information obtained from each of the countries in 2000/2001.
- All tank costs have been converted to $5m^3$ equivalent using sensitivity to size of 0.6.

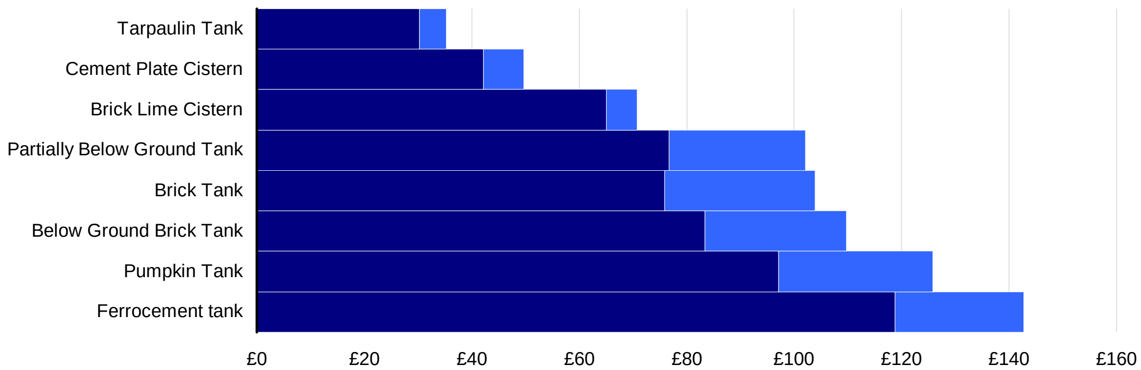
The tanks under consideration are taken from 4 countries in 3 continents. The countries are Kenya, Uganda (Africa), Sri Lanka (Asia) and Brazil (South America). The tanks are listed in Table 14 and the final costings are in Figure 9

Table 14: Tanks used for costing exercise

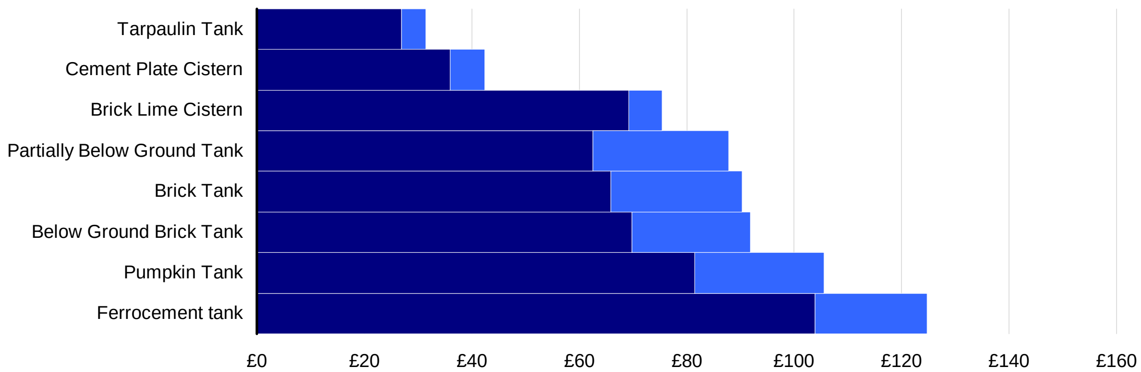
	Tank name	Tank size(s)	Source of information
1	PBG Tank	10,800 litres (size can be varied)	DTU Technical Release 01
2	Ferrocement tank	3,000 and 11,000 litres	Eric Nissen-Peterson, ASAL, Kenya (Nissen-Petersen & Lee, 1990)
3	Small brick jar	750 litres	DTU Technical Release 07 (Rees & Whitehead, 2000a)
4	Tarpaulin tank	4,000 – 5,000 litres	ACORD, Uganda and DTU Web Page
5	Ferrocement Pumpkin Tank	5,000 litres	Lanka RWH Forum and CWSSP Reports, Sri Lanka (Hapugoda, 1995)
6	Below ground brick tank	5,000 litres	Lanka RWH Forum and CWSSP Reports, Sri Lanka (Hapugoda, 1995)
7	Cement Plate Cistern	10,000 and 20,000 litres	Johann Gnadlinger, IRCSA. Data from Juazeiro, Bahia State, 1998 (Gnadlinger, 1999)
8	Brick lime cistern	10,000 and 20,000 litres	Johann Gnadlinger, IRCSA. Data from Juazeiro, Bahia State, 1998 (Gnadlinger, 1999)

Figure 9: tank costs in three countries

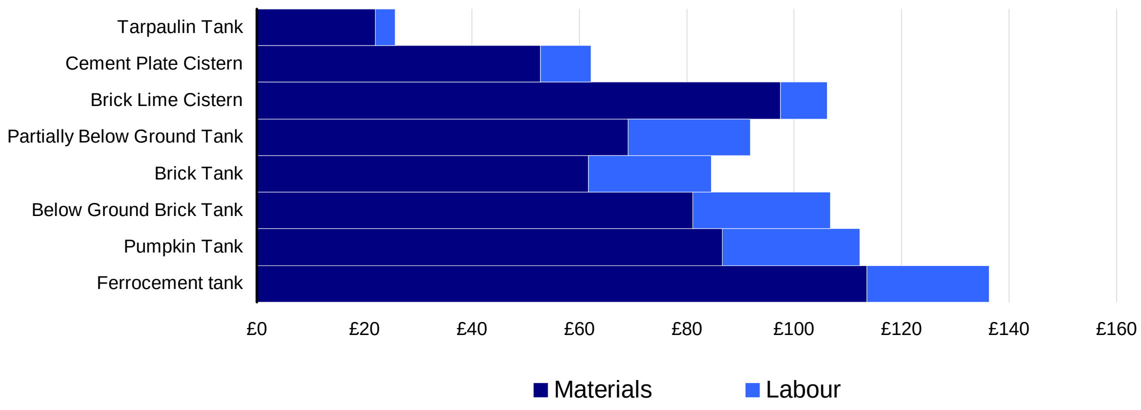
a Brazil



b Sri Lanka



c Uganda



As can be seen the cost of building the tanks differs by quite some margin:

- The traditional ferrocement tank fares most badly, this is probably due to the design being quite old and suffering many years of "improvements" at various hands
- The Pumpkin tank would appear to provide some savings mainly through its more optimised shape and also due to the youth of the design
- The partially below ground tank also provides good economies with reduced material usage and similar labour costs to other cement tanks
- The brick tanks show some materials savings but a slightly higher labour content tends to favour countries where labour is cheap and materials expensive such as Uganda
- By far the cheapest tank is the tarpaulin tank, due in part to its extensive use of "free" local materials and also because of its quick construction
- The cement plate cistern is also an inexpensive option particularly in countries with cheap cement such as Brazil and Sri Lanka. Its unique construction using closely controlled sections of concrete assembled together on site results in substantial savings in material thickness. In more developed countries, this would be an excellent option as it is both durable and desirable as a household asset

Further insight is gained by discounting the labour content of the tanks as the community itself can often provide this. The Brick lime cistern is a case in point here. In Sri Lanka it is a fairly inexpensive option, however its low labour content means that for a self-help project it is less attractive (in Uganda it is the second most expensive in terms of material use!).

9 CONCLUSIONS

Rainwater harvesting tanks represent a mature technology. Their use goes back many centuries and development has been going on throughout history. This does not mean, however, that there is no room for improvement of the technology. Modern techniques and materials have great potential for the manufacture of rainwater tanks. Some of the most promising areas for cost reduction are:

- Plastic bag tanks such as the tarpaulin tank and the DTU tube tank
- Mass produced parts such as the cement plate cistern and lift on covers

- Partially below ground structures to combine the economies of below ground tanks with the safety and desirability of above ground tanks
- Considering the diseconomies of scale inherent in large tanks and using smaller tanks to provide partial supply or seasonal supply.

In considering cost reduction local conditions play a large part.

- Labour and material costs vary widely throughout both the developing and developed world resulting in different designs becoming cheaper
- Conditions may favour one design over another e.g underground tanks are only suitable for areas with stable soils and low water tables, plastic bag tanks are only suitable where insects are not a problem

These caveats notwithstanding the designs tried within the EU programme and many encountered in the field have demonstrated the cost of tanks can be significantly lowered. Domestic rainwater harvesting remains almost unique in that it allows householders to provide their own water supply without the need to wait for outside intervention and the challenge is to produce a system within the means of every household. With appropriate dissemination, the designs presented in this paper should go some way toward this.

REFERENCES

Flugge, Wilhelm (1967) *Stresses in shells*, Springer Verlag New York.

Gnadlinger, J. (1999). "Technical presentation of various types of cistern built in the rural communities of the semi-arid region of Brazil" *9th IRCSA Conf*, IRCSA - Johann Gnadlinger, IRCSA, Brazil.

Gould, John and Nissen-Petersen, Erik (1999) *Rainwater catchment systems for domestic supply*, IT Publications London.

Hapugoda, D. (Nov/1995) *Action Research Study on Rainwater Harvesting*, (Report No. Project Report 2). CWSSP, Ministry of Housing, Construction and Public Utilities, Sri Lanka.

Lee, Michael D. and Visscher, Jan Teun (1992) *Water harvesting - aguide for planners and project managers*, IRC International Water and Sanitation Centre The Hague.

Nissen-Petersen, E., & Lee, M. (1990) *Harvesting Rainwater in Semi-Arid Africa - Manual Number 1: Water Tanks With Guttering and Handpump*, ASAL , Nairobi.

Pacey, Arnold and Cullis, Adrian (1996) *Rainwater harvesting: The collection of rainfall and runoff in rural areas*, IT Publications London.

Rees, D. (Nov/2000a) *Low-Cost, Thin-Shell, Ferrocement Cover - Instructions for Manufacture*, (Report No. TR-RWH04). DTU, UK.

Rees, D. (Nov/2000b) *Partially Below Ground (PBG) Tank for Rainwater Storage - Instructions for Manufacture*, (Report No. TR-RWH01). DTU, UK.

Rees, D., & Whitehead, V. (Sep/2000a) *Brick Jars : Instructions for Manufacture*, (Report No. TR-RWH07). DTU, UK.

Rees, D., & Whitehead, V. (Sep/2000b) *Plastic Tube Tanks : Instructions for Manufacture*, (Report No. TR-RWH08). DTU, UK.

DURABILITY OF COMPRESSED AND CEMENT-STABILISED BUILDING BLOCKS

By

Anthony Geoffrey Kerali

BSc., MSc., MUIPE., MASCE(A)., PGC(ICM)., C.ENG.

A thesis submitted in partial fulfilment of the requirements for the degree of Doctor of Philosophy in Engineering

University of Warwick, School of Engineering

September 2001

TABLE OF CONTENTS

APPENDICES

LIST OF FIGURES

LIST OF TABLES

ACKNOWLEDGEMENTS

The preparation of this thesis was a result of both direct investigations and wide ranging consultations involving a number of people.

While one cannot mention everybody, I would like to specifically record and express my deep gratitude and sincere appreciation to the following:

- Dr. Terry H. Thomas, my research supervisor for his many constructive inputs and invaluable help. His approach and guidance enabled me to remain focussed on the correct track right from the beginning of the research to the very end. Without his constant support and timely interventions, this thesis would have been difficult to accomplish on schedule.
- Professor Peter Carpenter, Dr. Phil Purnell and Dr. Derek Petley, all of the School of Engineering who variously contributed to making the research experience at the University of Warwick a worthwhile pursuit.
- Mr. Colin Banks, Mr. Tony Smith and Mr. Graham Conham for their help in allowing me to use their specialised laboratory space and equipment while conducting the main experimental investigations. Pam Paterson for all the time spent on streamlining the whole thesis.
- Dr. Jackson Mwakali, Dr. Mosses Musaazi and Dr. Barnabas Nawangwe of Makerere University for their contributions during the fieldwork. Further appreciation also goes to Engineers Kimeze Ssebbugga and Wilfred Okello both of the Central Materials Laboratory of the Ministry of Works and Housing in Uganda for assisting me with the documentation as well as inspection of CSB structures in the country.

- Members of the Commonwealth Scholarships Commission in the United Kingdom for sponsoring me; the Central Scholarships Committee in Uganda for nominating me; and Professor John Ssebuwufu, Vice-Chancellor of Makerere University for strongly recommending me for the award.

I remain deeply indebted to you all and to those yet unnamed collaborators who also assisted me during the course of study.

DECLARATION

This declaration serves to confirm that this thesis is the original and exclusive work of the author alone. The thesis does not include either in part or in whole, any previous material submitted by any other researcher in any form not acknowledged as required by existing regulations. No material contained in this thesis has been used elsewhere for publication prior to the production of this work.

This declaration also formally affirms that this thesis is being submitted for the degree of Doctor of Philosophy of the University of Warwick only and not to any other similar institutions of higher learning for the same purposes.

DEDICATION

This thesis is especially dedicated to the following:

My father, Claudio, and mother, Sylvia	-	for their extraordinary foresight, devotion, and sacrifice in educating all their children.
My wife, Hilda and children, Brian and Rupert, and to all my brothers and sisters	-	for their love, support and patience.

ABSTRACT

Adequate shelter is a basic human need, yet about 80% of the urban population in developing countries still live in spontaneous settlements as they cannot afford the high cost of building materials. The compressed and stabilised block (CSB) has been identified as a low-cost material with the potential to redress the problem and reverse the shelter backlog. While its other properties are well understood, the durability of the material remains enigmatic. The principal objective of this research was therefore to investigate the durability of CSBs, especially as used in the humid tropics.

The thesis examines the interplay between three main factors: constituent materials used (cement, soil, water); quality of block processing methods employed; and the effects of natural exposure conditions (physical, chemical, biological). Through a multi-pronged methodology involving literature reviews, laboratory experiments, petrographic analysis and an exposure condition survey, block properties and behaviour are rigorously investigated. The findings are presented under the two main division of the thesis: Part A and Part B.

Part A introduces a review of the literature on the main theoretical concepts of durability and cement-soil stabilisation. It discusses various deterioration modes, and examines in more detail mechanisms of stabilisation using Ordinary Portland cement. Part A also identifies and highlights critical stages of the CSB production cycle, and recommends a strict adherence to proper testing and processing procedures.

Part B presents the results of direct investigation methods used. Findings from the fieldwork confirmed that premature deterioration was widespread in exposed unrendered blocks, with defects exhibited mainly as surface erosion and cracking. Quality checks on site materials and practice established an urgent need for improvement through the provision of appropriate standards and codes. Laboratory experiments which compared the properties of traditional blocks (TDB) and blocks improved by the inclusion of microsilica (IPD), established that the latter significantly out-performed the former. A new quick predictive surface test, the slake durability test, which is more reliable and repeatable than existing tests, is proposed.

The thesis concludes that it is possible to significantly raise the strength, improve the dimensional stability and wear resistance of CSBs to the extent that they can be safely used in unrendered walls in the humid tropics. This improvement is achieved via better intergranular bonding, reduction in voids and lowered absorption. Using the slake durability test, it is now tenable to freely discriminate, classify, and compare not only blocks but other like materials of any category and storage history as well. New quantitative durability gradings are recommended for future incorporation into CSB standards. The findings are likely to contribute to the widespread use of CSBs. The research, however, also raises a number of new questions which are listed for further work.

ABBREVIATIONS

α	=	Degree of cement hydration in water
AAR	=	Alkali-aggregate reaction
ACR	=	Alkali-carbonate reaction
A_{CS}	=	Cross section area
ASL	=	Above sea level
ASR	=	Alkali-silica reaction
BDD	=	Block dry density
$C\bar{S}H_2$	=	Gypsum (C_aSO_4)
C_2S	=	Dicalcium silicate
C_3A	=	Tricalcium aluminate
C_3S	=	Tricalcium silicate
$C_4A\bar{S}H_{12}$	=	Monosulphoaluminate
C_4AF	=	Tetracalcium alumino ferrite
$C_a(OH)_2$	=	Calcium hydroxide
C_aCO_2	=	Calcium carbonate
C-A-H	=	Calcium aluminate hydrate
C_aO	=	Calcium oxide
CBS	=	Concrete block sample
cc	=	Cement content
CCD	=	Curing conditions
CLSB	=	Cement lime soil block
CO_2	=	Carbon dioxide

CP	=	Compaction pressure
CRM	=	Cement replacement material
CRS	=	Corner wall-section
CSB	=	Compressed and stabilised block
C-S-H	=	Calcium sulfate hydrate
CSSB	=	Cement stabilised soil block
DANIDA	=	Danish Agency for International Development
DCS	=	Dry compressive strength
DL	=	Design life
ECS	=	Exposure condition survey/humid tropics
EFF	=	East facing façade
FBS	=	Fired brick sample
GGBS	=	Ground granulated blast furnace slag
H_2O	=	Water (also 'H')
HBT	=	Hold back time
ILO	=	International Labour Organisation
IPD	=	Improved block
l_c	=	Lime content
L_R	=	Loading rate
LSF	=	Lime saturation factor
LST	=	Linear shrinkage test
LWS	=	Lower wall section
MCSB	=	Microsilica cement soil block
M_oWHUD	=	Ministry of Works Housing and Urban Development
MPa	=	Mega Pascal

MS	=	Microsilica
Mwc	=	Mix-water content
MWS	=	Mid wall section
NAB1	=	Namuwongo abandoned building 1 (8 years)
NAB2	=	Namuwongo abandoned building 2 (12 years)
NFF	=	North facing façade
OBS	=	Ordinary builders sand
OMC	=	Optimum moisture content
OPC	=	Ordinary Portland cement
PFA	=	Pulverised fuel ash
PSD	=	Particle size distribution
RBS	=	Rock block sample
RPC	=	Rapid hardening cement
S	=	Amorphous silica
SDI	=	Slake durability index (also I_d)
SDT	=	Slake durability test
SFF	=	South facing façade
SL	=	Service life
Soil 'S'	=	Artificial laboratory blended soil
SSA	=	Specific surface area
SSC	=	Soluble salts crystallisation
ST	=	Soil type
TDB	=	Traditional block
TVP	=	Total volume porosity
TWA	=	Total water absorption

UCR	=	Unhydrated cement residues
UWS	=	Upper wall section
w/c	=	Free water to cement ratio
WAD	=	Wetting abrasion and drying
WCS	=	Wet compressive strength
WFF	=	West facing facade

CHAPTER 1

INTRODUCTION

In Chapter 1, the background to the research, its aims and objectives, methodology used, and the structure of the thesis are described.

1.1 BACKGROUND TO THE RESEARCH

This section presents in outline form the general context in which the research is based, namely a brief history of compressed and stabilised blocks (CSB), their advantages, and the problems that have emerged since their introduction.

The majority of developing countries are today faced with an ever increasing problem of providing adequate yet affordable housing in sufficient numbers. In the last few decades, shelter conditions have been worsening: resources have remained scarce, housing demand has risen and the urgency to provide immediate practical solutions has become more acute. Adequate shelter is one of the most important basic human needs, yet 25% of the world's population does not have any fixed abode, while 50% of the urban population live in slums (ESCAP/RILEM, 1987; ILO, 1987). Indeed 80% of urban settlements in developing countries consist of slums and spontaneous settlements made of temporary materials (Keddi & Cleghorn, 1980; ILO/UNIDO, 1984). With the population in developing countries growing at rates of between 2% and 4% per year and the population in their major cities growing by double these figures, demand for low cost housing far outstrips the capacity to supply (UNCHS, 1981). No developing country without strategies for low cost materials is likely to meet its shelter targets (Webb, 1983; Hamdi, 1995).

Developing countries planning to expand their housing stock for the low-income groups will inevitably need to identify the lowest feasible unit housing costs. The main costs of shelter provision are for building materials (about 60%), machinery, manpower and loan interest repayments (BRE, 1980; Ashworth, 1994; Maclean & Scot, 1995). Strategies are therefore urgently needed to develop low-cost, readily available and durable building materials. A naturally abundant material such as soil that is found on most of the surface of the earth should be a significant resource for building in developing countries.

Research and development of stabilised soil as a building material is not new. The use of CSBs can be traced back 50 years (Fitzmaurice, 1958; Enteiche & Augusta, 1964; Fathy, 1973; Webb, 1988). From the early 1950s attempts were made to develop the material as an alternative walling unit to the modern and more expensive fired bricks and concrete blocks. The promotion of the material was originally introduced via the United Nations (UN Bulletin No. 4, 1950; Fitzmaurice, 1958). The idea of compacting earth to improve its quality and performance in the form of moulded blocks however dates back to the 18^{th} Century (Houben & Guillaud, 1994). The addition of a binder to stabilise the soil is more recent.

Apart from the early work of the United Nations, the history of the spread of the CSB is not well documented. During the 1950s use of the material was widely disseminated worldwide. The 1960s and early 1970s were however stagnant years. This was to change with the 1976 Vancouver Assembly of the United Nations Conference on Human Settlements (UNHCS, 1976; UNIDO, 1980). Noting with concern that the worlds population was expected to double by the year 2000, and worse still, to quadruple by the year 2030 (representing the largest single population growth in human history), the conference resolved to focus on the development of low-cost housing. Further momentum was to be given 12 years later following the declaration of the year 1987 as the International Year of Shelter for the Homeless (UN/IYSH, 1987). Subsequent proclamations were to follow in 1988 under the theme 'Global Strategy for Housing by the Year 2000'. The key targets of these resolutions were the guaranteed access to decent and durable housing for all from the beginning of the new millennium. Renewed world-wide interest was soon to provide an immense impetus that has ensured the now vibrant spread of CSBs throughout the developing world. It was within this international context and drive that the author became involved with the material in 1987 following the donation to Uganda in that year of several block presses and ancillary materials by the International Labour Organisation (Okello, 1989; Schmetzer & Kerali, 1994; Kerali, 1996).

Continued interest in CSBs will in future evolve around the several merits and attractions associated with its use. Firstly, as the basic raw material is soil, its source will remain abundant. This facilitates direct site-to-service application, thereby lowering costs normally associated with acquisition, transportation and production. Home ownership can then be delivered at comparatively low costs. Secondly, the initial performance characteristics of the material such as the wet compressive strength (WCS), dimensional stability, total water absorption (TWA), block dry density (BDD) and durability are technically acceptable. They are also comparable to those of rival materials (ILO, 1987; Houben & Guillaud, 1994; Houben et al, 1996). Houses constructed of CSBs also uniquely proffer better internal climatic conditions than other modern materials (Fullerton, 1979; Hughes, 1983). Thirdly, promoting the use of CSBs generates more direct and indirect employment opportunities within the local populace than would be the case with other materials. Fourthly, use of the material contributes directly to the social, cultural and educational advancement of the population (Schumacher, 1973; Anderson et al, 1982; Aksa, 1984). Their use also contributes to the training and re-training of artisans and to the provision of new skills. Use of the material through the provision of local infrastructure such as schools, community centres, health centres and administrative units results in the promotion of human interactions and social development. Finally, use of the material is environmentally friendly, appropriate and correct since it utilises the otherwise unlimited natural resource in its natural state. Moreover, this is achieved with little resultant depletion of other resources, or pollution and requires no excessive energy consumption and wastage as is the case with clamp fired bricks. The elimination of the need for wood fuel resources is seen as a major attraction over such bricks. The use of CSBs is thus in keeping with current sustainable development strategies (VTA, 1977; Plinchy, 1982; Lawson, 1991; Perera, 1993; Norton, 1997).

Despite the above advantages however, as with most relatively new materials, shortcomings associated with their use have recently begun to emerge, especially in tropical environments. These regions are characterised by frequent and intense rainfall, high relative humidity and high diurnal temperature changes (Bilham, 1962; Atkinson, 1970; Eaton, 1981). CSBs are produced from soil as the bulk constituent (over 90%). Soil is known to have poor resistance to erosion and to disintegration in water, a low tensile strength, low resistance to abrasion, high water absorption and retention capacity, and is dimensionally unstable during cyclic wetting and drying (Ellison; 1944, Carter & Bently, 1971; BOR, 1974; Das, 1983). The vulnerability of soil has in turn led to blocks showing considerable defects over short periods under conditions of normal and severe exposure in the humid tropics (Lunt, 1980; Agarwal, 1981; Eaton, 1981; Tibbets, 1982; Spence & Cook, 1983). Whereas the initial building costs might be low, the subsequent high maintenance costs, or even early rebuilding costs are not acceptable to many. Some promoters have also done harm to the image of the material by claiming a high degree of long-term technical performance only to be contradicted by premature deterioration only a few years later (Hammond, 1973).

Although the problem is more acute in the humid tropics than in the arid zone, it nevertheless has not been seriously addressed by research. Interest in studying the durability of CSBs is therefore likely to remain a major research concern for the foreseeable future. It is the long-term durability of the block, rather than any other factor that will be the key to their widespread acceptance (Gooding, 1994). It is therefore the goal of this research to investigate the feasibility of producing a high intergranular strength block of low permeability, which is inert, dimensionally stable and durable, even under conditions of exposure to wetting, abrasion and drying.

1.2 AIMS AND OBJECTIVES

Following on from the discussions in the preceding section, the objectives of this thesis are threefold. Firstly, to investigate the main constituent materials and the block production process, secondly to examine the main block properties and their performance, and thirdly to make recommendations for improved specification, testing and protection of CSBs for the duration of their service lifetime.

The scope covered under each of the above objectives are summarised below:

1. *Constituent materials and block processing methods:*

   (a) to investigate all present theoretical and practical methods by which soil and cement are classified, selected, and tested for CSB production.

   (b) to closely examine the mechanisms by which cement and associated binders and additives effect stabilisation in soils by increasing strength, dimensional stability and durability.

(c) to investigate the block production process with a view to identifying the critical sub-processes that influence the quality and performance of the block as a product.

2. *Surface and bulk properties and performance:*

(a) to study experimentally the effect of altering important variables like: soil type, cement content, cement replacement materials, mix-water content, moulding pressure and curing conditions on the properties and performance of blocks.

(b) to compare the performance of traditional and improved blocks as defined by the absence and presence of partial cement replacement materials respectively in the mix composition of CSBs.

(c) to review the concepts of durability and identify the main deterioration mechanisms involved, understand their methods of progression and propose appropriate remedial action.

(d) to identify the key surface and bulk properties on which durability is thought to depend, and monitor the performance of block categories mentioned in (b) in conditions simulating the action of the main deteriorating agents using accelerated tests.

(e) to collect in-service performance data from condition surveys and other records in order to estimate the service life of blocks.

3. *Improved design and specification for durable blocks*

(a) to recommend, from data and evidence obtained from 1 and 2 above, new approaches for achieving more appropriate, affordable and durable blocks.

(b) to develop and specify new initial and accelerated predictive tests for blocks that can be conducted on site, in laboratories and under in-service conditions.

(c) to recommend alternative surface and bulk improvement and protective measures for use of blocks under severe exposure conditions.

(d) to suggest suitable minimum performance requirements and thresholds for incorporation into future standards.

(e) to disseminate findings and flag questions for further research through publications, seminars and conferences.

The achievement of the above objectives are evaluated in the concluding parts of this thesis.

1.3 METHODOLOGY

There is very limited information available on the long term behaviour of CSBs. This is partly because no prior research has been conducted in the area and partly because inspection and maintenance records on the performance of blocks are not available. In view of such circumstances, the use of a combination of various approaches was considered to be inevitable. These approaches included:

1. *Literature review*: to establish the level of current thinking and knowledge and to provide the intellectual context for the research.
2. *Laboratory experimentation and testing*: of key surface and bulk properties of blocks as well as monitoring their performance using accelerated tests. Two categories of blocks were evaluated: traditional and improved blocks.
3. *Petrographic examination of CSB microstructural features*: using thin sections
4. *Exposure condition survey in the humid tropics*: done through:

(a) Inventorisation of CSB buildings and characterising their exposure conditions.

(b) Visual inspection of buildings to identify defect types and their severity.

(c) In-service condition measurement of the main defect types.

(d) Quality tests on constituent materials: soils, cement and water.

(e) Visits to block production sites for preliminary work-study assessment.

(f) Questionnaires and interviews for opinions, experiences and knowledge from stakeholders.

1.4 STRUCTURE OF THE THESIS

The body of this thesis consists of eight chapters presented in two parts, A and B. The organisation of the chapters is as follows:

Chapter 1 provides an introduction to the whole thesis. It discusses the background to the research and the context in which it is based. This Chapter also summarises the main aims and objectives of the research and explains why the different methodologies listed had to be used for the research. Chapter 1 ends by providing guidelines on the organisation and structure of the thesis as a whole including the ordering of the main parts, chapters, references and appendices. These are now described each in turn.

Part A: *Concepts of Durability and Stabilisation*

Part A consists of two chapters, namely Chapter 2 and Chapter 3.

Chapter 2 introduces the fundamental theoretical concepts of durability and deterioration in CSBs. The Chapter emphasises the fact that understanding of the main concepts of durability and deterioration in blocks is both necessary and yet long overdue. This Chapter identifies the principal deterioration agents and their perceived effects and attempts to rank them according to their severity. It also discusses the various deterioration mechanisms related to water, temperature and chemical reaction respectively. The main significance of Chapter 2 is that concepts previously not recorded in CSB literature are developed. It further emphasises the multidisciplinary nature of the research topic and reviews the wide store of knowledge accumulated from recent advances in durability research of cementitious materials. By presenting the subject in this manner, a more comprehensive understanding of the concepts of durability and deterioration in CSBs is achieved. The main surface deterioration mechanism in blocks (water-related surface wetting, abrasion and drying) is identified for further direct experimental investigation in Part B of the thesis.

Chapter 3 reviews and builds on the current understanding of cement-soil stabilisation principles and practices. This Chapter also examines the properties of the main constituent materials that form blocks (soil, cement, water) and reviews the effects of varying their proportions on the performance of blocks. The nature of each of these constituent materials, the manner in which they are selected and proportioned to form mixes for CSBs are closely analysed. The Chapter examines the adequacy of the theories of cement hydration and hardening, and the effects of the various cement hydrates on the durability of the block. It also reviews the block production process and its influence on the initial performance characteristics of CSBs. The main feature of Chapter 3 is therefore the identification of the main production variables affecting the properties and performance of the CSB. These variables are examined experimentally in Part B of the thesis.

Part B: *Main Investigation Methods and Findings*

Part B of the thesis consists of Chapter 4, 5, 6 and 7.

Chapter 4 describes the fieldwork undertaken in Uganda and the results obtained. It presents findings from the following: inventory of the types and numbers of CSB buildings; characteristics of their exposure environment; visual inspection of defect types and severity; in-service measurement of the main defects; quality tests on soils, cement and water; work-study evaluation of block site production practices, and results from questionnaires and interviews. The significance of Chapter 4 is that the defect types reported on were a result of genuine weathering conditions. Since the full effects of the entire range and distribution of deterioration agents acting on the blocks in their full scale size and within the restraints of adjoining blocks and mortar could be observed and limited measurements taken, the results obtained were fairly reliable and useful for generalisations to be made. The same applies to findings from block production site and quality of the constituent materials examined. The results from interviews and interactions with stakeholders are also discussed in Chapter 4.

Chapter 5 describes the main experimental design and sample preparation methods used for laboratory tests. It explains why the experimental soil type was fixed for all types of tests while the stabiliser content and type, mix-water content, moulding pressure, curing conditions were varied for the different categories of block samples prepared. Chapter 5 also presents the two main categories of block types produced for further experimentation: improved blocks (containing cement and microsilica) and traditional blocks (containing cement only and cement plus lime). It is the comparison of the properties and performance of these two types of blocks that constitutes the core of the experimental work in this research. Chapter 5 also shows the main block sample sizes produced and provides test results on the laboratory experimental soil which was blended for the research. It was from the samples produced as described in Chapter 5 that further surface and bulk property tests were conducted.

Chapter 6 investigates block bulk properties and compares the performance of improved and traditional blocks. The properties examined experimentally include: wet compressive strength and dry compressive strength, total water absorption, block dry density, and total volume porosity. The effect of varying the main input variables on these properties are analysed. Chapter 6 records the main features of the standard tests used, and argues for the need to establish more appropriate calibrated tests for CSBs. Factors likely to affect the various test types are discussed. The implications of particular test results and their correlation to other block properties are discussed.

Chapter 7 reports on the microstructural features of block surfaces and discusses the main surface test method used and the results obtained. It explains how thin-section micrographs were obtained to identify the general surface features of CSBs. This Chapter also argues for the need for a more reliable surface test method and describes a new accelerated surface test method. The results of surface performance tests using this method on improved and traditional block samples, as well as those from comparable materials, are discussed. A new block classification system based on their index values is presented. Factors considered likely to influence the results such as equipment type, sample dimensions, pre-treatment, duration of slaking and nature of the slaking liquid are all described. Chapter 7 also discusses the correlation between different block properties based and their slake durability index.

Chapter 8 is the final chapter of the thesis, integrating and summarising the main conclusions and recommendations from Parts A and B. The Chapter also highlights the implications of the research findings and identifies areas for further research.

At the end of the thesis, references and appendices are presented. The appendices illustrate and support sections of the thesis that could not be included within the main body of the write-up. Summaries of experimental methodology and full tables of results are included in the appendices.

PART A:

LITERATURE REVIEW ON DURABILITY AND STABILISATION

CHAPTER 2

CONCEPT OF DURABILITY IN CSBs

2.1 INTRODUCTION

Cement based building materials like CSB's and concrete were originally promoted as having an indefinitely long service life, and that they would require only minimum maintenance. Many cement based materials have indeed given excellent service. However, as these structures continue to be left exposed, it is becoming evident over time that even normal exposure conditions are actually more deleterious than originally thought (Baker et al, 1991; Sjostrom et al, 1996). Occurrences of undesirable, unpredicted premature deterioration where defects are clearly visible even to the casual observer, are becoming common. Defects in CSB structures are mainly presented as surface erosion, volume reduction, cracking and crazing, surface pitting and roughening, and detachment of render. These deterioration phenomena have been predominantly witnessed in the wet humid tropical regions of the world. No similar adverse reports have been documented from the hotter and drier regions (Spence & Cook, 1983).

In this chapter, it is noted that while much research has been undertaken in the recent past on initial properties of CSBs, very little similar research has been done on its durability. Recent advances have however been made in the durability research of comparable materials such as concrete. These are now well documented, and moves to redress identified shortcomings are following. By contrast as mentioned before, no durability research work has been conducted for decades in the case of CSBs. Yet the urgency is even more acute. Interest in the durability of CSBs is likely to become a major concern in the foreseeable future given the potential of the material in alleviating shelter backlogs in developing countries (Gooding, 1993).

Durability research is a complex undertaking. This is because in practice several causes of deterioration will occur simultaneously. These are compounded by cumulative as well as synergistic actions. In recognition of such intricacies, the objectives of this chapter are several, namely to:

- identify the most critical deterioration agents, their effects, and severity ranking;
- understand the main mechanisms involved, their modes of progression and propagation;
- suggest measurement techniques to quantify the main outputs of deterioration;
- recommend selected remedial measures

Chapter 2 is presented in four main sections. After this introductory section, the main expressions of durability and deterioration are discussed. This is followed by a discussion of the main deterioration mechanisms likely to occur in CSBs. A brief conclusion then follows.

2.2 EXPRESSION OF DURABILITY AND DETERIORATION IN CSBs

The terms durability and deterioration are perhaps the two most commonly used words in the field of construction materials. This section attempts to describe the basis of these two terms, and examines their relevance to the performance of CSBs.

Durability

The word durability originates from the Latin word '*durabilis*' which means 'lasting' (Franklin & Chandra, 1972). It can be used in the context of most building materials to mean resistance to weakening and disintegration over time. The term has been described in various ways by different authors although the substance appears to remain the same in all cases. According to BS 7543 : 1992, durability is defined 'as the ability of a building and its parts to perform its required function over a period of time, and under the influence of agents'. But according to BSI CP3 1950, 'durability is a measure, albeit in an inverse sense, of the rate of deterioration of a material or component'. More recent definitions state that 'durability may be regarded as a measure of the ability of a material to sustain its distinctive characteristics, and resistance to weathering under conditions of use for the duration of the service lifetime of the structure of which it forms part' (Baker et al, 1991; Sjostrom et al, 1996; Glanville & Neville, 1997). These definitions are too general to be of any practical use with CSBs.

The author proposes that the definition and concept of durability be based on three key parameters, namely:

- intended function of the material,
- the standardised conditions of its use, and
- the time the material is required to fulfil its functions

The *intended function* of a CSB is as an internal and external walling unit. The primary desirable characteristics of walling units are strength, dimensional stability and resistance to weathering (ILO, 1987; Carroll, 1992). These properties are to a large extent governed by the choice of constituent materials, and by the quality of the manufacturing process used in their production (Webb, 1988). In terms of intended function therefore, the ability of a block to sustain its distinctive characteristics under service conditions for the service lifetime of the structure is very important. Unfortunately, the values of initial performance characteristics of a block are not likely to remain constant over time. Variations in properties can come about due to the evolution of the block fabric as it undergoes changes induced by the effects of its exposure conditions. The changes can lead to loss of performance, implying that every material has its durability limit. The durability limit is the point at which loss of performance leads to the end of the service life of a material (BS 7543, 1992). The threshold for satisfactory performance for CSBs is yet to be defined.

Standardised conditions of use ought to be included in the definition of durability. As walling units, CSBs are used on the exterior of buildings. They are therefore exposed to physical, chemical and biological elements. Some of these agents can have deleterious effects on blocks even under normal conditions of use. The fact that CSBs when used in the humid tropics (characterised by heavy and intense rainfall) are more vulnerable to surface erosion than similar blocks used in dry areas, supports the reasoning that conditions of use be included in the definition of durability (Fitzmaurice, 1958; Spence & Cook, 1983). Harsh conditions of use can lead to wear, cracking, dampness and undesirable dimensional changes. However, CSBs are still required to resist the effects of exposure conditions for the service lifetime of the building. The requirement for resistance will vary according to the different types of agents involved. For each of the different deterioration agents identified, it will be helpful to specify the particular aspect of durability required. For example 'abrasion-durability, slake durability, heat durability, chemical durability' etc., all require matching durability thresholds. The main reason is that the mechanisms for each deterioration agent are different. Moreover, not all types of agents are likely to be in operation in different parts of the world. As can be expected, different tests will also be required to measure the effects of the different deterioration agents.

The *time* the block is expected to fulfil its intended functions (in the definition of durability) should also be specified more clearly to meet the users requirements. In the case of building structures, the time ought to be expressed in terms of years of satisfactory life. Guidelines on building life categorisation are provided in BS 7543 : 1992. These range from 10 years in the case of temporary buildings to over 120 years in the case of high quality buildings. The effect of exposure conditions leading to loss of performance is likely to be gradual but not abrupt. The rate of loss of performance or quality of a block is also likely to depend primarily on the actual conditions of exposure: blocks left exposed in the humid tropics will be more vulnerable to rapid deterioration than similar blocks used in arid regions (Fitch & Branch, 1960). Since blocks are meant to be maintainable materials rather than replaceable materials, specification of expected performance limits over a certain period of time and under specified conditions of exposure, are long overdue.

Deterioration

Deterioration has been defined by several authors as 'the time-related loss of quality of a material, usually under the influence of environmental agents' (BS1 CP3 1950; BRE, 1980; Baker et al, 1991). Premature deterioration has also been defined as 'failure to achieve the predicted service life' (BS 7543, 1992). The predicted service life of a block can be obtained from recorded performance or from accelerated tests. Unfortunately, such records are not available.

Failure due to the inability of a newly made block to fulfil its functions has to be clearly distinguished from failure brought about by alterations in properties over the service lifetime of a block. Indeed most building materials will have some of their properties altered over time although their durability may not always be called to question. The durability of a block can therefore be regarded as its ability to resist deterioration. It can be treated as the reciprocal of deterioration under pre-defined conditions (Sjostrom et al, 1996).

Due to deterioration however, the durability of a block is unlikely to remain constant. It may in fact change considerably. The implication is that durability of a block and its deterioration are likely to influence each other mutually but negatively. As can be expected, the more a block deteriorates, the less durable it is likely to become over time. For example bulk properties of a block such as water absorption and permeability are related to the type of microstructure and density of the block. However, the microstructure and density of a block may alter appreciably due to weathering (deterioration). This alteration can in turn increase the water absorption and permeability of the block. Such increases are likely to accelerate the rate of deterioration due to softening and dissolution of any unbound soil particles in the block. Further loss of performance can then be expected. The limit at which the loss of performance can be considered unacceptable is not yet well defined in CSBs. Unfortunately, even if it was, the limit may not be easily applicable without further qualification. This is because depending on the constituent materials used in a block, and on the quality of the processing methods used, no two blocks might be easy to compare. Unacceptable deterioration will therefore vary from block to block, and from property to property. Block properties that diminish over time reflect the past history of the block, both during and after manufacture.

2.3 DETERIORATION MECHANISMS IN CSBs

As is the case with most other building materials, deterioration mechanisms in CSBs are varied and complex. From the literature and experience gained through the use of the material, laboratory tests, building inspection records and the exposure condition surveys, three main deterioration modes can be identified, namely:

- Water related deterioration
- Temperature related deterioration
- Chemical based deterioration

These are now discussed each in turn in the following sections.

2.3.1 WATER RELATED DETERIORATION IN CSBS

Water related deterioration mechanisms account for most of the observed premature deterioration defects in CSBs (Fiztmaurice, 1958; UN, 1964; Lunt, 1980; Agarwal, 1981; Spence & Cook, 1983; ILO, 1987; Norton, 1997). Water also serves as a common denominator for other deterioration mechanisms occurring in blocks. The main sources of water linked to such deterioration mechanisms are rain, rising damp and condensation. The action of water in causing deterioration in blocks can occur in any one or all of the following ways:

- solvent action
- abrasive action
- swelling action
- catalytic action

The first two in the above list, namely solvent action and abrasive action are discussed in this section. The last two, namely swelling action and catalytic action are discussed in Section 2.3.3 (chemically related deterioration). For each action, an attempt is made to describe its nature, where it occurs, when it occurs, why it occurs and how it is likely to occur in a block. Where possible, references to similarities and differences with associated mechanisms in concrete materials are examined.

The *solvent action* of water is mentioned in the literature as one of the most common deterioration mechanisms occurring in many building materials (Sjostrom et al, 1996). The ability of a block surface to easily get wet, and the capacity of the block to absorb and retain water for sufficiently long periods of time, are two properties likely to leave the material vulnerable to the solvent action of water. The composition of a block fabric itself might also contribute to its vulnerability. Over 90% of the block bulk consists of soil, with the other 10% or less consisting of cement. In a stabilised block matrix, the process of cement-stabilisation is known not to affect all the constituents in the block (Herzog & Mitchell, 1963; Houben & Guillaud, 1994). Moreover, the hydration reaction between OPC and water which is responsible for the binding action in the block also produces soluble by-products such as calcium hydroxide (Illston, 1994). The microstructure of a block consists of materials which are juxtaposed with capillary pores. The block is therefore able to attract water and retain it. As water permeates the block, any unstabilised soil fraction present, together with the freed calcium hydroxide from the hydration reaction of cement, can be expected to dissolve. Dispersal and subsequent leaching out of these substances can then follow. Repeated action of this nature over the years can lead to overall softening of a block fabric. Such action can also have the effect of weakening and altering the microstructure of the hardened cement matrix in a block. The microstructure of a block is therefore likely to continue evolving throughout its service lifetime. This is a detrimental trend since the softening and leaching action is irreversible. The severity of the action can be expected to increase during the rainy seasons, and to depend on the proportions of materials present in the block which are vulnerable to dissolution and softening. Unfortunately, as this form of deterioration progresses, it has the adverse potential of making the block more vulnerable to other forms of deterioration such as the erosive action of rainwater droplets. The solvent action of water in causing deterioration is not investigated experimentally in this thesis.

Surface abrasion by rainwater has been identified from literature sources as one of the most common deterioration mechanisms associated with water (Atkinson, 1970; Agarwal, 1981; Eaton, 1981; Fullerton, 1979; Ola & Mbata, 1990). Fortunately however, surface erosion only occurs in areas prone to frequent and intense rainfall such as obtains in the humid tropics. The mechanism of surface erosion in blocks might not yet be well understood but the phenomenon is thought to proceed as follows. When rainwater strikes an exposed block surface, it will directly impact on it, with part of it turning into a spray. While the effect of the impact can be likened to the removal of loose particles, the effect of the spray is more likely to first wet the block surface. It has been estimated that up to 75% of the energy of a raindrop is dissipated on impact (Ellison, 1944; Goldsmith et al, 1998). The erosivity of raindrops depend on the state of bonding of the block surface, and on the characteristics of the rain. The state of bonding of a block surface is discussed in Chapter 3. The main characteristics of rain are defined by the drop size, its distribution, fall velocity and impact kinetic energy (Gunn & Kinzer, 1949; Laws, 1941; Hudson, 1963; Wilson, 1993). It is therefore the interaction between the raindrop size, velocity and shape, storm duration and wind speed that is likely to control the erosive power of the raindrop. It would be reasonable to expect that the higher the impact velocity of a raindrop and the weaker the state of bonding at the block surface, the greater would be the effect of surface erosion. Conversely, the lower the impact velocity of a raindrop, the greater is the effect of the raindrop forming sprays on the surface of the block. Any detached soil particles, (usually assumed to be from the unstabilised fraction of the block surface fabric), can then be easily removed by the resulting wall surface flow. The effect of surface abrasion is irreversible. The defects linked to this process are discernible even to the casual observer. They include recessed wall surfaces and volume reduction caused by mass loss. Indirect effects of surface erosion include lowering of surface hardness, lowering of compressive strength, loss of rigidity, lowering of density and increase in permeability. The loss of mass from a block surface can have other more serious consequences. Given the mechanism of quasi-static compression used in forming blocks, their inside core is the part least affected by compaction (Gooding, 1994; Houben & Guillaud, 1994). This part is therefore considered to be its weakest link. As can be expected, exposure of the interior due to recessed surfaces can lead to the speeding up of the rate of deterioration. Extra measures are therefore needed to strengthen the block surface in order to protect its bulk from exposure.

Unlike in CSBs, the phenomenon of surface erosion (to the extent it occurs), has not been reported in concrete literature. Given the low amount of OPC used in CSBs (5-8% by weight) as compared to concrete products (12-14% by weight), weaker inter-particle bonding in the former can be expected. Moreover, even with the low amount of cement used, full hydration of the binder might not be fully achieved. This is because unlike in concrete where the water-cement ratio can be pre-determined accurately, the effective water-cement ratio in CSBs is still difficult to define. Moreover the water required for the hydration of cement is shared between the cement and the highly hydrophilic clay in the soil. The water is also required to be at an optimum level to fully lubricate the soil particles to achieve maximum densification. The equilibrium between these three requirements for the mix-water are not yet fully understood. Until this is done the incomplete hydration of OPC will continue to lead to weaker block fabrics. A denser, homogeneous and impermeable block surface would probably minimise the effect of surface erosion more than one which is not. In this thesis, the performance of block surfaces produced by changing input variables such as cement content, compaction pressure, cement replacement materials, etc., are investigated experimentally (Chapters 6 and 7). The severity of surface erosion is also investigated through a case study conducted in Uganda (Chapter 4).

2.3.2 TEMPERATURE-RELATED DETERIORATION IN CSBS

As CSBs form the exterior part of buildings, they will inevitably experience regular temperature variations. The daily maxima and minima, diurnal temperature differences and temperature levels will vary depending on the geographic location of the CSB building (BRE, 1980; Wilson, 1993). High ambient temperatures are common in tropical and sub-tropical regions of the world (Hammond, 1972; Spence & Cook, 1983; McIllveen, 1998). Maximum daily ambient temperatures averaging 40°C to 50°C occur in such areas especially between late mornings and early afternoons, often peaking at midday on cloudless days. At night, temperatures may drop to below 0°C. The diurnal temperature variation can therefore exceed 50°C (Anderson, 1982). Moreover, sunshine and night hours are long, typically averaging 12 hours each day most of the year. Such extremes provide contrasting settings for temperature related deterioration to occur in CSBs.

Temperature variations of such magnitude can cause both reversible and irreversible changes in the physical and chemical properties of blocks. These changes are likely to influence the durability of blocks in three main ways, namely:

- expansion and contraction of the block fabric
- shrinkage and drying (of clay and hardened cement paste)
- catalytic action (for chemical reactions)

Expansion and contraction of a CSB fabric due to temperature variations is likely to be detrimental to the properties of the material in the long run. Similar dimensional changes are also reported in concrete research (Baker et al, 1991; Glanville & Neville, 1997). Deterioration is likely to result from stress levels induced within the block. CSBs have a positive coefficient of thermal expansion, typically ranging between 0.010 mm/m°C and 0.015mm/m°C (Rigassi, 1995). It is the absorbed radiation which is responsible for the temperature rise in blocks. The amount of radiation absorbed depends on the specific heat (C) and the thermal conductivity (λ) of the block. Typical values for blocks range between 0.65 and 1.00 kj/kg for the former and between 0.23 and 1.04 W/m°C for the latter (Houben et al, 1996). The values vary from block to block depending on the moisture condition at the time of the temperature change and testing, and on the composition of each block. At high temperatures, a block can easily expand. But the expansion can be restrained by adjoining blocks as well as the embedding mortar. The expansion of a block can induce significant internal stresses (compressive and tensile). Since blocks are weaker in tension, such stresses can be expected to be more harmful to its fabric. Further, as stress and strain tend to occur together, any restraint of movement for the expanding block introduces a stress corresponding to the restrained strain (Neville, 1995; Case, Chilver & Ross, 1998). If this stress and the corresponding restrained strain within a block are allowed to develop to such an extent that they significantly exceed the bulk strength or its strain capacity, then interfacial bonds that bind and hold the soil particles within its fabric together can be weakened. In more extreme conditions, they might even be severed apart altogether. Cracks are then likely to appear and propagate on the surface of a block. Not only can such cracks facilitate entry of moisture, but they are also unsightly.

Conditions at night represent the complete reverse of the situation during the day. Contraction of a block can be expected to take place, at temperatures about or below zero degrees Celsius. This occurs in an attempt to revert to the pre-expansion order. The effect of cooling is rather late since irreversible damage to the block fabric is likely to have already occurred due to expansion. It is such continuous, cyclic and repeated phenomena of expansion and contraction that can eventually degrade the block (Torraca, 1988). The effect of expansion and contraction of blocks due to high temperatures are not investigated experimentally in this thesis. A case study to link various cracking patterns to this mode of deterioration was however undertaken (Chapter 4).

Shrinkage and drying of CSBs can also be associated with high ambient temperatures. According to literature sources, block surfaces left unprotected from direct sunlight can absorb considerable amounts of solar radiation, raising surface temperatures beyond that of the surrounding air temperatures (BRE, 1980). Surface temperatures as high as 100°C and shade temperatures as high as 60°C have been reported in parts of the humid tropics (BRE, 1980). Such high temperatures can cause dimensional changes to occur in a block resulting in a fractional reduction in its volume .

Two different mechanisms of shrinkage are believed to take place in a block (BRE, 1979). These are shrinkage due to the expulsion of water from its capillary pores, and shrinkage due to the withdrawal of moisture from the clay fraction and the hardened cement paste. While the former is considered to be a reversible process, the latter are irreversible. At high ambient temperatures, moisture can be lost from a block through evaporation. Unbound water filling the capillary pores in a block are expelled in the process. Any dimensional changes that follow are likely to be insignificant. This reversible process has been reported as not being harmful to the block fabric.

However, even after the expulsion of all capillary pore water, there can still remain some water within a block fabric. This can occur in the form of strongly adsorbed water within unstabilised clay platelets and the hardened cement gels. This water can only be gradually withdrawn at high temperatures over long periods of time. The withdrawal is a slow process, but more significantly an irreversible one in the case of the hardened cement paste (Van Olphen, 1997; Torraca, 1988; Glanville & Neville, 1997). The mechanisms of withdrawal for clay and hardened cement paste are different but are not discussed further in this thesis. The effects of shrinkage and drying in CSBs are not investigated experimentally in this thesis. Defects arising due to their action were however evaluated during the fieldwork (Chapter 4).

The *catalytic action* of temperature variations in initiating and propagating chemical reactions is a well known phenomenon in most cement based materials. The phenomenon is also widely reported in concrete literature (Baker et al, 1991; Bungay & Millard, 1996; Jackson & Dhir, 1996). Most chemical reactions that would not have occurred within a block at low ambient temperatures are more likely to take place at higher temperatures. As stated earlier, fluctuations of temperature can influence moisture movement within a block. The combination of the presence of moisture within a block, and high temperatures has been known to provide the catalytic setting responsible for reviving otherwise dormant chemical activity.

Temperature variations are associated with the control of the rate of chemical activity. Reactions facilitated by this type of mechanism include soluble salts crystallisation, oxidisation, leaching, etc. It is reported that an increase in temperature of only 10°C can double the rate of chemical reactions in most cement based materials (BRE, 1980). Increasing the rate of deleterious chemical activity can be potentially harmful to a block in the long run. The mechanisms of the various chemical reactions linked to temperature variation are discussed in Section 2.3.3.

In summary, temperature related deterioration in blocks are likely to affect the following block properties: shape, dimensions, strength, surface hardness, rigidity, permeability, brittleness and appearance. The severity of deterioration will depend on the degree of cloud cover, degree of shading from direct sunlight, geographic location, orientation of the building façade, moisture condition of a block and its texture, opacity and colour of the block (BRE, 1980). The influence of some of these factors were investigated during the fieldwork that formed part of this research (Chapter 4). To minimise the effects of temperature variations, surface protective measures ought to be adopted. These can include use of reflective coating, surface render, low roof overhangs, etc., which can all be specified whenever blocks are thought to be especially vulnerable to high ambient temperatures.

2.3.3 CHEMICAL-RELATED DETERIORATION

The deterioration of CSBs can also be linked to the effects of chemical activity. According to literature sources, mechanisms associated with chemical action in CSBs remain the least investigated (Houben & Guillaud, 1994). Yet sources of potentially reactive chemicals in a block are soil and cement. Soils which constitute most of the bulk of a block contain minerals as well as contaminants (Lunt, 1980). Some of these substances can remain dormant and stable when not in active contact with environmental elements (rainwater, high temperatures, relative humidity, gasses). Ordinary Portland cement as the main binder in blocks also contains potentially unstable chemical constituents even in the hardened cement paste phase. Contact with environmental agents can catalyse chemical reactions in cement hydrates (Illston, 1994).

The precondition for chemical reaction to start in most cement based materials is the presence of moisture (Lea, 1970; BS 7543, 1992). Due to seasonal moisture variations from heavy rainstorms and humid conditions in the tropics, chemical reactions can be expected to occur within a block during its service lifetime. The rate of such reactions is likely to be influenced by variations in ambient temperatures as discussed in Section 2.3.2. Environmental conditions found in the humid tropics therefore provide the best possible setting for chemical activity to occur in a block. Based on the nature of their action and resulting effects, deleterious mechanisms of chemical action can be broadly categorised into three groups, namely:

- leaching out effect (clay and calcium hydroxide)
- expanded product formation (internal stress generation)
- direct decomposition (of the cement binder)

These are now each briefly discussed in turn.

Leaching out effect

Leaching is a phenomenon that involves the washing out of soluble substances from a material (Jackson & Dhir, 1996). There are two key sources of soluble substances in blocks: the calcium hydroxide found in the hardened cement paste, and the clay fraction likely to be found in residual unstabilised or partially stabilised matrix of a block (Houben & Guillaud, 1994; Young, 1998).

Calcium hydroxide ($Ca(OH)_2$) is known to easily dissolve in water (Illston, 1994; Neville, 1995). The dissolution process is irreversible once started, and is known to be facilitated by high temperatures, and the presence of carbon dioxide. Moreover, block properties such as water absorption and permeability, are likely to ensure that adequate moisture is absorbed and circulated within a block. Dissolved calcium hydroxide can be removed out of a block in either of two ways. It may simply be washed out of a block through surface flow on saturation during rainstorms, or it may be expelled onto the block surface by evaporation due to high temperatures. The phenomenon of leaching out of calcium hydroxide is also widely reported in concrete literature (Baker et al, 1991; Illson, 1994; Lea, 1976; Taylor, 1998; Young, 1998). There is no justifiable reason to expect that similar occurrences would not occur in CSBs.

Residues of unstabilised soil (usually clay) have been found in a stabilised block fabric (Herzog & Mitchell, 1963; Houben & Guillaud, 1994). Even within the recommended limit of less than 30% by weight of a block which is generally tolerated, the presence of clay is a potential source of problems. Owing to its fineness and high specific surface area (Chapter 3), not only can clay grains obstruct the stabilisation process, but they are also likely to compete for the mix-water required for the hydration of cement (Van Olphen, 1977). Clay can also coat the surfaces of coarse soil fractions (fine gravel and sand). Such coatings can inhibit the binding effect of cement on these particles. During rainy seasons, a block can rapidly absorb rainwater. The attraction of water by clay minerals has been explained by various mechanisms but ion exchange appears to remain the dominant mechanism (Carter & Bently, 1971; Franklin & Chandra, 1972; Ingles & Metcalfe, 1972). The amount and type of clay in a block can affect the degree of dispersion or flocculation. Kaolinite clays whose structure comprises platelets at a fixed distance are more stable in water, but are still capable of being disrupted. Ilite and montmorolinite clays on the other hand, which mostly contain interlayer potassium favour hydration in their dispersal (Houben & Guillaud, 1994). The swelling of clay lattice is known to assist in the mechanism of dispersal. Dispersed clay in a block fabric can easily be washed out as moisture permeates and circulates within it during rainy seasons.

The combined effect of leaching out of both calcium hydroxide and dispersed clays from a block is likely to be more severe in CSBs than in concrete. Extensive leaching is known to increase the porosity of a material (Neville, 1995). This can cause a block to become progressively weaker, and more permeable. A weakened block surface is more vulnerable to the direct abrasive action associated with driving rains. Since these mechanisms are likely to occur for the duration of the service lifetime of a block, deterioration over time can be expected.

The effects of leaching can however be minimised in blocks if certain preventive measures are taken early enough. These include the following:

- the use of pozolans and lime in combination with OPC during stabilisation. Pozolans and lime have the ability to fix both the calcium hydroxide present in hydrated cement paste and in any excess clay respectively (Hilt & Davidson, 1960). This approach is investigated experimentally in this thesis (Chapters 6 and 7).
- use of denser and more homogenous blocks of low permeability (less than 1.10^{-5} mm/sec) and of low water absorption capacity (less than 15%).
- careful soil selection that avoids use of soil with excessive clay content (<30% when OPC is used as the sole stabiliser).

- adequate curing of blocks.

Expanded product formation

Certain categories of chemical activity that can influence the durability of CSBs are associated with the formation of expanded products within a block. According to literature sources, such expanded products can occupy a greater volume within the block than the compounds which they replaced. By forcibly trying to occupy space that is not readily available, internal stresses can be generated within a block. Reactions of this category are well documented in concrete literature (Lea, 1970; Lea, 1976; Neville & Brookes, 1994; Illston, 1994; Neville, 1995; Sjostrom et al, 1996; Taylor 1998; Young et al, 1998). Apart from the occasional mention of the harmful effects of organic matter and other soil contaminants, no similar documentation of this phenomenon is covered in CSB literature. Yet the potential for such effects may be even greater in CSBs.

The three main categories of reactions likely to affect the durability of CSBs through expanded product formation include:

- Sulfate attack (on cement hydrates)
- Alkali-aggregate reactions (involving silica and carbonates)
- Soluble salts crystallisation (within the voids in a block)

Sulphates occur widely in natural soils in most parts of the world (Scot, 1965; Ingles, 1962; Ingles & Metcalfe, 1972; Jackson & Dhir, 1994). The type of sulfates vary greatly. But the common ones in soil are calcium, sodium and magnesium sulfates. These are mostly found in clayey soils rather than in sandy soils. The inclusion of significant amounts of sulfates in CSBs cannot be ruled out since no tests have so far been devised for their detection during soil selection. In the presence of sufficient amounts of moisture, sulfates present in soil can readily dissolve in water and react with certain hydrated cement products namely, calcium hydroxide and calcium aluminate (Neville, 1995). The dissolution of sulfates in water can create a sulfate solution within a CSB fabric. The sulfate solution might then react with both the $Ca(OH)_2$ and the hydrated C_3A to form calcium sulfate (gypsum), and calcium sulphoaluminates compounds (ettringite) respectively (Neville, 1995). The volume of these two by-products is much greater than that of the original substrates in the block. As these products expand in order to occupy more space within a block, and when this expansion is restrained by adjacent particles and phases within the core of the block, significant internal stresses are generated. The generated stresses are capable of disrupting bonding within the block. This can in turn result in a weakened block of lower strength, rigidity and hardness. The reactions are irreversible and their deleterious effects are noticeable within only a few years of their occurrence. The damage in blocks is commonly presented as defective edges and corners. These can also be followed by spalling and cracking of the block surface.

The severity of sulfate attack on CSBs depends on a number of factors. They include: type and amount of sulfates present in the soil, type of cement used, and the bulk properties of a block. The effect of sulfate attack on CSBs is not investigated experimentally in this thesis.

Alkali-aggregate reactions (AAR) can also be expected to occur in CSBs. According to literature sources, the reaction is essentially an inter-constituent material reaction also with the potential to form expanded products in a block. The reaction can occur between the active silica and carbonate containing soils and the alkalis (Na_20 and K_20) present in minute quantities in OPC (Glanville & Neville, 1997). Alkalis may also be present in remote amounts in most soils (ILO, 1987). Two kinds of alkali-aggregate reactions, both potentially harmful to blocks, are distinguished:

- Alkali-silica reactions (ASR)
- Alkali-carbonate reactions(ACR)

These phenomena and the mechanisms involved are also widely reported in concrete (Neville, 1995).

Defects on blocks resulting from AAR reactions will most likely appear as map cracking and spalling, occurring mainly on the surface of the block. Cracking of the star shaped pattern is the most common, but not necessarily the only type (Palmers, 1988). Factors likely to influence AAR reactions in CSBs include the following:

- availability of moisture
- high temperature environments (10°C-40°C)
- concentration of alkalis in cement and soil
- concentration of active silica and carbonates in soil
- porosity and permeability of the block

From the above factors, the main preventive measures for AAR in CSBs should involve procedures that attempt to lower the alkali content in the cement while it is still in the plastic state. The addition of pozolans to the soil-cement-water mix at the time of stabilisation could be helpful. The main reason for using pozolans is that they easily combine with the alkali content of the cement and soil, thus effectively lowering the alkali content. AAR can therefore be avoided in CSBs by using low alkali cements, non-reactive soils and pozolans blended with OPC (Glanville & Neville, 1997).

Soluble salts crystallisation (SSC) can occur within the pores and voids spaces of a block. According to literature source, the crystallisation of salts results in expanded product formation (Neville & Brooks, 1994). As before, such products have the potential to generate significant internal stresses within the pores and void spaces in a block. The phenomenon is widely reported in concrete literature (Sjostrom et al, 1996). Soluble salts are commonly found in most soils especially sandy soils. Sandy soils won from rivers can also contain appreciable amounts of soluble salts. Amounts as little as 6% of the mass of the sand are enough to trigger off such reactions (Neville, 1995). The most common salts are usually sulfates and chlorides (Neville & Brookes, 1994). Although these salts could easily be removed by washing of sand, the procedure is rarely followed in most developing countries. Sand is normally imported from various sources to improve the particle grading of soils needed for stabilisation (ILO, 1987; Rigassi, 1995). The soluble salts are however not reactive in the solid form in which they are normally present in the sand. They will only become reactive in solution. The alternate wetting and drying of block surfaces provides an ideal setting for such reactions. The mechanism of SSC is thought to be as follows.

When soluble salts in solution are present in a block fabric, they are likely to permeate into its capillary pores. Due to high temperatures leading to evaporation, moisture is driven off from the solution causing the salts to crystallise within the pores and voids spaces of the block. The volume of the crystals increase as the pore spaces get filled. But any further increase can be resisted by the rigid block fabric. This leads to the creation of significant stresses within the pores in the block. The induced stresses can cause cracking and disintegration at the surface of the block. Progressive deterioration of the block surface can then take place as moisture and temperature variations occur over the service lifetime of the block. The deterioration mechanism is known to be unaffected by the type of cement used (Jackson & Dhir, 1994). Limits on the soluble salts content of soil (especially its sand component) should therefore be specified during soil selection for CSB production.

Due to the threat from SSC, use of CSBs below the foundation level is still prohibited (Rigassi, 1995). Moreover, even blocks used at short distances above ground level in the lower courses of a wall may also be vulnerable to deterioration from SSC. The lower layers of a wall can be plastered to minimise such incidences.

Direct decomposition of the cement binder

Direct decomposition of cement within a CSB can occur due to attack from acidic conditions. No OPC is known to be resistant to acid attack (Neville, 1995). The direct decomposition of OPC can lead to the progressive break up of the bonds that hold the CSB fabric together and progress towards the interior. The phenomenon is widely reported in concrete literature (Jackson & Dhir, 1994).

In summary, apart from attempts to attribute common defects in blocks under service conditions to each of the chemical actions described, no attempt was made in the thesis to experimentally investigate their deleterious effects on blocks. Defects assessment conducted during the fieldwork confirmed the occurrence of chemically induced deterioration in blocks (Chapter 4). Further future research is recommended in the area of chemically induced deterioration in CSBs.

2.4 CONCLUSION

From the preceding discussions in Chapter 2, it can be concluded that the concept of durability and its expression are not well covered in CSB literature. It is proposed that expressions of durability in CSBs should revolve around three factors, namely: intended function of a block, the expected service conditions, and the time taken to satisfactorily fulfil the functions.

It was established through literature reviews in Chapter 2 that even under normal service conditions, deterioration agents can still influence the durability of a block. Under more severe conditions of exposure such as in the humid tropics, the effects of deterioration agents can lead to the premature deterioration of blocks. The durability of a block can therefore be regarded as its ability to resist deterioration. It was noted that due to deterioration, the durability of a block is not likely to remain constant, but can vary over time. Performance characteristics which were initially deemed satisfactory at the time of production can alter appreciably for the worse over time. Durability and deterioration therefore influence each other mutually but negatively.

According to literature sources, it was noted in Chapter 2 that the principal agents likely to influence the performance of a block while in service include: rainwater, temperature, and chemical action. The exact mechanisms of these actions are not yet fully understood. Their combined and interdependent action in causing loss of quality in a block is thought to be highly likely. It was further noted that water and temperature related deterioration mechanisms represented the main forms of deterioration in CSBs in the humid tropics. Water-related action not only causes loss in mass on the block surface due to wetting and abrasion, but also contributes to the initiation and propagation of otherwise dormant chemical activity. Water related deterioration was found to occur in various forms: solvent action, abrasive action, swelling action, catalytic action and dampness. Temperature related deterioration on the other hand causes volume changes which lead to the creation of cracks and weakening of the block fabric. Various mechanisms involving expansion and contraction, shrinkage and drying, and catalytic action were discussed. It was noted that high temperatures are linked with the speeding up of harmful chemical reactions. The combined action of wetting-abrasion and drying are investigated experimentally in this thesis (Chapter 7). It can be concluded that the mechanisms of water and temperature related deterioration are neither well covered nor properly understood in current CSB literature.

It was discussed in Chapter 2 that chemical action related deterioration mechanisms in blocks remained the least investigated and documented of all deterioration modes. Yet such reactions are potentially possible in CSBs due to the various minerals found in soils and OPC hydrates. Three categories of potential chemical reactions with deleterious effects were identified: reactions resulting in expanded product formation (sulfate attack, alkali-aggregate reactions, soluble salts crystallisation), reactions resulting in the direct decomposition of the cement binder (acid attack), and reactions resulting in the leaching out of substrates ($Ca(OH)_2$ and clay minerals). It will not be possible to experimentally examine these chemical action related deterioration mechanisms during this research. However, defects arising from their effects are investigated through the field exposure condition survey described in Chapter 4. Further research is recommended on all aspects of chemically-related deterioration mechanisms.

From these brief conclusions, the objectives of Chapter 2 were met.

CHAPTER 3

CEMENT-SOIL STABILISATION

3.1 INTRODUCTION

In Chapter 3, current documented principles and practice of cement-soil stabilisation as applied to the production of CSBs are discussed. Soil requires to be stabilised because the material as found in its natural state is not durable for long-term use in buildings. By properly modifying the properties of soil, its long-term performance can be significantly improved (Bureau of Reclamation, 1975; Dunlap, 1975; Herzog & Mitchell, 1963). Soil stabilisation processes focus on altering its phase structure, namely the soil-water-air interphase. The general goal is to reduce the volume of interstitial voids, fill empty voids, and improve bonding between the soil grains. In this way better mechanical properties, reduced porosity, limited dimensional changes, and enhanced resistance to normal and severe exposure conditions can be achieved (Gooding & Thomas, 1995).

The objective of this chapter is to closely examine current methods of soil stabilisation in general, and their application to CSBs in particular. The chapter describes the fundamental theoretical background on which subsequent experimental investigations which follow in Part B of the thesis are based. The approach used in Chapter 3 is to first identify and examine each of the three main constituents of CSBs (soil, cement, water), then evaluate existing methods used for combining them during the block production process.

Chapter 3 is presented in five sections. After this introductory section, subsequent sections describe the following: main constituent materials, cement-soil stabilisation principles, the block production process, and conclusion.

3.2 MAIN CONSTITUENT MATERIALS USED IN THE PRODUCTION OF CSBs

The three main constituent materials used in the production of CSBs are:

- Ordinary Portland Cement (for binding the soil particles)
- Soil (for the skeletal structure of the block)
- Water (for the hydration of cement and lubrication of soil particles

These three materials each have unique properties. Before discussing how they are combined to form blocks, a description of their nature and properties is presented. This approach is considered relevant because in the past, the individual properties of these materials were more or less taken for granted, with unfortunate consequences for blocks. The quality of material used and their proportioning can significantly affect the durability of blocks. Each material is therefore discussed in turn in Sections 3.2.1, 3.2.2 and 3.2.3 that follow.

3.2.1 ORDINARY PORTLAND CEMENT AS THE MAIN BINDER

Ordinary Portland cement (OPC) plays such a critical role in the performance of CSBs that the following aspects of the binder are briefly examined:

- Function of OPC in CSBs
- Physical properties of OPC likely to affect its performance
- Basic chemical constituents of OPC
- Hydration reaction of OPC following the addition of water

- Properties and influence of the hydration products on the durability of blocks
- Use of cement replacement materials

Functions of OPC in CSBs

Ordinary Portland cement is an important ingredient and variable in a CSB. Without its inclusion, compressed blocks would be no different from common sun dried mud blocks and would simply disintegrate on contact with water, or when subjected to moderate impact loads. Compared with concrete products where 12-18% by weight of cement is used, only about half of that amount (5-8% by weight), is required in stabilised blocks (ILO, 1987; Webb & Lockwood, 1987; Houben & Guillaud, 1994). Though not commonly recommended, amounts as low as 3% and as high as 10%, have been used depending on the nature of the soil requiring stabilisation (Rigassi, 1995).

The function of OPC is to strongly bind the constituent materials (soil particles) together, in a dense, strong, dimensionally stable and durable unit. Other common binders currently in use include lime, gypsum, pozzolans, resins and bitumen (Apers, 1983; Stulz & Mukerji, 1988). Discussions in this thesis will be restricted to the use of OPC as specified in BS 12, 1971 and ASTM C 150-92. OPC has been selected for two reasons. Firstly it has a unique and superior binding capacity. Secondly, it is widely available in most parts of the world.

The uniqueness of OPC in comparison with other binders is based on its ability to achieve extremely high strengths in only a short period of time (about 28 days). OPC stabilised blocks remain dimensionally stable even when in contact with water in a manner not possible for comparable unstabilised blocks produced in a similar way. Uncontrolled swelling and shrinkage are appreciably contained when OPC is used.

For stabilised blocks, variations in OPC quality and amount can drastically affect its properties and behaviour more than any other input variable (Gooding, 1994). The effect of varying the amount of OPC on the performance of CSBs are investigated experimentally (Chapters 6 and 7).

Unfortunately the manner of current coverage of OPC in CSB literature leaves a lot to be desired. The coverage is so limited, scanty and routine that widespread and incorrect use of the binder is now becoming the order of the day (Fullerton, 1979; BRE, 1980; Spence & Cook, 1983). Problems associated with the misuse of OPC are also described in Chapter 4 (findings from the fieldwork). Due to poor coverage, critical phenomena in cement chemistry such as the need for adequate amounts of water to ensure complete hydration of cement, and proper conditions to ensure preservation of moisture within the block to facilitate completion of the hydration process, are overlooked. For these and other reasons to be mentioned later, this section attempts to redress the shortcomings brought about by the narrow coverage of the subject in CSB literature.

Physical properties of OPC

Two of the most important physical properties of OPC are its:

- Specific surface area (SSA), and
- Particle size distribution (PSD)

The SSA and PSD of cement are important to CSB production because they govern the manner in which the binder stabilises soils. These physical properties are directly related to the process of manufacture of the binder. The basic source materials for OPC are a mixture of about 75% limestone ($CaCO_2$) and about 25% clay (Lea, 1976). These are intimately mixed, then ground together. In the modern manufacture of

OPC, the ground mixture is fed into a rotary kiln against a counter flow of hot air, and heated to about 1450-1800K (Neville, 1995; Taylor, 1998). The resulting melts from the mixture coalesce to form clinker, of approximate dimensions 5-10 mm. After being allowed to cool, the clinker itself is then mixed with about 3-5% gypsum ($CaSO_4$) (Taylor, 1998; Young, 1998). The gypsum is added to control the otherwise spontaneous capacity for initial setting. The mix is finally finely ground to give the powdery form in which OPC is traded. There can be as many as 1.1 x 10^{12} particles or grains of OPC per kilogram after the grinding process (Neville & Brookes, 1994). It is the grinding process that determines the SSA and the PSD of OPC. These are now discussed each in turn, with implications for the stabilisation of soil emphasised in each case.

The *specific surface area* (SSA) of OPC is in the range 300-350 m^2kg^{-1} (Lea, 1976; Illston, 1994; Jackson & Dhir, 1996; Taylor, 1998). This is lower than that of rapid hardening cement (RPC) which falls in the range 400-450 m^2kg^{-1}. Since the hydration reaction of cement while stabilising soils starts from the surface of the grains and then proceeds inwards, the higher the SSA, the faster can the rate of reaction be expected. The hydration reaction proceeds uninhibited if the cement grain surfaces are free. However, due to the large range of particles of varied SSA in soil, the likelihood of surface blinding is high. Fine sand and medium silt have SSA between 0.02 and 0.23 m^2g^{-1}, while clay grains have between 10 and 1000 m^2g^{-1} (lowest in kaolinite, highest in montmorollinite) (Akroyd, 1962; Grimshaw, 1971; Head, 1980). Interference due to the blinding of cement grains by any of these substances, and thereby inhibiting hydration, is likely but undesirable. Moreover, the large SSA of clay present in soil ensures that they can attract water in the mix otherwise exclusively meant for the hydration of cement, thus reducing the amount of water going directly to hydrate cement. This can impair the hydration process, with negative implications for strength development in CSBs. For this reason for example, use of clay contaminated aggregates is prohibited in concrete production (Jackson & Dhir, 1996). It would however, be highly uneconomical and impractical to try to eliminate clay from its parent soil before use in CSB production.

The *particle size distribution* of OPC is characterised by an almost uniform type of grading (or packing characteristics). Cement grains do not contain every size fraction between the maximum and minimum particle sizes. Approximately 90% of the cement grains in OPC measure more than about 5μm, with only 1% measuring less than about 90 μm (Weideman et al, 1990; Glanville & Neville, 1997). The average size of most OPC cements is about 10 μm. This can be compared to the size of the very fine clay particles in soil with average size less than 2 μm (Houben & Guillaud, 1994). The detrimental effect of the presence of clay has been mentioned already. Despite the potential setback, the use of clay in CSBs is likely to continue to be tolerated.

Basic chemical constituents of OPC

Identification of the main constituents of OPC, and description of their role in influencing the hydration reaction leading to the stabilisation of CSBs has received no previous coverage in CSB literature. From concrete literature sources, the summary of the main constituents, together with their approximate quantities and role in the hydration reaction, are shown in Appendix A.

The listed constituents of OPC are impure reactive minerals which exist as multi-component solid solutions (Weidemann et al, 1990; Taylor, 1998). The implication is that the reactions of these ingredients following the addition of water, and mixing with soil particles, is likely to be quite complex. As can be expected, not only will each of the ingredients react separately with water, but they can also influence the manner in which the others (including raw soil minerals), react with each other. In view of this, the exact mechanism of cement stabilisation of soil remains poorly understood and is likely to become a subject of active research for years to come.

Hydration reaction of OPC on addition of water

The hydration reaction that ensues when water is added to unreacted OPC grains follows two distinct phases: setting, followed by hardening, to form a cement paste (Young et al, 1998). The mechanism of the reaction involving the main phases in unreacted cement (C_3S, C_2S, C_3A and C_4AF), occurs at different rates for each phase. The process is reported to evolve as follows (Lea, 1976; Weidemann et al, 1990).

When water is added to the OPC grains, the reaction begins from the surface of the grains, then progresses inwards. This results in the formation of gels and ettringite (Taylor, 1998). The C_3S and C_2S form gels, while the C_3A form ettringite. Due to the increased contact between the formed gels on adjoining grains, and due to the interlocking of the ettringite crystals, cohesion develops signifying the start of the setting period (initial setting). This occurs within about 45-60 minutes of water being added (Illston, 1994). The implication of this is that the soil-cement-water mix should be compacted before the initial setting begins. The water continues to diffuse into the gels, causing pressure to build up within, resulting in rupture of the gel. The ruptured gel peels away from the grain, forming gel foils and fibrils as wells tubules in the case of C_3A. This exposes the grain surface locally to further hydration. The process then repeats itself. As each grain sprouts a multitude of these fibres and as they continue to grow and multiply, they start to interlock even more closely and rigidly than before. This signifies the end of the setting period. The final set occurs approximately 12 hours after water was initially added (Illson, 1994). The paste is however, still weak and therefore of low compressive strength. The implication of this is that freshly demoulded green blocks will remain weak at this stage and should therefore be handled with extreme care.

The paste then starts to harden. The continued hardening of the paste is due to the further multiplication, growth and stronger interlocking of the gel fibres, ettringite and portlandite crystals. Hardening is effectively 'completed' about 28 days after the initial contact with water. Although further hardening takes place after this at a decelerating rate, the defining timing is specified as 28 days. At 28 days, the paste, and by implication the block, should be hard enough to be used. The chemical reactions involving the four phases in OPC are summarised as follows (Lea, 1970; Illston, 1994; Neville & Brookes, 1994; Young, 1998; Taylor, 1998):

<u>For C_3S</u>:

$$2\,C_3S \quad + \quad 7\,H_2O \longrightarrow C_3S_2H_8 \quad + \quad 3\,Ca(OH)_2 \quad \Delta H\ 500 J.g^{-1}$$

tricalcium silicate + (water) → calcium silicate hydrate + calcium hydroxide heat of hydration

<u>For C_2S</u>:

$$2\,C_3S \quad + \quad 5\,H_2O \longrightarrow C_3S_2H_8 \quad + \quad 3\,Ca(OH)_2 \quad \Delta H\ 250 J.g^{-1}$$

dicalcium silicate + (water) → calcium silicate hydrate + calcium hydroxide heat of hydration

The reaction is accompanied by a high rate of heat evolution (exorthermic reaction), silicate polymerisation, rapid increase in [OH^-], and a concomitant rise in pH number to 12.6 (Neville, 1995). Both silicates are reported to require approximately the same amount of water for their hydration. The C_3S silicates generate about twice the amount of $Ca(OH)_2$ than the C_2S silicate. The release of $Ca(OH)_2$ has direct implications on the durability of CSBs (Chapter 2). The calcium silicate hydrates are fine amorphous particles in a colloidal state, often represented simply as C-S-H to emphasise their indeterminate nature as no specific formula is considered to be that accurate (Taylor, 1998).

<u>For C_3A</u>:

The reaction involves not only water, but also gypsum and the extra ettringite produced as a result of the interaction. The two reactions are thought to proceed as follows:

1. $C_2A \; + \; 3\,C\bar{S}H_2 \; + \; 26\,H_2O \longrightarrow C_6\,A\bar{S}_3\,H_{32} \qquad \Delta H = -1350 J.g^{-1}$

(tricalcium aluminate) + (gypsum) + (water) → (ettringite)

The excess ettringite formed reacts with more C_3A,

2. $C_6\,A\bar{S}_3\,H_{32} \; + \; 2\,C_3A \; + \; 4\,H_2O \longrightarrow 3\,C_4\,A\bar{S}H_{12}$

(ettringite) + (tricalcium aluminate) + (water) → (monosulphoaluminate)

Monosulphate aluminate is known to be the most stable phase in a mature cement paste (Young et al, 1998).

<u>For C_4AF</u>

The reaction involves both water and $Ca(OH)_2$ as produced before. Thus,

$$C_4\,AF \; + \; 2\,Ca(OH)_2 \; + \; 14\,H_2O \longrightarrow C_4\,(A, F)\,H_{13} \; + \; (A,F)\,H_3$$

(ferrite) + (calcium hydroxide) + (water) → (tetracalcium aluminate hydrate) + (ferric aluminium hydroxide)

The $C_4 (A,F) H_{13}$ is structurally related to the monosulfoaluminate, while the $(A,F) H_3$ remains amorphous (Taylor, 1998; Young et al, 1998)..

Properties and influence of the hydration products

As can be expected, the products of the hydration reaction of OPC can have major influences on the properties of the hardened cement paste and by implication, on the behaviour of CSBs. A summary of the hydration products and their likely impact on the durability of CSBs are shown in Appendix B. From this summary, various ways of improving the properties of hardened cement paste, and by implication the block, can be examined.

It is widely reported in concrete literature that the volume fraction of the cement hydrates, gel pores and capillary pores determine the properties of concrete (Baker et al, 1995; Sjostrom et al, 1996; Neville, 1995; Young et al, 1998). Given the similarities between concrete and CSBs, the same influence is likely to apply in the case of the latter. The role of hardened cement paste in both materials appears to be central to this hypothesis. The volume fraction of solids, gel and capillary pores are in turn determined by two factors : the water cement ratio (w/c) and the degree of hydration (α) (Illson, 1994; Taylor, 1998).

It is the w/c ratio that controls the porosity of hardened cement paste. Theoretically, the production of CSBs of high strength and of low permeability can therefore be achieved by:

- Lowering the water cement ratio
- Assuring a high degree of hydration

Low *w/c ratio* can be achieved by adequate proportioning of the main raw material ingredients; soil, cement and water. A particularly low w/c ratio can also be achieved in mixes by using partial cement replacement materials (CRM) (Neville, 1995). In situations where blocks are to be exposed to aggressive environmental conditions, such approaches are likely to be more beneficial than the short-term economics of increase in production costs. This is because mixes incorporating fine cement replacement materials are likely to result in high performance blocks which are more dense and more durable than blocks produced in the traditional manner. This theory is investigated experimentally in this thesis and the results are discussed in Chapters 6 and 7.

A *high degree of hydration* of cement paste in a block can be achieved by making sure that proper curing is done. The degree of hydration represents the fraction of the original cement grains which have hydrated. This requires that a sufficient length of time is dedicated for curing alone. During this time the green demoulded blocks should not be allowed to dry out quickly. This should help avoid causing the water still in the green blocks to evaporate, thus remaining available for the hydration reaction to continue. Adequate curing and low water/cement ratio are therefore potentially significant ways of improving the properties and performance of CSBs. The effect of varying curing conditions on the properties and performance of CSBs are investigated experimentally, with the results discussed in Chapters 6 and 7.

Modification of CSB properties using cement replacement materials

As discussed earlier, the properties of hardened cement paste can be improved by the partial replacement of OPC with CRMs. In this sub-section, the manner in which CRMs are able to modify properties of hardened cement paste, the types of CRMs available, their physical and chemical properties, and the mechanism of the reactions involved, are presented.

CRMs, sometimes referred to as mineral admixtures or additives, can be used as substitutes for some of the OPC in a block. CRMs modify properties of OPC by altering its setting and hardening behaviour (Neville, 1995). Improvements in strength and durability of the hardened cement paste is reported to be due to the pore-filling effect of the CRMs, which effectively lowers both the capillary and gel porosity in the resulting product (Young, 1998). Further, the transition zone between the sandy fraction of the soil and the cement paste, usually a major point of weakness, is likely to be considerably strengthened by the pozolanic reaction of CRMs. Amounts as low as 10% to 20% of the cement content are required. For blocks expected to be used in harsh environmental conditions, CRMs could be used only on the surface areas to reduce costs.

The *main types* of CRMs include the following (Jackson & Dhir, 1996; Neville & Brooks, 1994):

- Pulverised fuel ash (PFA), or fly ash
- Ground granulated blast furnace slag (GGBS)
- Microsilia, or condensed silica fume (CSF or MS)
- Natural pozolans, e.g. volcanic ash
- Calcined clay and shale
- Risk husk ash

The most commonly used of the above are the PFA, GGBS and CFS. They are all available as industrial materials, or as blends with OPC. The basic physical properties and chemical composition of PFA, GGBS and MS as compared to OPC are described in Illston (1994) and Neville (1995). The three CRMs contain a substantially greater amount of silica (SiO_2) than OPC (PFA, 48%; GGBS, 36%; MS, 97%; OPC, 20%).

Most importantly, most of the silica contained is in the active amorphous form (Neville, 1995) (a pozzolan by definition is a material that contains active silica, SiO_2). The silica is in its amorphous or glassy form in disordered structures. This is distinguished from the uniform crystalline structure form of silica found in sand. The latter is not chemically active and is therefore regarded as being dormant. It is only MS, with 97% SiO_2 and no CaO in its composition which entirely comprises active silica. For this reason, it is this CRM which is discussed further, and its effect on cement is investigated experimentally in Chapters 6 and 7.

The *physical properties* of MS make it one of the most effective CRMs. Firstly by having a significantly lower specific gravity than OPC (2.2 compared to 3.15), the substitution of the latter on a weight by weight basis with the former is likely to result in a greater volume for the resulting paste. This pore filling effect is just what is needed for surface layers of CSBs to prevent easy penetration of moisture. The pore filling effect is also known to enhance strength properties (Taylor, 1998). Secondly, by having a particle size range about three magnitudes of order below that of OPC (0.01-0.5 μm compared to 0.5-100 μm), the MS has the potential of ensuring more dense packing of the binder. Added to this, with its considerably higher specific surface area (15,000 compared to 350 m^2kg^{-1}), the speed of hydration is likely to be significantly higher. This is probably due to the small MS particles acting as nucleaic sites for the deposition of the C-S-H on the hardened cement paste. This can be quite useful in CSB production as it can effectively lower the water cement ratio in a damp mix. The lower the w/c ratio, the higher the strength (Webb & Lockwood, 1987; Baker et al, 1994; Taylor, 1999).

The *mechanism of the reaction* of the pozzolan mixed with OPC is as follows (Taylor, 1998). The pozzolan itself is not known to be cementitious on its own but in a finely divided form, and in the presence of moisture it can chemically react with the $Ca(OH)_2$ liberated during the reaction of the silicates in OPC. The reaction forms a fresh secondary set of calcium silicate hydrates whose properties are the same as those of the primary reactions. This represents one of the most significant reasons for partial substitution of OPC with CRMs. As was mentioned earlier, the $Ca(OH)_2$ liberated during the hydration reaction of OPC is a potential source of instability for the CSB. Being soluble in water, it can easily be leached out from the block, increasing its porosity and weakening the bond strength (Chapter 2).

The chemical equation representing the mechanism of the reaction is thought to be as follows (Illston, 1994):

$$\underset{\left[\begin{array}{c}\text{amorphous}\\ \text{silica}\end{array}\right]}{S} \quad + \quad \underset{\left[\begin{array}{c}\text{Calcium}\\ \text{hydroxide}\end{array}\right]}{Ca(OH)_2} \quad + \quad \underset{\text{(water)}}{H_2O} \longrightarrow \underset{\left[\begin{array}{c}\text{calcium silicate}\\ \text{hydrate}\end{array}\right]}{\text{C-S-H}}$$

The C-S-H from the secondary reaction can contribute to further strength development, and by implication to the durability of the cement paste within a CSB fabric. The effect of MS on the properties and performance of CSBs are investigated experimentally in this thesis and the results discussed in Chapters 6 and 7. Blocks containing MS are referred to as improved blocks (IPD) (Chapter 5).

3.2.2 CHARACTERISATION OF SOIL FOR CSB PRODUCTION

As stated earlier in the thesis, soil alone constitutes over 90 of the bulk of a CSB. Existing CSB literature appear to adequately cover fundamental theories of soil properties and behaviour.

According to BS 1377 Part 1: 1990, soil is an assemblage of discrete particles in the form of a deposit, usually of mineral composition, but sometimes of organic origin, which can be separated by gentle mechanical means, and which include variable amounts of water and air. Soil is also referred to as the loose material that results from the long-term transformation of the underlying parent rock by the simultaneous and evolutionary interaction of climatic factors and other physico-chemical and biological processes (Casagrande, 1947; Das, 1994; Houben & Guillaud, 1994; Craig, 1998). Most of soils consist of disintegrated rocks, decomposed organic matter and water soluble mineral salts. These descriptions confirm that soil is a highly variable and complex material in nature. Although soil properties can be modified to improve their performance, not all soils may be suitable for stabilisation as found. The decision on suitability requires the identification of the main constituents in the soil likely to have a direct bearing on its properties and behaviour (Head, 1980).

As the characteristics of a soil can affect the performance of a CSB, the review of literature discussed in this section will focus on the following:

- Composition of soils
- Classification of soil (to determine type and suitability)
- Current criteria for selection of suitable soils

Composition of Soils

Soil consists of three main phases, namely, solids, liquids and gasses (Das, 1994; Craig, 1998). The solids form the bulk of the material, while the liquids and gasses mainly fill the void spaces. The relative proportions of the three phases can have a significant influence on the behaviour of a soil. In this section only the composition of the solids fraction are discussed. Soils are made of varying proportions of four types of solids: gravel, sand, silt and clay (Fitzmaurice, 1958; Enteiche & Augusta, 1964; ILO 1987). Each of these different solids are briefly described in turn in the ensuing paragraphs.

Gravels are the larger granular particle sizes in a soil forming its skeletal structure. They range in size between 2mm and 20mm (BS 1377 Part 2: 1990). They are the cohesionless part of a soil resulting from the direct disintegration of underlying parent rocks and pebbles (Pettijohn, 1957). Gravels have a rough texture and may be found in almost all shapes, including rounded, angular, irregular, etc. (BS 1377 Part 2: 199). Due to their loose packing and stability, they are important for CSB production as they limit shrinkage and capillarity in a soil. Amounts of gravel in excess of 10% are not recommended for use in CSB production (Rigassi, 1995). The allowable maximum size fraction for gravel used for CSB production is not standardised. Some literature sources recommend 15 mm to 20 mm (Houben & Guilland, 1994), while others recommend 6mm (ILO, 1987). As will be shown later, the maximum size fraction has a considerable influence on the bonding properties of a soil-cement mix (PSD).

Sand particles in a soil range in size between about 0.06mm and 2mm (BS 1377 Part 2: 1990). The sandy fraction comprises granular grains of silica or quartz from the disintegration of sandstones and crystalline rocks. Sandy soils are very stable, lack cohesion, are non-sticky with a gritty texture. They also have a very high degree of internal friction and do not shrink. Because of these properties, they can provide the much needed mechanical strength to soil. Their pressure also limits swelling and shrinkage in a soil. According to literature sources, the specific bulk density of sands vary between 2500 kg/m^3 and 3000 kg/m^3, but their specific surface area is about 23 cm^2/g, and specific heat about 800 J/kgK (Jones, 1984; Houben & Guillaud, 1994). The recommended proportion of sand in soils for CSB production varies but is mainly between 70 and 80% (Lunt, 1980; Rigassi, 1995).

Silts are made up of particles whose size range varies between 0.002mm and 0.06mm (BS 1377 Part 2: 1990). Apart from the difference in size, silts are almost identical in nature to sandy particles. Their internal friction is however noticeably less than that of sand. Their specific surface area is about 454 cm^2/g and density between 1600 and 1800 kg/m^3 (Head, 1980; Houben & Guillaud, 1994). They have a smooth texture, are sticky and lightly cohesive, but their shrinkage capacity is not significant. Due to their lack of cohesion, gravels, sands and silts should not be used on their own for CSB production (reasons discussed under clay). According to literature sources, the recommended silty fraction in a soil for CSB production should be between 10 and 25% (Rigassi, 1995).

Clay particles form the finest fraction of soils with average sizes less than 2 μm (Scot, 1963). Their physical and chemical characteristics are not similar to those of the other three soil fractions. The specific surface area of clay is about 800 m^2/g, while their specific heat is about 965 J/kgK (Houben & Guillaud, 1994). Due to their fine grained nature, clays are cohesive and will form a coherent mass at suitable moisture contents (Vickers, 1983; BS 1377 Part 1: 1990). Clays are known to contribute to some of the important engineering properties in a CSB, and for this reason, are discussed in more detail. They are basically hydrated alumino-silicates of irregular but often hexagonal shapes (Torraca, 1988). Large clay molecules comprise a series of sheets or wafers of alumina and silica which are not electrically neutral (Van Olphen, 1977). The sheets have chemical make-up which varies according to the type of clay. Three major clay types exist; namely kaolinite, illites and montmorollinite (ILO, 1987; Houben & Guillaud, 1994). Within these three major types, there are about 20 different sub-groups of clay (Scot, 1963).

Kaolinite and montmorollinite represent the opposite extremes in the behaviour of clay when in contact with water. Kaolinite which is almost pure clay, is the most stable and therefore the least expansive of clay types. Since the two layer wafer has a fixed distance of about 7 A°, they are held together firmly. Its linear contraction is small, ranging between 3% and 10% only (Houben & Guillaud, 1994). Because of these properties, kaolinite clay was selected and used for blending the experimental soil type used for laboratory tests in this thesis (Chapter 5).

Illites and montmorollinites consist of type 2-1 structures which are more weakly held together. They are not stable in water and are highly vulnerable to swelling and shrinkage. Their linear contraction values are high, ranging between 4% and 11% and between 12% and 23% respectively (Das, 1994). The linear contraction value of the latter is more than twice that of kaolinite. No further tests are conducted in this thesis using these two clay types.

Generally, the presence of clay in moderate amounts in a soil is desirable (Smith & Smith, 1998). Being cohesive, they impart plasticity to the soil when under moist conditions. Plasticity is due to the thin film of absorbed water which adheres strongly to the clay layers thus linking the particles together (Grimshaw, 1971; RILEM, 1972). In this way, the clay minerals acts as natural binding agents for the cohesionless granular fractions of a soil (gravel, sand, and silt). This quality is particularly valuable during the production of CSBs. Green blocks after demoulding are still weak as the cement binder may not yet have had sufficient time to set. The presence of clay as a natural binder thus helps in the handleability of CSBs at this stage of the production process (Spence & Cook, 1983).

Clay minerals also have other properties which are unfortunately considered undesirable in a block. Being hydrophilic they have a very high affinity for water.

As the wafers attract water, clay particles can slide over each other resulting in an apparent increase in volume (due to dispersion). Conversely, as the clay wafers dry out, they shrink, causing cracks to appear in the clay mass, thus effectively irreversibly breaking their bond strength (Hilt & Davidson, 1960). Extreme swelling and shrinkage are not desirable properties in blocks. It is the uncontrollable swelling and shrinkage of clays which depend on moisture and temperature variations that makes them unique, and thus difficult to deal with. For this reason, most CSB literature sources recommend controlling the amount of clay in soils to be used for block production. Amounts of clay in excess of 40% is not recommended for soils for CSB production using OPC (Rigassi, 1995). In such cases, the use of lime is recommended due to its ability to fix the clay through a pozolanic reaction (Ingles, 1962; Spence & Cook, 1983; BS 1924, 1990).

Soil Classification

The classification of a soil is the first requirement needed to identify it. Knowledge of the soil type and properties can facilitate the optimisation of its use in CSB production. According to literature sources, soil classification is performed on particles nominally less than 60mm (Dunlap, 1975). Soils are classified in various ways depending on the prevailing local or regional standards in the particular part of the world. Whatever the geographic location, however, some common procedures are usually adopted in the classification of soils.

Soil classification methods are based on either one or a combination of the following: particle size distribution, plasticity, compactability, cohesion, and organic matter content (Casagrande, 1947; Fitzmaurice, 1958; Head, 1980; Vickers, 1983). Unfortunately, soil classification systems are not yet uniformly applied internationally. At the moment, the two classification systems widely used are based on the particle size distribution and on the plasticity of a soil (Lunt, 1980). According to these two methods, the soil size grading and its plasticity are divided into clearly defined ranges. For each range, a descriptive name and letter is assigned to the identified soil type to distinguish it from the others.

In the *particle size distribution* classification system, the term particle refers to an individual mineral grain within the disturbed soil mass. According to this system, the following size ranges are given (BS 1377: Part 2, 1990):

Name	Subdivision	Diameter of Particles (mm)	
Gravel	Course Medium Fine	60 20 6.0	20 6.0 2.0
Sand	Course Medium Fine	2.0 0.6 0.2	0.6 0.2 0.06
Silt	Course Medium Fine	0.06 0.02 0.006	0.02 0.006 0.002
Clay			<0.002

Table 1: Soil classification according to particle size distribution
(BS 1377 Part 2 1990; ILO, 1987)

In this system, each of the terms gravel, sand, silt and clay refers to a range of particles or grain sizes in a soil (table 1). In actual reality, soils are not found in this rigidly defined state. The normal natural state of soil is such that it is composed of grains from two or more particle size ranges. For example, a soil may be described as sandy CLAY, implying that the soil has significant amounts of sand and clay size ranges. In such a case, the size range with the higher percentage of particles is the last named range in the soil description (in capitals). The main terms used in this system are G for GRAVEL (60-2mm), and S for SAND (2-0.06mm). The qualifying terms are W for well graded and P for poorly graded (Pu, uniform; Pg, gap graded).

In the *soil plasticity* classification system, the soil is identified by its behaviour when in contact with water. The system is mostly used for the finer fraction of a soil, i.e. clays and silts smaller than the 425 μm sieve only (Houben & Guillaud, 1994). The soil is then classified using a soil plasticity chart (Stulz & Mukerji, 1988; Das, 1994). The advantage of this system is that it recognises the formation, and therefore the behaviour of soil groups can be predicted easily.

Use of both the particle size and plasticity based classification systems are recommended in analysing soils for CSB suitability. The crucial point is that either system should be able to describe the soil in a manner clearly understandable by engineers. The identification of a particular soil can be considered complete with the inclusion of its colour, particle shape and composition, soil name based on its grading and plasticity, and the soil group symbol. The approach is useful in that it clearly recognises the difference between the coarse and fine soil fractions.

Experience has also shown that within each class of soil, similar characteristics are displayed. With coarse grained soils (gravel and sand) for example, the similar characteristics are dependent not only upon the size of the particles, but also on the manner in which the sizes are distributed within the soil mass. With fine grained soils (silt and clay), it is the moisture content of the soil and the clay mineral type which play more significant roles in determining the behaviour of the soil (Ingles & Metcalf, 1972).

Current criteria for selection of suitable soils

The varied composition and properties of a soil in its natural state introduces difficulties during the selection of the material for stabilisation. As stated earlier, not all soils found in their natural state will necessarily be suitable for CSB production. In this sub-section the basic suitability requirements are outlined and the existing criteria for selection based on literature sources are summarised.

Basic suitability requirements are varied with a broad set of requirements proposed in CSB literature. For a soil to be suitable for stabilisation, its particle size distribution, plasticity and compressibility should be desirable. Existing suitability criteria require that the soil be:

- well graded with a continuous or dense gradation. It should be neither gap-graded nor uniformly graded. The size of the maximum soil particle should be less than 6 mm in diameter (ILO, 1987). Particle sizes greater than this size may easily get dislodged from the block fabric due to poor bonding. The gravel and sandy fraction should be densely packed not only to provide the skeletal structure of the block, but also to take up applied loads. The silt and clay fraction in a soil should be adequate enough to provide sufficient cohesion. As stated earlier, this is valuable when demoulding green blocks and when handling them during wet curing. In addition to the amount of clay in a soil, its type should also be ascertained. Not all clays have the same degree of shrinkage and swell. This property of clay is a potential source of disruption for the future performance of a block. Generally, when using OPC as the stabiliser, best results can be obtained with predominantly sandy soils. When using lime as the binder, best results can be obtained with predominantly clayey soils. It is however still possible to improve any poor gradation of a soil prior to stabilisation. This can be achieved by adding the missing fraction, or by removing the excessive fraction. The removal of excessive coarse fraction can be done by sieving the soil, while removal of fines from the coarse fraction can be done by washing it.

- of low plasticity index, thus able to exhibit low rather than high cohesion. Sandy soils of low clay content (about 10% or less) do not have an appreciable plastic limit. The clay content once again is the critical factor. Soils of high plasticity can have liquid limit in excess of 50% (Fitzmaurice, 1958; Houben & Guillaud, 1994). The corresponding clay content in such cases may be in excess of 40%. The plasticity index of a soil can be altered by modifying its particle size distribution. The plasticity index can be lowered by adding sand, and raised by adding clay (Rigassi, 1995). Adequate plasticity facilitates shaping a soil as it determines its ability to remain in close association, thus contributing to moulding and handling.
- compacted at its optimum moisture content for the maximum dry density to be attained (Guillaud et al, 1995). At the maximum dry density, the porosity of the soil is at its minimum. This increases both its shearing strength and compressive strength on loading. For every soil, reduction in porosity attained at maximum dry density will depend on the gradation of the soil, its optimum moisture content and on the compaction energy used (Ingles & Metcalf, 1972; Vickers, 1983).
- free of soluble salts and organic matter. These impurities can have harmful effects on OPC both during hydration and even after hardening. Presence of organic matter higher than about 1% represents a potential hazard (Houben & Guillaud, 1994). Organic matter is harmful because it contains nucleic acids,

tarturic acid and sometimes glucose. These substances can interfere with the proper setting of OPC, thus weakening the hardened cement paste (Neville, 1995). Soluble salts and sulfates can react with moisture in a soil and hardened cement past resulting in expanded product formation in a block (Chapter 2). Soils with soluble salts and sulfate contents higher than about 3% should not be used for CSB production (Rigassi, 1995). At the moment, there are no available tests specified in the literature to determine the presence of sulfates in soils to be used for CSB production. The criteria is also not well documented in most CSB literature.

Comparison of existing criteria for suitability is now possible because over the past few decades, several authors have published various recommendations. These have been successfully used in the past for the selection or rejection of soil for stabilisation. It is however noted that some of the suggested criteria, while based on similar properties, still differ from each other. Such dissimilarities only confirm the premise that the variability of a soil makes selecting a suitable soil a difficult exercise. Nevertheless, although the effect of climatic type and infinite variability of soils may influence some of the existing guidelines, most of the criteria still appear relevant to date. It is however, still rare to find authors recommending 'comprehensive' criteria based on all the four main soil properties, namely: particle size distribution, plasticity, compressibility, and chemical mineralogy. The summary of existing criteria for soil selection is shown in Appendix C.

The *adequacy of the criteria* for soil selection is still debateable, but the summary in Appendix C shows convergence on two main soil properties, namely: the particle size distribution and the plasticity of a soil. The guidelines are useful as basic criteria, but should not serve as rigid specifications for soil selection. This is because even soils that may fall out of the recommended ranges can, with modification, still be used to produce good blocks.

The size of the maximum size fraction (6 mm) and its distribution, and the clay content (and type) emerge as the major factors to consider in soil suitability selection. The chemical composition of a soil and its potential influence on the durability of the cement paste that bonds blocks, is likely to gain prominence as a new factor in the near future. Criteria limiting the presence of soluble salts and sulfates in soil samples are likely to become critical in light of recent scientific findings regarding their long-term harmful effects (Lea, 1976; Lunt, 1980; Neville, 1995). It is further noted that certain special soil types such as lateritic soils may not conform to these guidelines (Hammond, 1972; Ringsholt & Hansen, 1978; Stulz & Mukerji, 1988). Chemical tests for their composition is recommended in addition to the summarised criteria in Appendix C. The validity of the various criteria are not investigated further in this thesis.

3.2.3 QUALITY OF WATER FOR MIXING AND CURING

Water is required in CSB production at two critical stages: during the mixing of soil with cement and during the wet curing of green blocks. Existing CSB literature appear to place more emphasis on the quantity of water required, rather than on its quality (Webb & Lockwood, 1987). Water is basically needed for the hydration reaction leading to the gradual hardening of Portland cement. In the opinion of the author, both the quality and quantity of water ought to be given equal consideration. In most developing countries where CSBs are to be used, water is still a scarce resource, forcing many producers to use raw water from a variety of sources (Smith & Webb, 1987). The likelihood for the use of water with high levels of impurities cannot therefore be ruled out. Investigations during the fieldwork revealed widespread incidences of use of water of unknown quality (Chapter 4). In this section, the sources and suitability criteria for water are briefly discussed.

Sources

The main sources of raw water in developing countries include rainwater, rivers, lakes, swamps, groundwater, seawater, and rarely, tap water. Naturally occurring water may contain different substances such as dissolved solids, dissolved gases, suspended solids, bacteria, fungi, algae, protozoa (BRE, 1980). Apart from the few tap water sources where treatment works process the water (through screening, coagulation, aeration, flocculation, clarification and disinfection in that order, thus making the water potable), all other water sources in developing countries are unlikely to be treated. The quality of water used therefore remains unknown. High levels of impurities found in untreated water sources can be detrimental to the performance of a block (Parry, 1979). There use is likely to result in low strength, dimensionally unstable, and less durable blocks being produced (ILO, 1987). The effects of water of unknown service record on block properties are investigated experimentally in Chapter 4. Problems associated with use of water of unknown quality are also widely reported in concrete literature (Neville, 1995).

Suitability criteria

The suitability of water used for CSB production should be ascertained if good quality blocks are to be produced. Current specifications generally require that the water needed for the proper hydration of cement should be fit for drinking purposes (ILO, 1987; Rigassi, 1995). The criteria for potable water may not yet be absolute, but the following guidelines for quality are considered useful (BS 3148, 1980; ASTM C 94-

92a, 1992);

- Water with a high concentration of sodium or potassium should be considered unsuitable for use in cement hydration (due to dangers of AAR reactions)
- Water with pH of between 6.0-8.0, which does not taste saline or brackish, may be suitable for use in cement hydration
- Water containing humic acid or other organic acids should not be used (affects hardening of cement paste in the blocks)
- Use of sea water is not recommended (presence of chlorides >1000 ppm)
- Water with silt as suspended solids may be used even if concentration of 2000 ppm is found as long as the water is first left to stand in a settling basin or tank for at least 24 hours.

Since water sources are scarce in most developing countries, ways to comply with the above guidelines without rejecting the water should be sought. At the moment no known on-site test methods exist to determine the suitability of water for CSB production. Preliminary treatment methods for raw water of unknown sources could include screening, temporary storage, removal of algae, boiling and cooling.

3.3 CEMENT-SOIL STABILISATION PRINCIPLES

3.3.1 BACKGROUND

The stabilisation of soil to improve its properties for building purposes is an ancient practice. The procedures were passed on from generation to generation without necessarily understanding the main mechanisms involved. It was only from the 1920s that systematic scientific approaches were to emerge (Rigassi, 1995). Attempts were then made to replace the longstanding ad-hoc techniques previously adopted for soil stabilisation. Unfortunately, despite all the recent scientific advances made, soil stabilisation still remains an inexact science (Dunlap, 1975).

By soil stabilisation is meant the modification of soil properties by varying the soil-water-air interface (Fitzmaurice, 1958; UN, 1964; Ingles & Metcalfe, 1972). This is done to achieve more lasting characteristics than hitherto possible when the soil was still in its natural state. Some of the methods used to modify soil can result in irreversible changes, while others may result in reversible changes. The latter are likely to occur due to the lack of resistance offered by soil to environmental agents, especially water (PCA, 1970; Aksa, 1984). Evidence of poor resistance can be seen in most of the Third World where houses built of soil require to be regularly maintained during and after rainy seasons (Agarwal, 1981; Fullerton, 1979; BRE, 1980). Perennial problems of this type can be effectively overcome by stabilising the soil. Addition of a suitable stabiliser, especially a binder, can enable the soil retain its shape and dimensions. The soil will also gain in compressive strength and durability (Fitzmaurice, 1958).

As several input variables are involved, soil stabilisation is likely to remain a complex process. For effective stabilisation to be achieved the soil should be modified to give it the properties it lacks. There are several options for stabilising a soil, but the courses of action likely to be more effective should consider targeting its interstitial voids and improvement of bonding between its particles. Thus, it is generally accepted that:

- by reducing the volume of interstitial voids in a soil through mechanical compaction, direct action is taken to significantly reduce its porosity (Rigassi, 1995). Reduction in porosity is an effective way of increasing density and shear strength in a soil. By filling the voids in the soil which cannot be eliminated completely through compaction, direct action is also taken to reduce its permeability (Houben & Guillaud, 1994). Reduction in permeability has the positive effect of restricting circulation and retention of water within the soil fabric.

- by improving the cohesion and bonding in a soil, action is taken to cement and link the soil particles together. In this way dimensional stability, increase in compressive strength and improved durability can all be expected to be achieved. The method also ensures that changes in volume that would normally occur due to shrinkage and swelling are significantly reduced. Improved bonding also minimises the vulnerability of the soil to surface abrasion and erosions caused by rainwater and wind (DoHUD, 1979; Evans, 1980).

These two approaches are investigated experimentally in this thesis (Chapter 5).

3.3.2 CEMENT SOIL STABILISATION METHODS

Current soil stabilisation methods can be broadly categorised as follows:

- Mechanical stabilisation (by using a compressor)
- Physical stabilisation (by improving the soil grading)
- Chemical stabilisation (by using a binder to improve bonding between the soil particles)

Normally a combination of all three methods are used (Ingles & Metcalf, 1972). Each method is now discussed in turn to examine the degree of effectiveness in the stabilisation of soil.

Mechanical stabilisation involves compressing the soil particles together to increase density and reduce porosity. Compression leads to the redistribution and rearrangement of soil particles. It is the compaction energy used which forces the particles together and in the process most of the air is eliminated from the soil voids. Compaction is best achieved when the grain size distribution of a soil is continuous, not uniform or gap graded. The presence of grains of different sizes facilitates the occupation of voids left by other soil particles. Unfortunately, the effect of mechanical stabilisation when used alone is easily reversed, especially when the soil comes into contact with water (Jagadish et al, 1981). Water causes the lubrication the soil grains, forcing them to move about within the otherwise densified but still unbound fabric. It therefore follows that in addition to densification, the use of a binder will normally be required mainly to overcome the reversible effect of contact with water (Norton, 1986).

Physical stabilisation involves modification of soil properties by introducing the missing size fractions (Rigassi, 1995). The texture of a soil can be altered by calculated and controlled mixing of the different fractions together. When this is done, most of the voids that existed prior to physical stabilisation are closed due to closer packing of the grains. An anisotropic network is created limiting the movement of the grains in a soil (Ingles and Metcalf, 1972). Unfortunately, as was the case with mechanical stabilisation, the effect of physical stabilisation alone is not permanent (Rigassi, 1995). On saturation with water, soil grains are easily dispersed, or washed away. For better results, physical stabilisation of soil should therefore be combined with the other two methods (PCA, 1971).

Chemical stabilisation involves the addition of a binder or bonding agent to a soil. The binder modifies the soil properties through cementation or linkage of its particles (Houben & Guillaud, 1994). Both cementation and linkage are a result of chemical reactions involving the binder and water. Cementation creates a strong and inert matrix that can appreciably limit movement in a soil. The voids in the soil are also filled with insoluble by-products of the hydration reaction while some soil particles are coated and firmly held together by the binder (Ingles, 1962). The key binder that acts in this manner is Ordinary Portland cement. The full mechanism of the reaction as presently understood is discussed in the next section. It is generally reported in CSB literature that the effect of chemical stabilisation is more permanent, and may take several years or even decades to partially reverse. For this reason, chemical stabilisation of soil is so far considered to be the superior method of choice. It is also well established that the effect of chemical stabilisation is significantly increased by improving the soil grading and compacting the mix (Dunlap, 1975; Gooding, 1994). Combination of the three methods is therefore strongly recommended, and is used in the production of all experimental samples used in the research. The use of cement admixtures and lime in addition to OPC are also investigated experimentally (Chapters 5, 6 and 7). Other known chemical stabilisers include: pozzolanas, gypsum, bitumen, resins, whey, molasses, etc., (IIHT, 1972; Stulz & Mukerji, 1988; Houben & Guillaud, 1994). Use of these other binders are not discussed further in the thesis.

3.3.3 MECHANISM OF CEMENT-SOIL STABILISATION

The stabilisation reactions that follow from the addition of water to a soil-cement mix leads to the formation of a number of by-products (Ingles, 1962; PCI, 1970; BS 1924 Part 1, 1990). Since soil as the bulk constituent contains different fractions of gravel, sand, silt and clay, each of these fractions will respond to the reaction with cement in different ways. Moreover, as cement itself contains different minerals, each of them will also react differently. Not only will they interact amongst themselves, but they are also likely to affect the manner in which the others react (Weidemann et al, 1990). The main reactions involved and the nature of the resulting microstructure are described in the sub-sections which follow.

The Main Chemical Reactions

According to CSB literature sources, two main chemical reactions can be distinguished; a primary reaction involving the hydration of cement with water, and a secondary reaction involving the clay minerals and the liberated lime from the primary reaction (Houben & Guillaud, 1994). The hydration reaction between cement and water results in the formation of hydrated cement paste and conventional mortar (embedding gravel and sand fractions). The secondary reaction also results in the formation of a binder like by-product (Spence & Cook, 1983).

The mechanism of the reaction is thought to be as follows:

1. Primary reaction involving OPC constituents:

$$\left.\begin{array}{l} C_3S \\ C_2S \\ C_4AF \end{array}\right\} + H_2O \longrightarrow C\text{-}S\text{-}H \quad + \quad C_4A\bar{S}H_{12} \quad + \quad Ca(OH)_2$$

$$(\text{Cement}) \quad + \quad (\text{Water}) \quad \left(\begin{array}{l}\text{Calcium}\\ \text{silicate}\\ \text{hydrate}\end{array}\right) \quad + \quad (\text{monosulphoaluminate}) \quad + \left(\begin{array}{l}\text{Calcium}\\ \text{hydroxide}\end{array}\right)$$

The main products of the above reaction are: calcium silicate hydrates, monosulphoaluminates and calcium hydroxide. It is the first two, namely the C-S-H and $C_4A\bar{S}H_{12}$ that are responsible for strength development in a block (Ingles & Metcalf, 1972). It is the gravel and sandy fractions in the soil that are affected by this reaction. Both the C-S-H and $C_4A\bar{S}H_{12}$ are known to have high binding capacity. The binding forces they generate are responsible for intertwining and embedding the gravel and sand fractions creating a strong network within the soil fabric. This inert and anisotropic network introduces rigidity not previously present in the soil. Due to the network, movement of the coarse soil fraction is resisted and subsequently becomes highly limited. The net effect results in the development a particulate composite structure. As can be expected, the properties of the composite are influenced by the amount of cement used relative to the soil fraction, and by the nature of the bye-products resulting from the reaction. The reaction is known to liberate free lime which then sets off the secondary reaction with the clay component in the soil.

2. Secondary reaction involving freed lime and clay

A. S	+	$Ca(OH)_2$	$\longrightarrow$	C-S-H	+	C-A-H
(Clay)		(Calcium hydroxide)		(Calcium silicate hydrate)		(Calcium alumate hydrate)

The two main products of this reaction (the C-S-H and the C-A-H) both have binding capacity not very different from the ones of the primary reaction. This reaction is mainly pozzolanic with the gelatinous amorphous hydrates equally contributing to hardening of the block. Following the reaction, a stable chemical bond develops between the clay crystals, through a mechanism known as linkage. The reaction proceeds slowly but is dependent on the quantity and quality of clay, and on the amount of free lime available (Spence and Cook, 1983; Houben and Guillaud, 1994). The amount of calcium hydroxide released is limited by the lime saturation factor (LSF) of the OPC. The LSF is fixed at the time of manufacture of the OPC, often ranging between 0.66 and 1.02 (Spence and Cook, 1983). Restriction of the upper limit is mainly done to control the amount of free lime in the cement paste which is otherwise associated with unsoundness and undesirable expansion.

The Resulting Particulate Composite Matrix

As a result of the preceding reactions, the particulate composite fabric that constitutes the block is thought to comprise the following matrices (Herzog & Mitchell, 1963; Ingles & Metcalf, 1972; Dunlap, 1975; Lea, 1976; Houben & Guillaud, 1994):

- cement hydrates (calcium silicates, calcium aluminates, sulphoaluminates, ferrites)
- conventional gravel-sand-cement mortar
- calcium hydroxide ($Ca(OH)_2$)
- unhydrated cement residues (UCR)
- stabilised clay
- unstablised soil (gravel, sand, silt, clay)
- capillary pores

The proportions of each of these matrices in the block and the strength of bonding between the cement hydrates and coarse soil fraction are thought to influence the compressive strength, dimensional stability and durability of a block (Mitchell & El Jack, 1978; Lunt, 1980; Rigassi, 1995). The various by-products are also summarised in Appendix B.

For better performance of a block, it is desirable that the cement hydrates coat a high proportion of the coarse soil fraction, as well as filling the spaces between the particles. For this to be achieved, an optimum proportion is needed between the sand and cement. It can generally be expected that the lower the cement content, the higher the resulting voids content in a block. A high voids content (porosity) is often associated with a weak block (Houben & Guillaud, 1994). This phenomenon is investigated experimentally in this thesis (Chapters 6 and 7). It is potentially possible that with the low amounts of cement used, the voids content of a block fabric is likely to remain high. Moreover, the low quantity of binder used can also result in the presence of a greater proportion of unstabilised soil in a block fabric. Such an outcome would be highly undesirable due to vulnerability to deleterious effects of water, temperature and relative humidity (Chapter 2).

The presence of the matrix of calcium hydroxide in a block is a potential source of instability. Calcium hydroxide is soluble in water and is therefore likely to be leached out during the service lifetime of a block (Chapter 2). Moreover, calcium hydroxide is also known to react readily with the CO_2 from air to form expansive products. Though the reaction is very slow, the expanded products formed can easily contribute to the disintegration of a block over time (Chapter 2). A means of eliminating the presence of $Ca(OH)_2$ in a block by providing a substance with which it can react (microsilica) to form a secondary binder is investigated experimentally in this thesis (Chapters 5, 6 and 7).

An attempt is also made to examine the microstructure of block samples in the course of this research. This is done using petrographic examination of thin sections (Chapter 7).

3.4 STABILISED BLOCK PRODUCTION PROCESS

3.4.1 BLOCK PRODUCTION CYCLE

The production process of CSBs is broadly similar to that of concrete blocks. Similarities exist between the products, manufacturing process and in the organisational control methods. The processing method represents a major input variable. It can significantly influence the quality and long term behaviour of a block (Rigassi, 1995). Yet most of the CSB literature sourced appear to take this variable for granted. For these reasons, the separate treatment of the processing method as is done in this section was thought to be necessary.

CSBs are produced by compressing a damp mix of soil and cement in a press mould. After demoulding, the green blocks are not used immediately, but are first allowed to cure. This is because the strength of a block, just as it the case with concrete blocks, increases with age (Apers, 1983; Ruskulis, 1997). The duration of curing is dictated by the specification for the type of stabiliser used; 28 days when OPC is used and 56 days when hydrated lime is used (BS 12, 1971; BS 890, 1972; Lea, 1976). The production of CSBs can be organised as a small scale cottage concern or as a much larger mechanised industrial unit. Whatever the approach adopted, the production cycle is likely to remain similar, and can be categorised into four basic stages, namely:

- Soil extraction (and preparation)
- Mixing (soil, cement and water)
- Moulding (of the block)
- Curing (of the green blocks)

The order of the production process stages is not commutative and should therefore follow one after the other. This is illustrated by the schematic shown in figure 1.

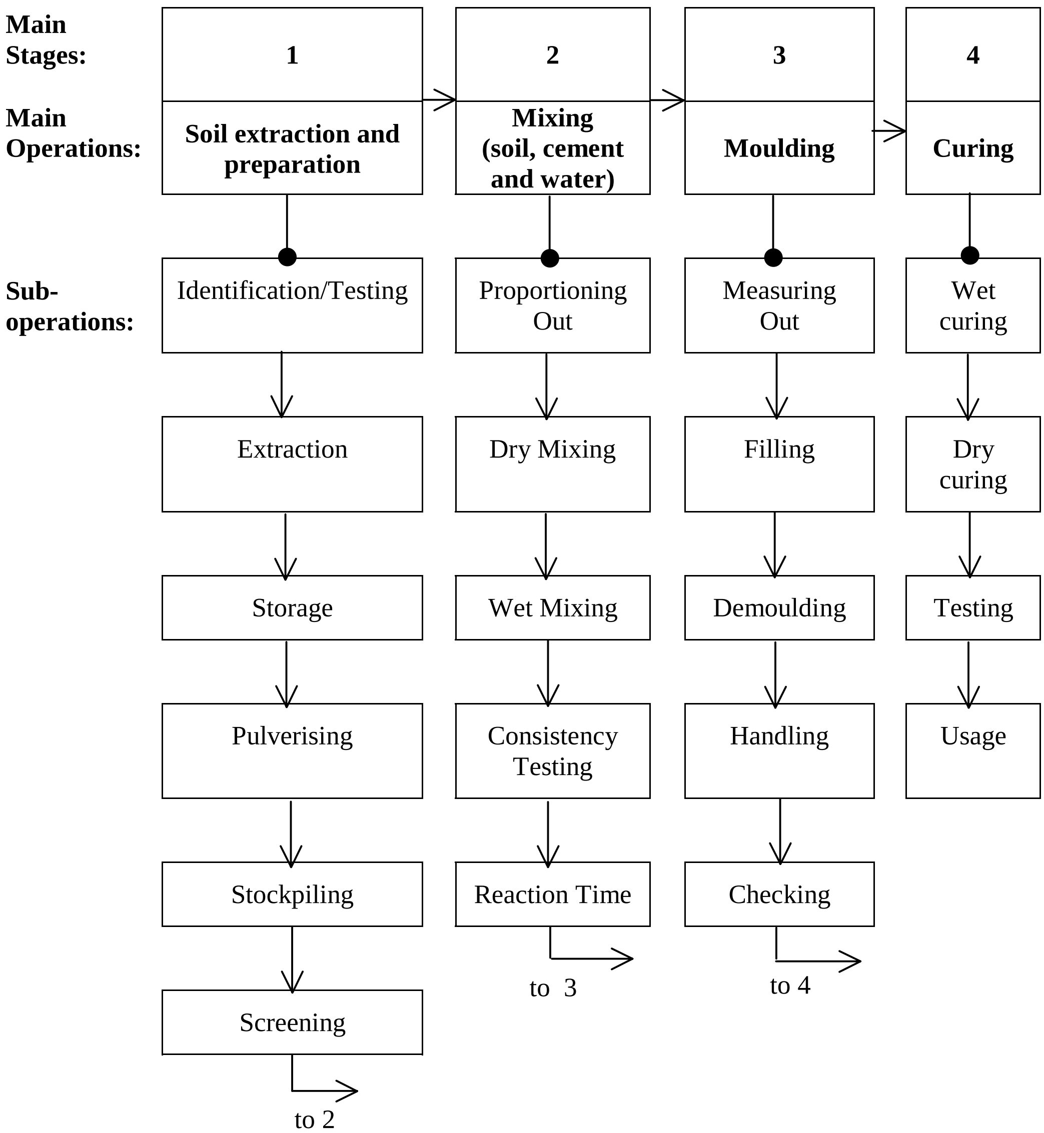


Figure 1: Schematic showing the main stages and operations of a CSB production cycle.

(Adapted from: Houben & Guillaud, 1994; Rigassi, 1995; Houben et al, 1996)

The schematic in figure 1 shows that within the four main production stages, there are several sub-operations. These operations are so interdependent that the sequencing adopted has to follow the order shown since each preceding operation must be completed before the next one can start (Webb & Lockwood, 1987). For efficient productivity, a block production site should be organised taking into account the unique nature of the various operations with a view to harmonising them. This has often been underrated with the result that key operations are missed or interfered with. In such cases both the quality of the block and the productivity of the site can be adversely affected.

As the properties and performance of a block are heavily dependent on the outcome of each of the production cycle processes, each stage is now discussed in turn. The experience gained by the author during the production of block samples for experimental investigations in the course of this thesis, and prior to this, during the manufacture of blocks for large CSB building projects in Uganda between the period 1987 and 1994, are also drawn upon. The discussions are presented in Sections 3.4.2 to 3.4.5 that follow.

3.4.2 SOIL EXTRACTION AND PREPARATION

Soil to be used for CSB production should preferably be extracted from or near the proposed building site (Williams, 1980). Indeed this is one of the major attractions of using CSBs for building purposes (DoHUD, 1955; Fitzmaurice, 1958; Denyer, 1978; Agarwal, 1981). Sourcing the main raw material in this manner can significantly minimise expenses normally associated with transportation (Cinva-Ram, 1957; ILO, 1987). Prior to extraction however, the soil at the potential site has to be properly identified and classified (Norton, 1997). It is only after this has been done, with the results being acceptable, that subsequent procedures including extraction, may follow.

Identification of soil at the proposed extraction site may be facilitated if information on local soil types, or information based on experience can be obtained. Documented sources may include maps, previous construction records, etc. Even then, the soil will still need to be identified using field indicator tests followed by laboratory tests in that order (Akroyd, 1962; Stulz & Mukerji, 1988; BS 1377, 1990). Details regarding both types of soil tests are discussed in Chapter 4 and Chapter 5. As stated earlier, the soil needed for CSB production should contain some coarse fraction (fine gravel and sand), and some fines fraction (silt and clay). The clays in the fines fraction significantly contributes to binding the fine gravel, sand and silt together (Lunt, 1980). If the soil is deemed satisfactory either for direct use as it is, or following blending with the missing soil fractions, then the extraction process can commence.

The soil is best extracted from the sub-soil level and not from the top-soil level (from about 300 mm downwards from the surface) (Webb & Lockwood, 1987). Disregarding of top-soil layers ensures that undesirable inclusion of organic matter, normally dominant at this level, is avoided. Another important consideration before soil extraction can commence is whether the ground to be used as a pit can indeed supply enough soil for the particular size of the project. The pit should be located in an area large enough to generate and sustain the supply of sufficient raw material to satisfy the building requirements. In short, the process of soil extraction should only begin after it has been established that the soil is suitable, or can be modified to become suitable and that it is available in sufficient quantity (ESCAP/RILEM/CIP, 1987; ILO, 1987; Gwosdz & Sekivale, 1998).

The soil may then be extracted either manually or by mechanical means. Manual extraction involving labourers using basic hand tools such as shovels and picks has the disadvantage of low output. Experience of the author also showed that daily output in such cases rarely exceeded 3 m^3 per day per man. Yet when motorised mechanical means are used (excavators, bucket loaders), the output significantly can increase to about 100 m^3 per machine per hour. The high cost of hire or purchase of such equipment, coupled with the need to create jobs, might however continue to dictate that manual labour be used for the foreseeable future. Which ever approach is adopted, after the extraction of soil, further preparatory operations are still required. This is because freshly extracted soil is not immediately suitable in the natural state in which it is found (due to the presence of solid concretions in lumps or large pieces, or friable aggregations in powdery form). Further, the soil when freshly dug out, depending on its natural moisture content, may still be wet, moist, damp or even dry. It is not advisable therefore, to immediately use soil in any of these forms for block production without further preparations. The main objective of preparing the soil is to transform it into a more useable form of known moisture content and of the correct size functions (Cinva-Ram, 1957).

Soil preparation after extraction involves drying out, temporary storage, pulverisation, stockpiling and screening. Storing and stockpiling simply follow the key operations of drying and pulverisation. The just extracted soil can be dried out by spreading it out in thin layers on a hard level surface. By drying out the soil, an attempt is made to gradually free the soil particles from their entanglement and to obtain a material of almost even minimum moisture content. To achieve this fast enough, the thinly spread out soil should be regularly raked through in order to turn it over repeatedly. The same drying out operations were used for the ordinary builders sand used for manufacturing block samples for the experimental investigations in the course of this research (Chapter 5). The drying operation can be verified as satisfactorily completed through visual examination of any changes of colour of both the top and bottom layers. If the bottom soil is still of dark complexion even after several turning-over operations, it simply means the drying process has yet to be continued. When both the bottom and top layers of the spread out soil show a uniformly light colouration than when first obtained, then the soil can be considered to be dry enough. At this stage it should also be easy to break up any remaining soil lumps by hand. These should mostly be clay lumps, since it is such fines that are responsible for forming nodules. Nodules of diameter greater than 10 mm should not be allowed (Arrigone, 1986; Houben & Guillaud, 1994). Simple hand tools such as wooden hammers, hoes, pincers, etc., can be used to pulverise and break up soil lumps. Pulverisation and breaking up of lumps in this manner also helps speed up the drying process (by increasing the surface area). The pulverised soil can then be screened.

Screening of soil is a crucial process. Screening may be done by sieving using wire mesh sieve screens of agreed maximum aperture sizes. Recommended aperture sizes for sieves to allow the maximum size fraction of soil to be included in a block vary. Two recommended size ranges are reported in the literature: 5 to 6 mm (Webb & Lockwood, 1987; ILO, 1987), and 10-15 mm (Houben & Guillaud, 1994; Rigassi, 1995). The maximum soil size fraction used for block making in the course of this research was 5 mm (fine gravel and below). The process of sieving also serves to eliminate any undesirable materials still in the soil after the general preparations have been done. But the objective remains to rapidly ensure that only soil below the required maximum size fractions are the only ones obtained. The screen used may be a fixed one set at an oblique angle, or it may be a suspended one on top of a collecting bin. In the former case the soil is thrown at the screen while in the later case it is poured through it. When a fixed screen was used in Uganda, the rate of screening was about 1 m^3 per hour per person (Kerali, 1996). If no screen is available on site, it is still recommended that some form of removal, even by hand, be conducted. Large objects are usually easy to detect and to remove by hand.

After screening, the soil can then be stockpiled awaiting use. From this stage onwards, the soil should be covered in order to keep it dry and prevent clay lumps reforming once again. In the event that the soil obtained is predominantly of one type or size, then controlled mixing with other soil types and size fractions imported from nearby quarries should be done. This will improve the grading of the soil. For example, a predominantly sandy soil could be mixed with some clay (recommended minimum 10% and maximum 30%). Controlled mixing such as this will also help avoid unnecessary rejection and wastage of soil. During this research for example, the ordinary builders sand which was supplied, was mixed with pure Kaolin clay (15%) to form the soil of the desired design properties (Chapter 5).

In the preparation of soil samples for this thesis, it was not necessary to go through all the steps mentioned here. It was only necessary to dry out the soil on the laboratory floor, screen it through the 5 mm circular sieve screen, then store it in a well covered bin. The bins were kept in a dry area of the laboratory. Further discussions on the procedures are presented in Chapter 5.

3.4.3 MIXING OF SOIL, CEMENT AND WATER

This stage of the production process initially involves the dry mixing together of the main constituent materials (soil and cement), before wet mixing with water to hydrate the OPC. The sufficient distribution of OPC throughout the soil, and the homogeneity and uniformity of the resulting block, can be significantly affected by the procedures adopted at the mixing stage. Considering that over 90% of a block comprises soil, and that only less than 10% comprises cement, achieving an even distribution of the latter in the former is far from being straightforward. Yet the procedure is often underestimated, with severe implications for the quality of the resulting block. The main operations during the mixing stage include: proportioning out, dry mixing, wet mixing, consistency testing and hold-back time (retention time).

Proportioning out of soil and cement in their dry state is the first crucial step which requires care. The total volume of the separate dry ingredients to be mixed should be based on a practical criteria (Mukerji & Worner, 1991). From experience, the proportioning criteria is normally based on the hourly output of the press being used. This means blocks have to be produced in separate batches, requiring mixes to be prepared only in sufficient quantity to be consumed by the press within approximately one hours operation. Large batches are undesirable for several reasons. If larger batches are mixed without immediate compaction following, the water may evaporate causing the cement to set prematurely. This can easily be expected especially in hot tropical climates (Fullerton, 1979; BRE, 1980; Spence & Cook, 1983; Norton, 1986; Stulz & Mukerji, 1988). Moreover, with large batches, it is also very difficult to achieve an even and homogeneous mix. Use of large batches also increases the risk of moisture variations developing in a mix. OPC is known to set within about 45 minutes (Lea, 1972; Illston, 1994). If within this time the wet mix has not yet been compacted, then a significantly weakened block might be produced (Lunt, 1980). Moreover, as cement is usually scarce and therefore expensive in developing countries, wastage of the binder through premature setting should be foreseeable and avoidable. This can be done as stated earlier by proportioning materials based on batch sizes that can be compacted within the hour (Mukerji, 1994).

Proportioning out of dry soil and cement can be done either by weight, or by volume measurements (Webb & Lockwood, 1987). It is important that the materials being proportioned out remain in the same dry physical state. With volume measurement proportioning, either a single gauge box, or two different gauge boxes may be used. The use of a single gauge box which is meant to measure both soil and cement is common. The amount of cement and soil required for an hourly batch of blocks is then measured by filling, levelling and emptying the gauge box up to the required number of times for each separate material. This method is not without its failings. Apart from contamination, problems may arise if the moisture content of the soil will vary. When this happens, so will its specific surface area. Variation in specific surface area of a soil will result in different amounts of soil being measured out each time. This remains a major area of concern. Attempts should be made to ensure that the moisture content of the 'dry' soil remains constant. This is achievable through covering of the dry samples and avoiding humid environments. The mode in which the gauge boxes are filled is also a potential source of error. All loose material should ideally be filled in and tampered to avoid under-filling with the top also be levelled off to avoid over-filling. During the proportioning out of dry materials in the course of this research, proportioning was done by weight, not by volume (Chapter 5).

As stated earlier, following proportioning, the soil and cement should be mixed in two separate stages; first in a dry physical state, then in a wet state, in that order. Dry state mixing is best done by spreading the cement evenly over the spread out dry soil. The two are then mixed together thoroughly till a uniform homogeneous colour is observed. Samples of the mix can be scooped up and visually examined to certify that uniform colouration has approximately been achieved. Uniform colour of the mix is therefore the only useful control tool or indicator at this stage. Since no other obvious test exists, mixing of the two dry materials should therefore continue until a uniform colour is obtained. Mixing can be done manually or by mechanical means. For the experimental investigations during this research, dry mixing was done using mechanical means (Chapter 5). Mechanical mixing is preferable over manual mixing for several reasons. With hand mixing, the same high level of concentration that should be maintained throughout is not humanly possible all the time. It is not uncommon to find mixes produced earlier in the day being more uniform than those produced later in the clay by the same person(s). Exhaustion, familiarity, lack of concentration or interest, lack of knowledge of the implications, coupled with inadequate constant supervision, may all contribute to insufficient mixing.

After dry mixing to uniform colour, water can then be added to the still dry soil and cement mix. The purpose of the water is to hydrate the cement and to enable the mix to be compacted at optimum moisture content. Determination of the right amount of water to achieve both aims remains an area requiring more investigation in future. If mixing is to be done manually, then the determined amount of water should first be lightly sprinkled using a shower rose head container. This should be done in such a way that it just moistens the surface of the well spread dry mix. The water should not be poured onto the mix all at once, as was observed on some sites (Chapter 4). Neither should it be poured onto a heap of the mix as is also commonly and wrongfully done. This is to avoid creating particles of damp soil that may roll down the side of the heap while growing even larger in size. Contact with water should be made uniform and widespread. The mix is then turned over before more water is added to the soil and cement mix. The wet mixing operation has to be done repeatedly until two things happen: the damp mix achieves a uniform colour and it also passes the 'drop test'. Even on achieving uniform colour, any lumps still in the mix should be broken down. Lumps can form if the mixing time is too long, the moisture content is too high, or when incorrect mixing procedures were followed. Both dry and wet mixing should ideally be done within 3 to 4 minutes (Houben & Guillaud, 1994; Rigassi, 1995). For clayey soils, the moisture content of the mix should preferable be slightly higher than the OMC: for sandy soils, it should preferably be slightly lower than the OMC (Houben & Guillaud, 1994). In the absence of any other consistency test similar to the slump test used for concrete products, the drop test remains the only satisfactory indicator for approximating the OMC of the damp soil and cement mix (ILO, 1987). The slump test as used in concrete production may not work as a consistency test for CSB production due to the near-dry mix state required for the latter.

Unfortunately, passing the drop-test may not necessarily mean that further control measures be set aside. During the production of blocks in the course of this research, it was noted that in some of the mixes which had passed the drop test, excess water was observed dripping on the sides of the mould during compression. Similar experiences were recorded in Webb & Lockwood (1987). The amount of water in the mix had to be reduced as mentioned earlier. In the experiments conducted during the thesis, the blocks made from mixes where the water dripped by the side of the mould were not rejected. They were kept aside and labelled accordingly. The surprising outcome when these blocks were tested for wet compressive strength twenty-eight days later, was that values obtained were 25 to 30% higher than for the blocks made from the reduced water content mixes. The difference between the mean dry and wet compressive strength were also reduced. The only feasible explanation of this unexpected outcome was that the excess water in the mix contributed to the maximum hydration of cement. The issue of OMC and the sufficiency of water for hydration of cement is recommended for further future research (Chapter 8).

The time between wet mixing and compaction in the mould should be as short as possible when OPC is used as the stabiliser. A period of between 5 and 10 minutes has been suggested (Houben & Guillaud, 1994). For this reason, water should only be added for wet mixing precisely before the start of the moulding process. If any delay between dry mixing and moulding is anticipated, then wet mixing should be deferred.

The dry mix should be covered with a polythene sheet or similar protective cover. Delays of more than two hours due to lunch breaks or other human needs should not be allowed. The effect of hold-back time on the performance of blocks is investigated experimentally (Chapter 6).

3.4.4 Compressing the Damp Soil and Cement Mix

Compressing the damp soil and cement mix is a key stage in the production of CSBs. The effect of cement stabilisation of soil is significantly enhanced by compressing it (Ingles & Metcalf, 1972; Gooding & Thomas, 1995). Compression reduces voids by driving off air (compaction) and any excess water (consolidation) from the damp soil and cement mix. The combined expulsion of air and water, and the squeezing of the solid particles together increases the density of the mix. Uncompressed soil-cement mix of the same mass prior to moulding may have density ranging between 1000 kg/m^3 and 1400 kg/m^3 (Norton, 1997). After compression, the increase in density for the same mass of mix is between 30% and 120%, commonly ranging between 1700 kg/m^3 and 2200 kg/m^3 (Houben & Guillaud, 1994; Rigassi, 1995). Higher densities are associated with improved durability (Spence, 1975). Compression of the mix should be done as soon as it has passed the drop test. The compression stage involves five distinct steps, namely: measuring out, mould filling, moulding, demoulding, and handling of the green blocks (Rigassi, 1995). Underestimation of any of these five steps can lead to the production of inferior blocks of low compressive strength and durability.

The damp soil-cement mix has to be correctly measured out before filling the mould. Any slight variations in the amount of mix fed into the mould can result in blocks of differing density and sizes. Cases of differences in density are normally associated with fixed-volume type of compression machines while size variations with fixed-pressure ones (Lunt, 1980). The two types of errors are cumulative and should be minimised or avoided completely. To do this, the amount of damp soil-cement mix to be filled into the mould should be strictly controlled. This can be achieved by measuring out the exact amount of mix to be placed into the mould each time using either a graduated bucket, scoop, or measuring box of fixed volume. Measurement by weight, though considerably slower, can also be done. Presses equipped with adjustable measuring devices which either use sliding valves or tipping boxes are extremely rare. Whichever method is adopted, the important point is to ensure that the correct amount of mix, and of even moisture content, is fed into the mould each time. The moisture content of the mix matters a great deal, and variations should preferably be avoided. Variations can lead to changes in specific volume causing differing amounts of mix to be placed each time. During the manufacture of block samples for experiments in the course of this research, measuring out was done exclusively by weight (Chapter 5).

Filling of the mould with the measured out mix should then follow promptly. Before the actual filling of the mould box, its interior should be cleaned. Although mould types vary, they are generally designed to be completely filled with the mix. After filling the mould, the top of the last layer should be scraped off level with the sides of the mould. The filling is best done in two or three equal layers. The filling operation for each layer should be checked each time, with the fingers being used to press the mix into the corners after each layer has been placed. Pressing the corners is necessary because the bottom corners of the mould are the most difficult part of the mould to fill. In addition to this topping up, removing excess material and any lumps of soil or stones, etc., should all be done for the top layer at this stage. After the mould has been filled and checked, it can then be covered with the mould lid. The lid itself has to be correctly positioned ensuring that no mix is left entrapped between the lid and the edges of the mould. There is yet no known device to indicate that the mould has been correctly filled (Mukerji, 1988; Rigassi, 1995).

The mix can then be compressed. As machines vary widely, the operations needed to compress the block will differ according to the specified characteristics and operating manual for each press. Block making presses will vary by:

- Compression type (static compression, dynamic, or kneading)
- Moulding pressure (low < 3 MPa; moderate 4-6 MPa; high > 7 MPa)
- Compression ratio (1:1.65 to 1:2)
- Productivity (maximum daily or hourly output)
- Mould size and type (fixed, adjustable: solid, hollow, frogged, interlocking)

In most manual presses, the force applied to the compression lever will depend on how much mix has been placed in the mould. This force should neither be too high, nor too low. If the force required is too high, either the machine will gradually get damaged, or the operators will tire out quickly. If the force is too low, the block will be insufficiently compressed (Norton, 1997). Such difficulties are experienced even with motorised press units. As the compression force remains uniform, it becomes impossible to check during compression if the mould has been correctly filled. If the correct amount of mix is at its OMC is placed each time and the same moulding pressure is applied, it can be expected that blocks of constant density are produced. Such blocks will also tend to be of uniform dimensions. For the Brepack machine used to produce samples in this research, an attached pressure gauge provided the needed indication of the pressure exerted (6 MPa and 10 MPa). The transmission of energy to the mix was through an hydraulic system. The main advantages of this compression machine were that the number of operations needed were few, thickness of the block could be controlled, and laminations usually associated with other compression methods avoidable. Laminations will tend to occur if the speed of compaction is below 1-2 seconds (Houben & Guillaud, 1994). A further advantage was that the sides of the blocks were well compacted. This should be expected as the internal friction of the soil-cement mix increases, so does the pressure on the surface of the mould. The mix closer to the mould surface is more thoroughly compressed than that further from it. Which is why the least compacted soil-cement mix is likely to be found at the middle of the block which is subjected to the least shear (Houben & Guillaud, 1994). This has clear implications for the durability of a block. It explains why block surfaces should be protected at all costs and not allowed to deteriorate prematurely. If allowed to recede inwards, it will expose the core of the block which is its least compacted part. The rate of deterioration is then likely to increase significantly from then on (Chapter 2).

After moulding, the green blocks have to be ejected usually through the same piston that compressed the block (in most presses). On ejection, the block should be carefully removed and handled. Since the green block is still weak and fragile, great care should be taken during the handling operation. The surface area of contact between the block and the mechanism of removal (hands, block pincers, wooden pieces) should be as large as possible to reduce any unreasonably high pressures on the green blocks to a minimum. Special precautions should be taken while removing blocks from certain types of moulds. Removal of solid blocks is easier than removal of hollow, frogged and interlocking blocks of a similar size. Non-solid blocks will tend to have several points of vulnerability, such as protrusions, indentations and thin sections. Areas of the block such as edges and corners still remain particularly vulnerable and should not be touched. These considerations are often underrated with the result that blocks with broken edges and chipped corners are commonly produced. On removal from the mould, green blocks should undergo certain quality control checks. Between 5 and 10 blocks per batch can be selected at random for such tests (Rigassi,1995).

Details of the tests conducted at this stage and findings are presented in Chapter 5. These tests were done to identify variations in blocks produced and their possible reasons. For example, any variations in the direction of compaction (height) could easily be attributed to irregular filling, which could then be corrected. Density variations can be detected by weighing the samples and taking their dimensions. Low density for the same mix could then be attributed to insufficient mould filling. These quality control tests are useful since they contribute to reduction in wastage and early detection of poor procedures which would otherwise lead to the production of inferior quality blocks, of low compressive strength and durability (Chapter 4).

For quality output to be sustained, regular maintenance of the press should be conducted. After each day, or even on brief stoppage of work, the press should be thoroughly cleaned and left in a state ready for the next production to begin. Maintenance procedures normally included in the manual for each press should be strictly adhered to. Supervisors can assign one person the responsibility to do the daily maintenance. Each inspection and maintenance conducted should be verified and a record of what was done and when should be kept.

3.4.5 CURING OF GREEN BLOCKS

Curing of green blocks may be the last stage in the production process but remains one of the most consequential. As stated earlier, the strength of CSBs, as with almost all concrete products in which OPC is used, increases with age (ILO, 1987; Lea, 1970). The hardening of OPC takes time, and so will the development of strength in a block. For the OPC to harden normally, it requires the continued presence of moisture in a block which enables it to complete the hydration process. Insufficient w/c ratio, and low degree of hydration can result in considerably weak blocks. If the green block is not allowed to retain sufficient moisture, then the hydration process will have been interfered with. This can result in unsatisfactory blocks of low strength and poor performance (Enteiche & Augusta, 1964; Odul, 1984).

The objective of the curing stage is therefore to ensure that moisture still in the block is allowed to facilitate the hydration process and to come out gradually and evenly. The two variables during curing that can affect this objective are the duration (time) and conditions (wet, dry, temperature, relative humidity, wind). The curing duration is often dictated by the specifications for the type of binder used and is based on the time needed to achieve the maximum degree of hydration. For OPC the recommended length of time is usually 28 days (BS 12, 1971; Spence, 1980; ILO, 1987; Taylor, 1998). Curing conditions specifically refer to the microenvironment in which the green block is placed. Normally the conditions are such that wet curing is followed by dry curing. During this research, the effect of varying curing conditions and time were investigated. The results are discussed in Chapters 6 and 7.

The curing process normally consists of two phases (ILO, 1987; Neville & Brookes, 1994):

- Primary curing phase (3-5 days)
- Secondary curing phase (up to 28 days for OPC, up to 56 days for lime)

The *primary curing phase* follows immediately after the demoulding of green blocks. During this phase, emphasis is placed on ensuring the retention of moisture within the block. The green blocks should be shielded from direct sunlight and strong winds. The process usually takes three to five days with seven days being the maximum possible (ILO, 1987). If the green block is not shielded, then rapid evaporation is likely to take place, promoting the undesirable loss and uneven distribution of moisture in the block. This can result in surface shrinkage cracking. For this reason, the surface of green blocks should be well protected using light coverings such as polythene sheeting, tarpaulins, or other suitable light materials. Polythene sheets are quite useful since they allow the temperature to rise, while at the same time ensuring that approximately 100% relative humidity is achieved (Rigassi, 1995). During the curing of samples for this research, green blocks were superficially covered on the first day, then placed inside sealed polythene sheeting 24 hours later. The blocks were then laid next to one another in a designated primary curing area within the laboratory.

Current indicators of sufficient primary curing are based on colour changes of the block, and sometimes on the degree of evaporated moisture trapped beneath the covering polythene sheeting. Freshly demoulded blocks, due to the relatively high moisture content still in them, tend to be of dark complexion. As the moisture is used up for hydration, with some escaping, the complexion of the demoulded block begins to adopt a much lighter colour. When sustained light colouration is attained then primary curing can be considered complete. Generally, the moisture content of the block should not be allowed to vary by more than 1-2% during the primary curing period (Houben & Guillaud, 1994; Rigassi, 1995).

The *secondary curing phase* follows on from the previous phase, with the objective this time being to allow any moisture still in the block to evaporate out gradually and evenly. The gradual evaporation of moisture out of a block affects both the OPC hydrates and the clay minerals in the block. Secondary curing also allows the semi-cured blocks to be conveniently stacked nearer to the actual building site proper. Even then, the blocks have not yet fully developed the required compressive strength. They should therefore be stacked not more than 10 blocks high on a hard, flat and level surface (Rigassi, 1995). The stacked blocks should continue to be protected from direct sunlight, wind and rain. This can be done by dry stacking the blocks under a covered shed or shelter for 2 to 3 weeks in the case of OPC stabilised blocks. After 28 days from the date of demoulding, the blocks are deemed to have achieved sufficient strength. After this period, there will be no further noticeable significant increase in strength for the OPC hydrates that bind the blocks particles together (Fitzmaurice, 1958; PCA, 1970). For lime stabilised blocks, twice the time recommended for OPC should be provided for during both primary and secondary curing (BS 890, 1972; Bessey, 1975; Coad, 1979; Apers, 1983; Webb & Lockwood, 1987).

The fully cured blocks can then be placed on wooden pallets, or stacked in easily counted lots. Initial performance tests can then be conducted on randomly selected samples from each batch (ILO, 1987). These are then compared with local acceptable minimum standards for buildings (Houben et al, 1996). Results of initial performance tests for blocks produced during this research are presented in Chapters 6 and 7.

3.5 CONCLUSION

From the preceding sections, a number of important conclusions can be made. These are summarised below, presented in the order of coverage in Chapter 3.

Main constituent materials (cement, soil, water)

CSBs will perform well for the service life of the structure of which they form part if sufficient attention is paid to the choice of materials and their proportioning. A thorough knowledge of the nature and properties of the three main materials (cement, soil and water) is thus required.

Cement constitutes between 5 and 8% by weight of a block. It was noted in Chapter 3 that the main function of OPC is to bind the soil particles in the block together, thus forming a composite structure with increased compressive strength (both wet and dry), limited dimensional movement and improved durability. For this to be attained, the mechanism of cement hydration and the properties of the resulting cement hydrates that can influence the durability of the block, should be well minded. It was established from literature sources that present approaches in the use of the binder appear to take these factors for granted. Yet for OPC to have maximum effect in binding the soil particles together, and for the block to develop high strength, the water-cement ratio used should be low and a maximum degree of hydration has to be achieved. The former can be facilitated by use of cement replacement materials, while the latter can be achieved by proper curing. These two considerations rarely feature in current CSB manuals. The effect of these two factors on the durability of blocks are investigated experimentally in Chapters 6 and 7.

Soil constitutes the bulk of CSBs. Amounts as high as between 90 and 95% by weight of the block are normally used. Soil is composed of fine gravel, sand, silt and clay. In natural soils, the proportions of these four main soil constituents can vary infinitely, and with each variation, so do the properties of the soil. Not all soils can therefore be considered suitable as found for CSB production. To assess the suitability of a particular soil, the soil will first need to be identified and classified. It was established from literature sources that the most important basis for soil classification are through its particle size distribution and its plasticity. Current suitability criteria for soil requires that they be well graded. The soil has to contain almost every soil size fraction between the maximum (usually gravel less than 20 mm, or 6 mm), and the minimum particle size (usually clay less than 0.002 mm). In a well graded soil, the packing of the soil particles is considered to be in its most dense state. The plasticity of soils on the other hand is associated with the presence of fines, usually clay. According to literature sources the desirable plasticity index of soil suitable for stabilisation should vary between 5% and 30%.

Clay types vary immensely, though three main groups are identifiable (kaolin, illite and montmorolinite). It was widely reported in the literature that the clay type and amount is a major factor in determining the suitability of soil for stabilisation. Soils with clay content below 30% can be stabilised using OPC, while those with more than 30% using lime. Lime is known to have the capacity to fix the clay, through a pozzolanic reaction.

Water is added to the dry soil-cement mix to hydrate the cement and to lubricate the soil to attain maximum densification. Unfortunately the literature reveals that current emphasis is generally placed on the required quantity of water, but not on its quality. Due to the scarcity of water in most developing countries, all kinds of sources are likely to be used for CSB production. Water of unknown service record may contain contaminants that may adversely affect the hydration reaction of OPC. Such water may contain suspended solids and soluble substances in excess of current limits for drinking water. The effect of the use of water of unknown service record is investigated and discussed in Chapter 4.

Soil-Cement Stabilisation

Chemical stabilisation of soil using OPC as the binder of choice was the main stabilisation method described. It was noted that the effect of cement stabilisation is more long lasting than pure mechanical or physical means of stabilisation used alone. It was also reported in the literature that the combination of all the three methods is more effective. It was noted in Chapter 3 that in the mechanism of stabilisation using OPC, the binder joins the soil particles together by forming strong interlocking bonds with the fine gravel and sand fractions of the soil. The lime that is released from the hydration reaction of OPC then reacts with the clay to form a secondary binder, with similar binding effects.

The composite matrix that results contains cement hydrates, conventional mortar, calcium hydroxide, unhydrated OPC residues, unstabilised soils, and capillary pores. According to CSB literature sources the exact proportions of each of these matrices in the block fabric is still unknown. The proportions of each of these matrices is likely to influence the durability of blocks significantly. Attempts are made in this thesis to identify some of these matrices using the petrographic analysis of thin sections (Chapter 7).

Block Production Cycle

It was established from literature sources that the CSB production cycle comprises four main stages, each with several sub-processes. The main stages include soil winning and preparation, soil-cement-water mixing, moulding and curing. It was noted that the processes were so interdependent and interrelated that they require to be conducted in a proper sequence. Omission of any of the stages is likely to adversely affect the properties of the final block.

With the preceding conclusions, the objectives of Chapter 3 were met. The key issues identified in Chapter 3 are to serve as the theoretical background for the experimental investigations described in Part B of this thesis.

PART B:

MAIN INVESTIGATIONS AND FINDINGS

CHAPTER 4

EXPOSURE CONDITION SURVEY OF CSB BUILDINGS

4.1 INTRODUCTION

The performance of CSBs can be better understood through a combination of theoretical knowledge, study of precedents and assessment of the experience of its users. This chapter reports on methods and findings from a study of the last two approaches.

As part of the research, a fieldwork was undertaken in Uganda between January and March 2000. Uganda was selected for two main reasons: firstly, its geographic location within the humid tropics, and secondly due to the large stock of CSB buildings found in the country. The exposure conditions were considered to be representative of similar conditions in most of the humid tropics. Further, CSBs were first officially introduced in the country for low cost housing in high density urban areas in 1987 (Okello, 1989). Since then, several hundred CSB structures were built mostly under the auspices of donor agencies like the ILO. During this period (1987 to 1995) the author, apart from involvement in similar projects in other parts of the country, directly supervised the construction of a number of residential buildings using CSBs (Kerali & Schmetser, 1995). At the time of the fieldwork visit, the use of CSBs for low-cost housing had been extended to other large urban municipalities in the country.

The objective of Chapter 4 is to report on the main methods and key findings from the fieldwork. Four methods were used during the fieldwork, namely:

- Collection of documented data on the inventory of CSB buildings and environmental exposure conditions in Uganda.
- Conduct of exposure condition survey of CSB structures of various ages and stages of completion. This involved: random inspection of existing buildings, in-service testing, scrutiny of maintenance records and other test records. Photographic records of inspected structures were then kept.
- Observation of methods of work on current CSB production sites including the conduct of suitability tests on soils and quality test checks on cement and water used.
- Interviews and questionnaires to gauge the opinions and experiences of randomly selected respondents.

The scope of coverage of Chapter 4 therefore focuses on the discussion of findings resulting from using the above methods. The chapter is divided into six sections. After this introductory section, the rest of the chapter covers background documentation, condition survey methods and findings, block production site visits, interviews and questionnaires, and conclusion.

4.2 BACKGROUND DOCUMENTATION

In this section, the inventory of CSB structures in Uganda and the characterisation of the exposure environment are presented.

4.2.1 INVENTORY OF EXISTING CSB BUILDINGS

The purpose of seeking information on the inventory of CSB buildings in the country was to obtain an indication of the overall total number of existing CSB structures. The same exercise was also used to get information on current building programmes and future plans.

Since the introduction of CSB structures in the country in the late 1980s, over 400 buildings have been constructed. As mentioned earlier, CSB structures were introduced officially under the auspices of the International Labour Organisation (ILO). The ILO was implementing an earlier resolution of the United Nations Conference on Shelter Strategy that had been held in Vancouver, Canada in 1978 (DoH, 1992). At the time, the projected housing backlog in Uganda by the year 2006 was estimated at 3 million dwellings. CSB structures were targeted at the high density, low income urban areas (Davidson & Payne, 1983; Taylor & Cotton, 1994).

Other CSB structures were built in rural areas in the form of public buildings such as health centres, schools, community centres, etc. These were initially built in the central region districts of Kampala, Luweero, Mpigi and Kiboga. The largest single housing estate in which CSBs were used remains at Namuwongo (in Kampala)where over 100 residential buildings were erected. At the time of the fieldwork visit, the project site consisted of buildings in various stages of completion (completed, on-going, abandoned, etc.). Also at the time of the visit, another large CSB project site had been initiated at Malukhu, in Mbale Municipality. Over 80 structures had been erected, with plans to construct at least 100 buildings annually over the next few years. This latter project was being funded by the Danish Agency for International Development (DANIDA). These two sites therefore represent the largest single concentrations of CSB structures in the country. Both sites were extensively inspected by the author and in-service tests conducted on buildings as well as on green blocks being produced. Photographs of some of the main features at the two sites were taken.

The findings are discussed in Sections 4.3, 4.4 and 4.5 of this Chapter.

Even with these promising developments, the housing deficit in the country, as is the case with many other developing countries, remains acute (Hamdi, 1995). The demand for CSB structures is therefore likely to remain very high in the foreseeable future.

4.2.2 Characteristics of the Natural Exposure Conditions in Uganda

Characterisation of the exposure environment in which CSBs were being used was considered to be a crucial undertaking during the fieldwork. The objective was to identify the main naturally occurring agents whose effects were likely to remain deleterious to the block structure over its service lifetime. The approach which led to the listing of the different types and ranges of deterioration agents was based on the deterioration mechanisms discussed in Chapter 2. The highlights of the mode of occurrence of the main deterioration agents (rain, temperature, relative humidity) and the results of the visual inspection of defects were used to produce a provisional severity ranking of deterioration mechanisms (Appendix D).

The type of agent acting on a block and the severity of its actions are closely correlated to the geographic location of the CSB building structure (BS 7543, 1992; Sjostrom et al, 1996). Moreover, local topography and geographic features are known to modify climate. Before presenting the average climatic conditions in Uganda therefore, it is necessary to first of all describe some of the main geographic characteristics of the country.

Uganda is located astride the Equator, lying between latitudes 4°12N and 1°29S, and within longitudes 29°34E and 35°0E. Although the total land area is 241,000 square kilometres, about 20% of it is covered by water, and about 30% by forests (Briggs, 1994; Hood, 1996; Tetley, 1998). Located at the highest altitude in Africa, the elevation above sea level (ASL) varies between 620 metres ASL and 5110 metres ASL. About 85% of the country lies between 900-1500 metres ASL. As can be expected, these geographic features (water bodies, forest cover, elevation ASL) do have a considerable influence in modifying the climatic conditions in the country. The use of CSBs under such unique climatic conditions can therefore be expected to present special problems. The description of the average climatic conditions in the country would not have been complete without mentioning these geographic features. In terms of macro-climatic and global weather classification, Uganda falls within the Equatorial belt. This humid, tropical belt stretches between 6°N and 6°S (BRE, 1980; Webb, 1988). The climatic characteristics of interest to this research are rainfall and temperature.

The *mean annual rainfall* in Uganda is about 1500 mm per annum (Hood, 1996). The rainfall, which is seasonal, is fairly well distributed throughout the country. Two distinguishable rainfall seasons are the long rains of March to May and the short rains of September to November. In analysing the potential deleterious effects of rainfall on CSB structures, it is the mode of occurrence of the rain within the immediate proximity of the block which is critical (intensity and duration of the rain) (Ola & Mbata, 1990). The intensity of rainfall in the country, a measure of the quantity of rain falling in a given time, is reported as being greater than 7.5 mm/hr. This falls within the classification for heavy rains (> 7.5 mm/hr) as opposed to light rains (< 2.5 mm/hr) or moderate rains (2.5 – 7.5 mm/hr) (Linsey et al, 1975; Wilson, 1993). The maximum fall of rain in any 24 hours was recorded as 300 mm in Ssesse Islands. The average drop size was reported to vary between 0.5mm and 6.0 mm (Wischmeier &

Smith, 1958; Kirkby & Morgan, 1980). The duration of rainfall in the country, a measure of the period of time in which it falls, also varies a great deal. Periods of between less than one hour and six hours are reported as being typical (Newman, 1986; McIllveen, 1998). It is well documented that the higher the intensity of rainfall in the country, the shorter is the duration in which it occurs. It is this intensity-duration relationship that can considerably influence the erosive potential of rain (Blanchard, 1948; Bilham, 1962). The erosivity of rain can also be determined by the rain drop-size, its distribution, fall velocity and impact kinetic energy (Ellison, 1944). As can be expected, an erosive threshold below which no surface erosion will take place ought to exist. A similar approach has been successfully used in soil erosion sciences. It was established that the erosive threshold for loose soil in terms of rainfall intensity was about 25 mm/hr (Kirkby & Morgan, 1980). This is a theoretical cut-off point above which erosion of soil can take place. Since CSBs are much denser, stronger and more structurally stable than natural soils, the erosive threshold is likely to be several times higher than the 25 mm/hr suggested for loose and weakly bonded natural soils. Intense rainfall on a CSB surface is more likely to initially wet the surface and generate surface flow than immediately dislodge material from the block surface. The mechanism of water-related deterioration was discussed in detail in Chapter 2. The rainfall characteristics in the country suggest that water-related deterioration of exposed block surfaces is likely to take place during the service lifetime of the block. Defects associated with this mechanism of deterioration are described in Section 4.3 of this Chapter.

The *ambient temperature* in the country is quite high. The average daily ambient temperature is 25°C. The highest mean daily temperature recorded in the country was 35°C (Karamoja region in the dry north east). The lowest mean temperature recorded was -5°C (at the peak of the Rumenzori Mountains in the west of the country). The sunshine hours in the country average between 8 and 10 hours. The mean total evaporation is reported as 1950 mm (Hood, 1996; Tetley, 1998). As can be expected these temperature conditions provide the basic setting for temperature-related deterioration to occur in blocks. Mechanisms of temperature-related deterioration were discussed in Chapter 2. The presence of large, fresh water bodies in the country such as lakes and rivers and the high temperatures ensure that the level of humidity in the country is also high. Typical ranges for relative humidity are reported as lying between 30 and 90% depending on the cloud cover.

The data presented in this section was considered to be adequate enough to provide sufficient information to link the deterioration of CSB structures to the most common deterioration agents known. The condition survey that follows describes in more detail the common types of defects found on exposed CSB wall structures.

4.3 CONDITION SURVEY METHODS AND FINDINGS

The condition survey of exposed CSB structures was perhaps the most important undertaking during the research. Three methods were used for the survey, namely:

- Visual inspection (of CSB buildings)
- In-service condition measurement (of defects)
- Field indicator soil testing (at the major CSB project sites)

These are now discussed each in turn, in the following sections, 4.3.1, 4.3.2 and 4.3.3.

4.3.1 VISUAL INSPECTION OF EXPOSED CSB STRUCTURES

As with most building materials, the initial detection of their exposure performance is initially based on visual inspection (BRE, 1982; Bungay & Millard, 1996). Visual inspection is therefore the first phase of any in-service evaluation of a material such as a CSB. In this section, the following are discussed:

- Reasons for choosing visual inspection as a method for evaluating the performance of CSBs
- The number of types of CSB buildings inspected
- The type and range of defects observed

Visual inspection as a way of assessing the performance of CSBs under natural exposure conditions was selected for several reasons. They include the following:

- the CSB specimens being inspected within the exposed wall structure are at their 'full scale' during the assessment. This makes it possible to closely examine their current condition on a full scale basis. Any defects due to dimensional changes and the effects of the restraining action of adjacent blocks and mortar, can be observed directly. The effect of such restraint is very difficult to accurately simulate experimentally.
- the weathering conditions under which the defects were caused are genuine. Because of this, the full effects of the entire range and distribution of deterioration agents acting on the wall surface can be directly observed. A cause-effect link between defect and action of agent(s) can be deduced.
- through visual inspection, more severe cases of deterioration can be distinguished from less severe ones. Using the severity ranking (defects, agents), further in-service tests and measurements can be recommended based on visual observations. The selection of test types can only follow on from the visual inspection report. This is time and cost saving (BS 8210, 1986).
- it is possible to use a number of non-destructive measurement techniques and instruments. Some of the instruments used included: depth gauges, electronic

callipers, crack gauges, hand-held microscopes, rulers, set-squares, etc.

- the in-use conditions of the buildings being inspected are genuine. All user induced influences on the normal wear and tear of the CSB structure can be assessed.
- through the use of a sufficient number of samples, it is possible to reach fairly reliable results and therefore generalise. In this way the interpretation of findings from visual inspection can be considerably facilitated.

With the above reasons in mind, a systematic inspection was made of several CSB buildings in Uganda.

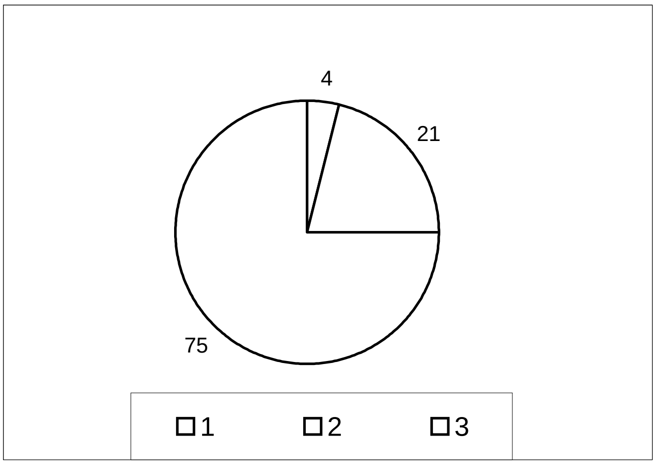
The *number and types* of CSB buildings inspected were varied, all chosen at random. Seven out of the thirty five districts where CSBs had been used for building were visited. In this way a total of 58 CSB buildings were inspected, representing a sample size of about 15% of the officially recorded number of CSB buildings in the country (above the 10% minimum normally required statistically for reliable inferences to be made). Using a checklist of all possible types of defects, the average time taken to inspect each structure was about 45 minutes. The inspected buildings were of different periods of exposure ranging from one month following substantial completion to those with over twelve years of exposure. The buildings were also in various stages of completion: completed, on-going construction and abandoned structures. Buildings found abandoned at wall-plate level and below without roofing appeared to be the most severely damaged (equitable to normal experimental exposure situations). It is from such structures that further in-service measurements (recessed volume of block, width of cracks, degree of pitting and roughening, etc.) were made. These are discussed in Section 4.3.2 that follows.

The line Ministry of Works and Housing provided background information on the period of exposure of each building, types of constituent materials used and the processing method employed to make the blocks. By compiling the list of defects and comparing the findings with the information obtained on each building, a number of useful conclusions were made.

The *type and range* of defects observed from the fifty-eight CSB buildings inspected are summarised in Appendix E. Also shown are the type of defects, façade and section of wall in which they occur, likely causes, age of structure and frequency with which the defects were observed. Comments on some of the features observed by the author, together with information from the users and co-inspectors, are also included.

From the information summarised in Appendix E, it is noted that:

- Surface erosion (resulting in mass loss, or volume reduction) and surface cracking (resulting in bond breakage and segregation) are the two most common defects observed in CSB structures left exposed to the elements. In the tally of number of buildings inspected and frequency with which a particular defect was observed, surface erosion (including roughening and pitting) occurred in 75% of all the cases. Surface and bulk cracking occurred in 21% of all cases. Other defects all counted together only occurred in 4% of the buildings observed. The above results are shown in the form of a pie chart in figure 2.



Key:

1: Others (4%)
2: Surface and bulk cracking (including crazing) (21%)
3: Surface erosion (including pitting and roughening (75%)

Figure 2: Relative frequency of observed common defects in CSB buildings in Uganda (January-March 2000)

- Surface erosion occurs more severely on the lower sections of a wall, rather than on the middle and upper sections. The combined effect of direct abrasive action of rainwater, surface run-off and splash from the ground is thought be account for this difference (Chapter 2).
- Surface cracking and crazing occur more on the east-west facades than on the north-south facades. With the country located astride the Equator, the effect of the direction and period of sustained solar radiation from the east (sunrise) and west (sunset) ought to be taken into account when explaining the difference.
- Cracking of the bulk mostly occurs within the framework of the wall rather than in the corners. The unusually thick and non-uniform mortars used (10mm to 20mm) is believed to be responsible for some of the cracking in the bulk. Mortars are designed to be weaker than the block to allow for flexibility due to dimensional changes. Where the mortar is unnecessarily thick, the restraint on movement can result in enhanced cracking (Neville & Brooks, 1994; Walton,

1995).

- The corners of walls were the worst affected. A likely explanation is that at wall corners, rain is able to strike the block from all directions. Moreover, wind velocities are highest at corners. The level of erosion is therefore likely to be higher in such parts of the wall than in others.
- Defects due to causes other than environmental factors can also occur in CSBs. These include defects due to overall foundation settlement, biological agents and impact from users. Also observed were defects related to improper material design, workmanship and processing methods (Odul, 1984).

The results of the visual inspection of exposed CSB buildings confirm that premature deterioration of CSBs can occur in humid tropical environments. In Section 4.3.2, the extent of surface erosion and cracking, being the two most common defects observed, are determined by direct measurement for the most severely affected units.

4.3.2 IN-SERVICE MEASUREMENT OF VOLUME REDUCTION, DEPTH OF PITTING AND CRACK WIDTHS

As mentioned in the previous section, the two most common defect types (surface erosion and cracking) were identified for further direct measurement. The measurements were conducted on two of the oldest exposed structures located at the Namuwongo Urban Slum Upgrading project site in Kampala City. Both CSB structures had been left abandoned at wall-plate level without roofing. A third building, also abandoned at a similar level, that had also been selected for similar assessment, was inaccessible (recently fenced off for rebuilding). The two structures were taken as being representative of the worst case of severe deterioration from long-term exposure. The walls could be equated to similar walls built on normal experimental exposure test sites (BRE, 1980). Lack of protection from environmental elements due to the absence of roof cover and external render meant that the full extent of deterioration from weathering agents could be said to have reached its maximum during the eight and twelve years of exposure respectively. Moreover, the weathering conditions (normal and severe) under which the defects were caused were all genuine.

The direct measurements involved assessment of the following:

- Volume reduction (including pitting depth) to estimate the extent of surface erosion
- Crack width measurements

The methods and results obtained for each defect type are now described.

Estimates of Surface Erosion (by volume reduction)

Surface erosion leads to irrecoverable mass loss. This in turn results in the reduction of the volume of a block. By measuring the overall depth, width and length of surface material lost due to erosion, the volume of the recessed block surface can be determined. By deducting the recessed volume of the block from the original volume (determined from original block dimensions), a volume reduction percentage for each block can be obtained.

The procedure adopted to obtain the recessed volume for blocks in each of the two abandoned buildings was the same. For each building, thirty six blocks per building were measured. This total number was arrived at as follows. For each abandoned building, the nine most severely affected blocks per façade (north, east, south, west) were identified for measurement. The nine blocks on each façade comprised three blocks each from the upper, middle and lower courses of the wall. In this way, not only would any differences in defect severity per façade be obtained, but also differences per section of the wall in which the block was embedded. Where the degree of recession was high, the determination of the recessed volume was easy to measure and calculate. Where loss of mass was spread out on the block, the block surface was divided into four sectors. In each sector, the dimensions of recession were measured, and the total recessed volume obtained by adding up. All measurements were done using an electronic calliper complete with a depth guage (Mitutoyo brand). This light, hand-held calliper displayed the depth, width and length of eroded surface zones directly on its mini-screen. From the results, the histogram shown in figure 3 was obtained for each building. They show the volume reduction percent (%) for each wall façade and sector.

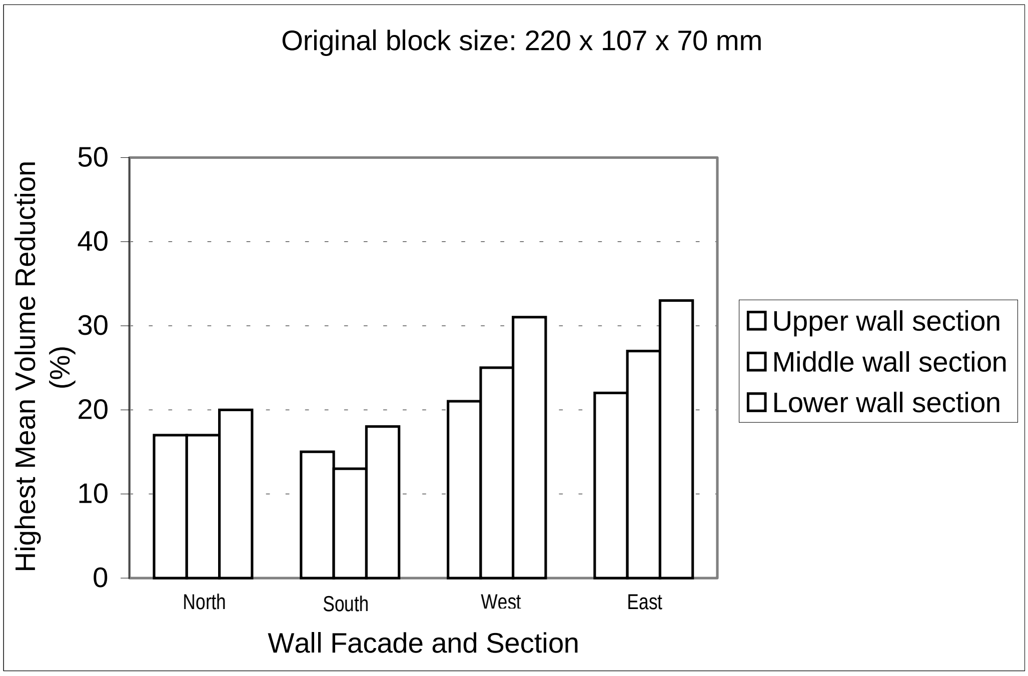


Figure 3(a): Abandoned building (NAB1: 8 years exposure)

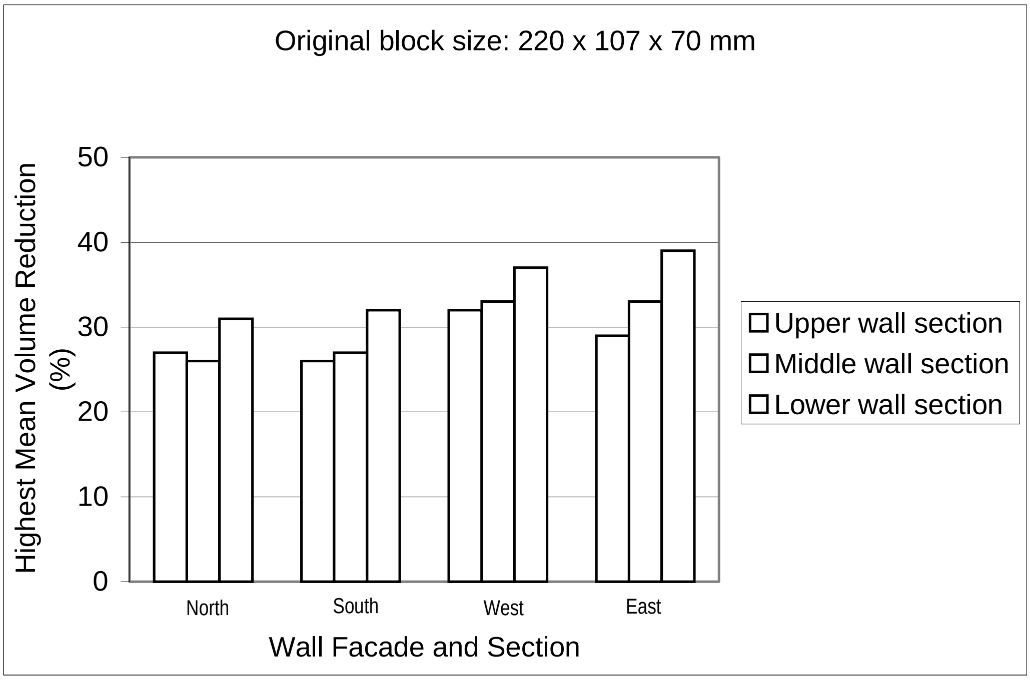


Figure 3(b): Abandoned building (NAB2: 12 years exposure)

Histograms showing the highest mean volume reduction percentage for each wall façade and sector for NAB1 and NAB2 (Uganda, January – March 2000)

From the histograms in figure 3, it can be confirmed that mass loss resulting in volume reduction does occur when CSBs are left exposed and unprotected in a humid, tropical environment. The reduction in volume is however not the same for all facades and levels in a wall. The results show that surface erosion varies according to the:

- elevation of the block within the wall (upper, middle and lower sections)
- orientation of the façade (north-south and east-west)
- age of the building (period of exposure under similar conditions)

The explanation for the above variations are likely to be several as discussed in the following sections.

The *elevation of a block* within the upper, middle or lower section of a wall can influence the rate of deterioration for several reasons. To begin with, the author was advised by the users and technical staff that most of the surface erosion appeared during the rainy seasons. Very little surface erosion if any occurred during the dry seasons. The main mechanism for surface erosion is therefore rainwater related. In each wall sector, although the amount and intensity of rain striking the wall might be about the same, the overall effect varies.

The histograms show that the reduction in volume is greater at the lower courses of a wall than at the higher ones. For the lower courses in both structures, volume reduction percentage for the most severely deteriorated blocks varied between 18% and 34% in NAB1, and between 30% and 38% in NAB2. Similar values for the upper sections were between 15% and 22% in NAB1, and between 26% and 28% in NAB2.

The amount of surface run-off created by raindrop splash from the upper section of a wall, and from the ground appear to contribute to the higher erosion at the lower wall sections. At the upper wall sector, whereas raindrops may strike the block surface, the surface might not begin to erode immediately. The raindrop striking the surface expends some of its energy in striking the wall and some in creating a splash (Chapter 2). It is the splash which then wets the block surface and may also progressively soften it. Erosion is likely to take place after a period of wetting and softening of the surface fabric. Meantime, the accumulation of rain splash transformed into surface run-off will flow downwards along the vertical profile of a wall. In the process, the middle and lower sections of the wall, in addition to being struck directly by rain drops from the same storm, will have to contend with the surface flow from the upper sections. The surface flow can increase in momentum and volume, washing away any loose soil particles from the blocks along its path. It is unfortunate that for the lower course blocks, surface erosion can be further increased by back-splash from raindrops striking the apron or ground immediately below it. The combination of direct raindrop impact, spray surface run-off and ground back splash appear to account for the increased severity of surface erosion in the lower courses of a wall than in the higher ones. As can be expected, the effect of raindrop erosion can be considerably increased in storms accompanied by strong winds (> 20 m/s). Despite these theories, the mechanism of rain erosion on CSB structures is not yet well understood. A considerable scope for reappraisal and review still remains.

Another observation made was that the lower corners of walls appeared to be more severely eroded that similar blocks at the same level. The fact that it is only at the corners that rain from all directions can strike the block is thought to account for this variation. More research is needed into this and other phenomena associated with raindrop erosion of block surfaces.

The *orientation of a wall façade* (north-south and east-west) appears to affect the extent of volume reduction. The highest average volume reduction percentage for east-west facing walls for NAB1 and NAB2 were 27% and 34% respectively. The highest average volume reduction percent for north-south facing facades were 17% and 28% for the former and latter buildings respectively. Corresponding volumes for roofed buildings can be expected to be lower. Whereas several explanations to account for the differences may exist, the most plausible one is likely to be connected to the direction of sunrise (east) and sunset (west). While all facades may experience similar amounts and intensity of rain abrasion, the east-west facades may dry up much faster on the reappearance of the sun soon after the rains stop. The duration of most storms in the humid tropics as mentioned earlier in the thesis is short (between 2-6 hours at a time). After such short periods of wetting, the reappearance of the sun can ensure that the wet block surfaces absorb considerable amounts of solar radiation. Absorption of solar radiation causes temperatures of the block surfaces to rise. This warming up effect can cause the block surface to dry up more quickly on the east-west facades than on the north-south facades. This can happen within a matter of only a few hours, causing moisture to evaporate from the wall surface, thus changing the moisture profile in the block.

The absorbed radiation can raise the temperature of the block by an amount depending on the specific heat of the block (on average between 0.65 and 1.00 kJ/kg), and on the thermal conductivity of the block (on average between 0.23 and 1.04 W/m°C) (Houben et al, 1994). As both values are positive for CSBs, thermal expansion and contraction of a block surface can therefore occur with changes in temperature. This is likely to lead to both temporary and permanent alterations in the physical and chemical properties of the block. Surfaces of blocks experiencing such cyclic changes in temperature can ultimately crack. Cracking can then expose the block surface to easy entry of moisture. Moisture within a block is likely to initiate otherwise dormant chemical activity between the constituent materials which make up the material. The range of chemical actions likely to occur were discussed in Chapter 2.

This phenomenon is also likely to occur in the reverse order of heating and cooling. Before the rainy seasons, sunlight can heat the block surfaces very fast (more on the east-west facades than on the north-south facades). Raindrops striking the already hot block surfaces can apply a severe quenching shock to it. The bonds between the soil particles and OPC hydrates can thus experience their first disruptive action (Baker et al, 1991). This can lead to weakening of the surface fabric thus exposing it to further abrasive attacks from raindrops. Surfaces which are weak can be easier to erode than those which are more intact. The combined cyclic action of wetting-and-drying can progressively lead not only to mass loss, but also to loss of strength, loss of hardness, rigidity and stiffness, as well as loss of appearance (pitting, roughening, cracking, etc.)

(ASTM D 559-55, 1975).

The *period of exposure* corresponding to the age of a CSB structure also appears to affect the amount of deterioration in the block. For the two buildings (NAB1 and NAB2) which were made from like materials and exposed under similar natural conditions in the same locality, the amount of deterioration varied according to the period of exposure. NAB1 had been left exposed for eight years, while NAB2 exposed for 12 years. The amount of deterioration in NAB1 was markedly less than that in NAB2 for each façade and at all wall levels. The highest average volume reduction percentage in NAB1 within the eight year period was 22%, while the corresponding amount for NAB2 within 12 years was 31%. Other factors being constant, the highest estimated annual volume reduction rate for NAB1 was 2.75% per annum, while that for NAB2 was 2.58% per annum. The difference in the two rates was only 0.17%. This result shows a certain degree of convergence. It can be interpreted to mean that, on the basis of the measurements taken, the highest average rate of volume reduction percent in CSB structures exposed under similar circumstances can be expected to be less than about 3% per annum.

The rate of volume reduction is likely to be influenced by the degree of resistance to surface abrasion that the block can offer. A block surface that is smooth, impermeable, non-reactive and of high inter-granular strength, is likely to offer more resistance to surface erosion than one which is not. The abrasion resistance of block surfaces can be increased in a number of ways. These include the use of surface render, surface coating and surface layering with mixes of higher inter-granular strength. These protective procedures can transform the block surface into a layer of significantly greater wearing resistance. As mentioned earlier in the thesis, protection of block surface should remain the main strategy in enhancing its durability. If the block surface is eroded, exposure of its core to similar deleterious action can prove to be more severe since the bulk is its least compacted zone (Houben & Guillaud, 1994; Gooding, 1994). Ways of improving the durability of blocks through the use of CRMs are investigated experimentally in Chapters 6 and 7.

Crack dimension measurements

Cracking on CSB surfaces, sometimes extending deep into the bulk, were commonly observed defects. Classification of the main crack patterns and direct measurement of the most extensive crack widths were done in order to link them to likely deterioration mechanisms and to assess the severity of the phenomena. It is the width of a crack, rather than its length or depth, that is commonly measured in like building materials (Neville, 1995). Moreover, maximum permissible crack dimensions are normally specified strictly according to limits based on crack widths.

The procedure adopted involved visual identification of three of the most badly affected blocks on each wall façade then measuring their crack widths. The average of the greatest crack widths from each of the three blocks were then determined. To make the measurements, two hand-held crack width measuring instruments were used, namely: an optical crack microscope and a crack comparator scale (Baker et al, 1991; Sjostrom et al, 1996). Both instruments were originally developed for measuring similar cracks in concrete structures, and the author had used them several times before. Use of the two instruments side by side did not present any difficulties. The crack microscope used was of the 'ULTRA LOMARA' Mess-Mikroskop make. The instrument powered by a battery, was held against each block surface right over the crack to be measured. The surface was then illuminated by the small internal bulb within the instrument and the magnitude of the crack width measured directly by comparing it against the internal graduated scale that was clearly visible through the eyepiece.

To complement the measurement, the simple hand-held (unmagnified) comparator scale was also used for estimating the same crack width (sometimes referred to as the crack calculator). The type used was the Colebrand/Abbot Brown crack calculator. The procedure involved is slightly different. To estimate the crack width using this instrument, the comparator was placed directly against the targeted crack on the block. By sliding it upwards or downwards until a comparable thickness was determined, the crack width could then be estimated accordingly. The range of crack widths on the comparator ranged from 0.100 mm to 2.0 mm. Crack widths wider than this maximum value had to be estimated using an electronic calliper whose double tips were inserted between the cracks and extended in opposite directions till firm contact was made. The use of these instruments was found to be necessary because, allowing for human eye variations, the minimum crack width that can be seen by the naked human eye is about 0.13 mm (Neville, 1995). As the procedure was laborious and time limited, measurements were only done on NAB2. The summary of the widest average dimensions of crack widths measured are shown in the histogram in figure 4.

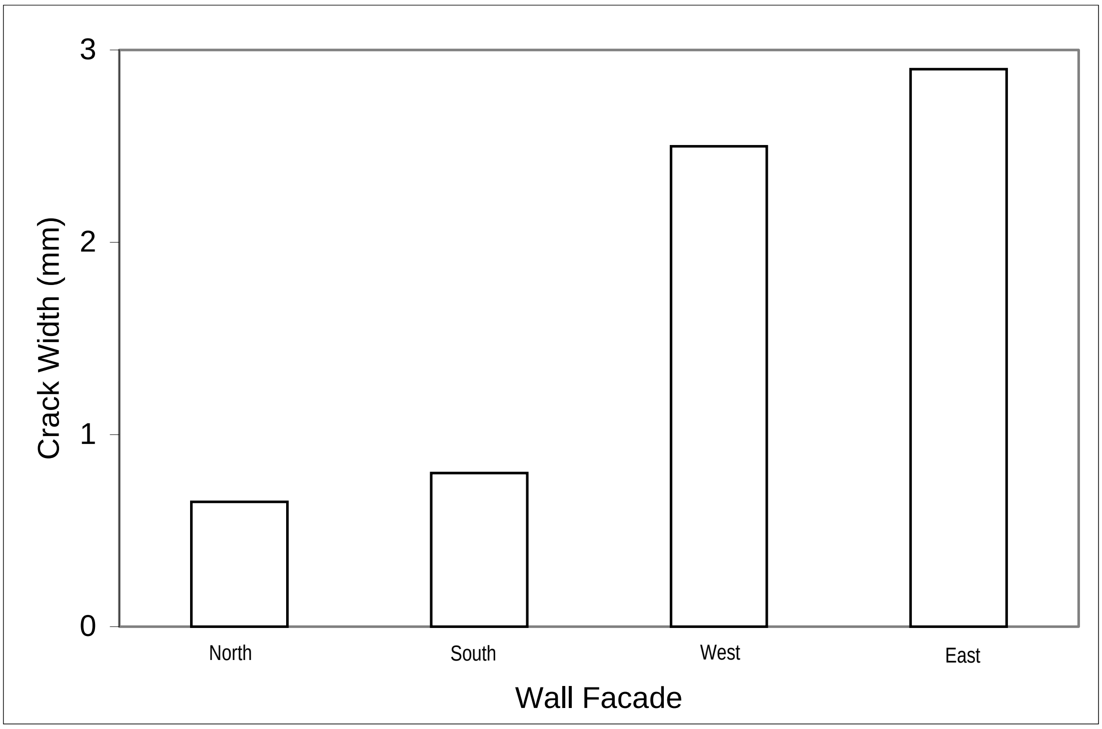


Figure 4: Histogram showing the mean crack width of three of the worst affected blocks on each wall façade for NAB2 (Uganda, January-March, 2000)

From the histogram shown in figure 4, it can be seen that the phenomenon of cracking occurs on all the wall facades of an exposed CSB structure. The highest mean maximum size crack width measured was 2.9 mm on the east façade of the building. The corresponding lowest mean crack width was 0.65 mm on the north façade. These results compare unfavourably with the maximum permissible crack width limits normally specified for concrete structures (Neville, 1995). The permissible crack width for exterior wall concrete used in normal, and under severe conditions of exposure are given as 0.25 mm and 0.15 mm respectively. The values obtained from the measurements of exposed CSB walling units (0.65 mm to 2.9 mm) are much higher than the permissible limits for concrete. The comparison will however need to take into account the presence of clay in CSBs as opposed to concret where its presence is not allowed. The presence of clay (amount and type) is likely to severely affect the magnitude of cracking in CSBs (Spence & Cook, 1983).

The results also show that cracking is more pronounced on the east-west facades than on the north-south facades. The reasons for the differences were discussed earlier. It should be mentioned here that whereas the kind of cracks measured on a block may simply be symptoms of many causes, the common feature is that they all result from the restraint made on strains. Since stress and strain are supposed to occur together, any restrain of movement can introduce a stress corresponding to the restrained strain. If these stresses and the restrained strain are allowed to develop to the extent that they exceed the strength or strain capacity of the block, then cracking can occur. The diagnosis of the exact cause of cracks in a block might therefore not always be that straightforward. Indeed, cracks can be a result of several causes such as: plastic shrinkage, drying shrinkage (clay and OPC hydrates), chemical action resulting in expanded product formation within the block fabric, settlement of the foundation, improper curing and thermal movement. Some of the mechanisms involved were discussed in Chapter 2.

In summary, whereas a particular cause within or outside the block might be responsible for initiating a crack, its subsequent development and propagation may be due to other causes. The types of cracks observed on CSB structures (star shaped, linear, interconnected) could therefore have been a result of more than just one cause. Further research is recommended to link particular crack patterns in CSBs to specific deterioration mechanisms. Limits such as have been specified for concrete should also be set for CSBs. Such limits should however take into account the presence of clay in CSBs. The limits can be specified as maximum permissible crack widths for use of CSBs under normal and severe exposure conditions.

4.3.3 *FIELD INDICATOR SOIL TEST RESULTS*

In this section, field indicator soil tests conducted during the fieldwork, together with the results obtained, are described. The results are compared with available laboratory test results for the same soils at the two major CSB project sites in the country (Namuwongo in Kampala and Malukhu in Mbale). This section covers the following:

- Need for field indicator tests
- Types of indicator soil tests available
- Results of conducted indicator soil tests

Need for field indicator tests for soils

In order to classify a particular soil, two types of tests can be done: indicator (or field tests) and laboratory tests (Webb, 1988; Norton, 1997). Only the former will be described in this section since the latter are presented in Chapter 5. The two test categories should normally be done together with the one following the other (indicator, then laboratory testing). It is normal to conduct indicator soil tests first since it is from the results obtained that justification for further laboratory testing is usually based.

Although field indicator testing might be regarded as being preliminary, it is only through such tests that the rapid evaluation of important soil properties can be made. The tests might also appear empirical but they enable the general suitability and acceptability of a soil for CSB production to be determined quickly. As can be expected, there are now many types of indicator soil tests. The common factor in all the tests remains their relative simplicity and speed of execution. These tests are also quite inexpensive to conduct given that they require little or no equipment. The only drawback (according to the experience of the author) is that these tests over-rely on the competence and judgement of the operator. The veracity of interpretation by the operator is often taken for granted despite errors likely to be caused either by human weakness or lack of competence. With better experience and training however, such problems should be easy to overcome. Soil indicator testing is likely to continue to be highly regarded as they help avoid unplanned and premature laboratory testing. At the moment, there can be no serious alternative to soil indicator testing as a precursor to laboratory testing.

Types of indicator soil tests available

There are currently several soil indicator test types to choose from. Over the years, a multitude of test types have been put forward by various authors and researchers. Despite the numbers, the common objective remains the identification of the presence and predominance of the main soil fractions (gravel, sand, silt and clay). After the various types of field indicator tests have been done, further laboratory testing may follow to determine the precise proportions of each soil fraction, and perhaps more importantly, the overall behaviour of the soil type when in contact with water.

In order to make the comprehensive review of current available field indicator test types easier, a tabulated summary listing the test methods has been drawn up. According to literature sources, up to 15 different types of soil indicator field tests are currently available for preliminary use in determining the suitability of a soil for CSB production. The different types of tests and the various authors who have reported them, are presented in Appendix F.

Not all authors managed to describe all the available tests with the exception of Houben & Guillaud (1994) and Stulz & Mukerji (1988). Although some of the authors attempted to combine some of the tests, or tried to use different names for the same test, the summary list was compiled to show separate distinct tests. Tests with similar names are shown in brackets. During the fieldwork, all the listed tests with the exception of number 10 (Dry Strength) and number 14 (Decantation), were done. The tests were done on soils taken from the sub-soil level after removal of the top soil (varying in depth between 150-300 mm). This was done to ensure that the presence of organic matter (normally found at the top-soil level), was avoided. Apart from trying to ascertain the suitability of soils used at the two largest CSB project sites, the tests were also conducted to get a feel of the operational difficulties and levels of accuracy and convergence expected. The main procedural steps involved in each test can be found in the references shown. The results are presented in Appendix G.

Summary results of soil indicator tests done in Uganda

From the summary of the results shown in Appendix G, it is noted that:

- there was no significant presence of organic matter (the soil is likely to be suitable for CSB production)
- the fines content (silt and clay) in both soils are high enough (24-35%) (the soil is likely to be suitable for CSB production)
- the coarse soil fraction content (fine gravel and sand) is above 60% in both soils (falls within the recommended limits) (Chapter 3).

With these preliminary results, the soils at the two project sites were found to be suitable for CSB production. The soil indicator test results were later compared to earlier documented laboratory test results from the Namuwongo project site which had been done in 1986 (Okello, 1989). For comparison purposes, extracts from the laboratory test results showing particle size distribution, linear shrinkage, sedimentation, natural moisture content, as well some of the test results form the initial performance testing of cured blocks, are presented in Appendix H.

The laboratory test results in Appendix H confirm that the soil used for CSB production at Namuwongo:

- was well-graded, with adequate percentage of fines and course soil fractions
- had moderate shrinkage levels, confirming a low to medium proportion of clay in the fines
- was found to contain almost similar levels of fines and coarse fractions using both the sedimentation test (soil indicator test) and the particle size distribution test (laboratory test)
- had a moderate natural moisture content.

The records from the initial performance tests done on blocks produced from the above soil showed that the blocks compared well with most minimum requirements of performance. The average wet compressive strength was above the minimum recommended values of between 1.2 MPa and 2.8 MPa (Lunt, 1980; ILO, 1987; Houben & Guillaud, 1994). The mean total water absorption capacity for the blocks were lower than the maximum permitted value of 15% (ILO, 1987). These results confirm that, in any post mortem diagnosis of possible causes of premature deterioration of exposed CSBs within the area, the inclusion of non-suitability of the soil used may not make sense. Any premature deterioration will therefore have come from factors other than soil selection and suitability.

The near convergence of field indicator test results and those obtained from laboratory records for the same soil further confirms that the former can be a very useful indicator of soil suitability. The field indicator tests should however be done following a logical order to ensure a coherent approach to testing. Use of soil indicator tests are especially recommended for CSB production sites in rural areas where no sophisticated equipment exists and where the cost of direct laboratory testing might be quite prohibitive. On very large CSB project sites, however, both field and laboratory tests should be conducted so that results from each category can be used to compliment the other. Moreover for large project sites, especially in areas known to be underlain by special soil types such as laterites, additional laboratory tests will need to be done. From Appendix H, it can be seen that this consideration was overlooked. Additional laboratory tests should have been done to provide information on the following:

- the plasticity index of the soil (using Atterberg limit tests)
- the acidity of the soil (using pH value tests)
- chemical composition of the soil minerals (using chemical analysis tests)

4.4 INSPECTION OF CSB PRODUCTION SITES

In Chapter 3, it was stated that the block production process was one of the three major influencing variables that can affect the properties and long-term performance of CSBs. The other two major variables of equally significant influence were identified as constituent material quality and action of environmental agents. In the CSB production process, any departure from widely accepted good site practice is likely to adversely affect the quality of the block produced (Guillaud et al, 1996).

In this section, results from the following investigation methods are presented and discussed:

- visits to block production sites (at two on-going CSB project locations in Uganda)
- quality checks on OPC and water used on CSB production sites.

Each of the above are discussed separately in Sections 4.4.1 and 4.4.2, that follow.

4.4.1 EVALUATION OF BLOCK PRODUCTION PROCESSES AND PRACTICE

The objective of inspecting production sites was to assess the organisational set-up of the site, and to compare individual production sub-processes against a pre-prepared checklist of good practice. Departures from the norm were carefully noted.

Two on-going project sites were visited: the large CSB building site at Malukhu (as before) and the smaller, single residential building site at Temangalo (farm in Mpigi district). While the former is an extensive project site with over 80 CSB structures built and another 200 or more planned, the latter is a single residential unit. At both locations however, the same type of machine was being used to produce blocks. The description of the machine is as below:

- Make: Hydraform block making machine (from South Africa)
- Type: Motorised diesel engine 10 kW air cooled
- Dimensions: 1000 l x 1400 w x 1300 h
- Weight: 750 kg
- Output: + 130 blocks per hour
- Mould: various (including interlocking dry stacking blocks)
- Block size: 240 x 220 x 115 mm; and 200 x 220 x 115 mm

At both locations, there was no centralised yard for mixing, proportioning, etc., as was described in Chapter 3. Details of the observations made are summarised in Appendix I.

From the summary findings shown in Appendix I, the following are deduced:

- Pre-extraction soil test records are not kept on site or nearby where they can be referred to. While manual extraction may be suitable for a small site (output about 1-3 m^3/day/man), mechanical extraction would be preferable on the larger project sites (output about 100 m^3/hour). The extracted soils are not prepared well before further use. They should be properly dried out and pulverised. Soils that have been dried out and screened ought to be stored in a protected area to preserve their moisture state and avoid changes in moisture content.

- The concept of batching is not closely followed so wastage and misuse of the stabiliser is likely. While proportioning is done by volume, checking that the gauge containers are properly levelled off each time is not strictly enforced. The stabiliser is mixed with the soil irrespective of the latter's moisture condition. As no obvious mix indicator such as the achievement of uniform colouration of mix is used, insufficient and uneven distribution of the stabiliser and water in the soil are possible. Moreover both water and stabiliser are poured down on the heaped soil instead of on a spread out soil. As no drop test is used to check the consistency of the mix, compression of blocks will take place below or way above the optimum moisture content of the soil. This will result in poor compaction of the soil, with blocks of low density being obtained. Moreover, the mixing process is also not closely supervised.
- Measuring out of the soil mix fed into the mould is not strictly done. This can result in variation in density and sizes of blocks produced. The filling of the mould is not done in layers and corners of the mix in the mould are not pressed by hand. Moreover, in motorised units as the compression force remains the same, it is important to check each time if correct filling is done.
- Demoulded green blocks are removed by hand, without use of pincers or

wooden pieces that could ensure that a large surface area is placed in contact with the yet weak block. No special attention was being given to the corners and edges of green blocks. Random quality checks normally conducted at this stage for each batch of blocks produced, were not being done as required. These shortcomings are likely to compromise the quality of blocks produced.

- Curing conditions appear not to be clearly categorised into wet and dry stages. Curing was being done under light cover under direct sunshine. Blocks were apparently being used earlier than the specified curing periods for the OPC and lime (28 and 56 days respectively). Due to high evaporation rates and pre-mature use of blocks, the likelihood of low quality blocks being used cannot be ruled out. Moreover, the blocks being cured are not separated according to batches and are poorly stockpiled at random. This could lead to the use of improperly cured blocks.

It was not possible to immediately evaluate the affects of these variables on the performance of the blocks produced. The observations do confirm that poor site practice and bad workmanship do take place (constituting significant variables likely to affect the quality and properties of a block). There is clearly a big difference between block production under strict laboratory procedures and field practice. The above problems can be attributed directly to the absence of codes of practice and checklists that should normally accompany trade standards. The findings also confirm earlier fears that variations in processing methods, especially due to inadvertent departures from the norms, could severely influence the durability of the block. Block production processes should be properly executed under close supervision if good quality and durable blocks are to be produced. It is recommended that all current impediments to the dissemination of standards and codes of practice for the production and use of CSBs be identified and resolved (Lowe, 1998; Schildermann, 1998). The above findings and their implications were brought to the attention of the supervisors found at the project sites at the time. It was clear that they had not received any prior briefing on good site practice, and could not therefore appreciate the adverse implications of their actions.

4.4.2 FINDINGS FROM QUALITY CHECKS ON OPC AND WATER

In Chapter 3, the importance of the quality of each of the three constituent materials used in the production of CSBs (soil, cement, water) were emphasised. The quality of soil used on CSB production sites has already been reported on (4.3.3). In this section, attention is focused on the basic quality checks conducted on OPC and water found being used on CSB building sites.

Although the objective of the tests at the time was to routinely ascertain the quality of OPC being used, the results obtained were rather surprising. Broadly it was found that both cement and water quality were poor, so the study was extended to cover why this was so. Although the quality of these ingredients were examined from a CSB production standpoint, the findings also have implications for all the cement-using activities in Uganda. Most of the OPC used in East Africa is expected to conform to the requirements of BS 12: 1991. It is also normally included in bills of quantities and specifications that contractors, users and consultants carry out periodic quality checks on any products in which OPC has been used. Despite the existence of this requirement, it is the normal practice in countries like Uganda to take the quality of OPC supplied (in sealed 25 kg bags) for granted. In addition, although the quality of water used for mixing of soil and cement and for wet curing of CSBs is required to be high, the normal practice on block production sites appears to disregard this consideration. It is not yet clear whether this is due to scarcity of water or other reasons such as lack of awareness of the dangers involved in using poor quality water (Chapter 3). The highlights of the procedures used for the quality checks are described next.

There are several forms of quality checks that can be done on OPC: comparing setting times, strength, or even chemical composition with standard requirements (BS 4550: Part 2, 1970; ASTM c 114-88). Tests involving the analysis of the chemical composition of OPC were considered to be beyond the scope of this research. Instead, the quality checks used were based on the comparison of the values of the wet compressive strength and tensile strength of prisms made from the OPC in question and the specified values from prevailing standards.

In the wet compressive strength test, three 50 x 50 x 50 mm cement and sand mortar prisms were cast. The prisms were made from a cement-sand mix proportion of 1 : 2.75 with a water-cement ratio of 0.49. They were cured under controlled conditions for 28 days (in water at temperatures of about 23°C). After 28 days, the cubes were tested and the value of the mean wet compressive strength of the prism made from the OPC in question, obtained. The results were compared to those specified for the type of OPC that was being used on site. The results are shown in table 2. To check the quality of water used, the same procedure was followed but this time using the water of unknown service record (found being used on the block production site) (BS 3148, 1980). The results from prisms made with clean tap water and those made from the site water were then compared. The results are also shown in table 2.

For the avoidance of doubt, an additional test was simultaneously done on both the OPC and water. In this tensile strength test, nine small prisms of dimensions 175 x 25 x 6 mm were cast (Rigassi, 1995). The sand-cement mortar prisms were cast using the ration of 1 : 3 (cement : sand) and water cement ratio of 0.49 as before. Some of the bars were wet cured for only 24 hours, while the rest were similarly cured for 28 days. For each test, three bars were tested for direct tensile strength by subjecting them to available loads of up to 100 g (24 hour cured prisms) and of up to 500 g (28 day cured prisms). To conduct the test, a simply supported prism bar of the material was loaded at the end span. The load at which the bar snapped was noted. The results are all shown in table 2.

S/N	TEST	SAMPLE AGE (days)	UNITS	COMPARISON OF RESULTS	
				Obtained Value	Recommended standard Class 32.5N OPC
	A: Cement	28	MPa	27.4	32.5
1	Mean wet compressive strength (50 x 50 x 50 prism): clean water				
2	Mean tensile load (175 x 25 x 6 mm prism) "	28	g	350	500
3	Mean tensile load (175 x 25 x 6 mm prism) "	1	g	75	100
	B: Water	28	MPa	21.2	24.76
4	Mean wet compressive strength (Same prism but site water used)				
5	Mean tensile load (Same prism but site water used)	28	g	245	500
6	Mean tensile load (same prism but site water used)	1	g	55	100

Table 2: Results of site quality checks on OPC and water (Uganda, March 2000)

OPC quality test results

The results in table 2 show that the mean wet compressive strength value of the site cement-sand prisms was 27.4 MPa. This is less than the 32.5 MPa value specified as the minimum for the same grade of OPC (class 32.5N OPC, or equivalent). The difference was even higher (15%) than the allowable difference in strength of 10% due to errors. Some variation was expected but the result obtained was rather surprising. Similar trends were shown in the results from the prisms tested for tensile load. The values obtained were between about 25% and 30% lower than the recommended load values at 1 day and 28 days respectively. Since the bags in which the OPC found on site were examined (to conform to BS 12, 1990), the only conclusion that can be reached at this stage is that the contents could have been adulterated. Recent press reports from the country confirm the widespread contamination of OPC with clay before the bags are resealed (New Vision Newspaper, June 2001).

In the experience of the author as a practising civil engineer, incidences of this nature were not uncommon even on large concrete production sites. The problem is therefore a long standing one, and is more widespread than was originally thought. As can be expected, use of low quality OPC is likely to adversely affect the properties and performance of CSBs. Due to the above findings, the author decided to find out more about use of OPC in the country. From other site visits and interactions with users, stakeholders, suppliers, contractors and consultants it was established that OPC related problems were varied. These ranged from supply problems, unsuitability, incomplete hydration and misuse.

Water quality test results

The results of water quality tests show that the sand-cement mortar prisms cast using the dirty site water were of low strength. The wet compressive strength value of 21.2 MPa was 23% lower than the equivalent value for the cube cast using clean tap water (27.4 MPa). The allowable difference should not have been more than 10%. Tensile load tests also showed that prisms cast using the site water were about 43% lower in tensile strength than similar cubes cast using clean water. As stated earlier, water quality checks had not been planned for before (had it not been for the unusual appearance of the water being used on site). These results confirm that the quality of CSBs can be compromised when water of unknown quality is used for the hydration of OPC. The quality of water being used should therefore not be taken for granted. Water is scarce in most parts of the world. Even where available, clean piped water is inaccessible to most. Use of unsuitable water for hydrating OPC can therefore not be ruled out.

4.5 FINDINGS FROM QUESTIONNAIRES AND INTERVIEWS

As part of the research, a direct survey of the personal experiences of various stakeholders with respect to the production and use of CSBs was conducted in Uganda (January-March, 2000). In this section, highlights of the methodology used and results of the main findings are presented.

Methodology

At the start of the survey, two separate contact methods were planned, namely: interviews and questionnaires. It was later decided that a combination of the two methods into one would be more cost-effective and time saving. Face-to-face interviews using pre-structured questions facilitated the process making it more systematic and relevant to the situation on the ground. Moreover in a country where telephone, postal and communication systems were all undergoing major rehabilitation, there was no alternative to direct contact with respondents. By combining the contact method, problems associated with illiteracy, need for reminders, clarifications etc., were overcome. Respondents were also able to make suggestions and to raise other simpler or more complex questions associated with the production and use of CSBs in the country.

A sample size of 35 respondents from all walks of life was used. This was considered large enough for the purposes for which the survey was intended. The respondents were chosen at random from amongst the stakeholders: users of CSBs, government officials, private contractors and consultants, potential clients and funding agency representatives. It was assumed that the contacted respondents represented an unbiased sample of the population. Other highlights of the procedure were as follows:

- Interviews were conducted at various locations. These included dwellings where CSBs had been used, work locations, block production sites, on-going CSB building sites, and on substantially completed building sites.
- All respondents were assured of future confidentiality before the start of each interview. This was done to obtain their consent and ensure that their views would be freely expressed. In this way respondents answered questions put to them while at ease, and freely shared their experiences with the author. It was made clear to all of them that the results would be used purely for research purposes only.
- Each interview took approximately 20 to 30 minutes from start to finish. With the exception of interviews conducted on block production sites and on building sites where the respondents wanted to show the author further items,

the above time frame was maintained throughout the survey process.

- The response rate was 100%. All stakeholders directly contacted as above were willing to participate freely. This led the author to conclude that the survey method adopted was the right one under the prevailing circumstances in the country. Helpful inferences could then be made from the experiences of the respondents.

Findings from the Survey

The results of the findings which were manually tallied, coded and categorised relate to the type of questions put to the respondents, namely:

(a) : current walling material of preference

(b) : reasons for making the particular materials choice

(c) : preferred block types

(d) : common defect types encountered

(e) : preferred method of protection for blocks

(f) : suggestions on ways to improve CSBs.

The results of the findings are shown in the form of pie charts in figure 5.

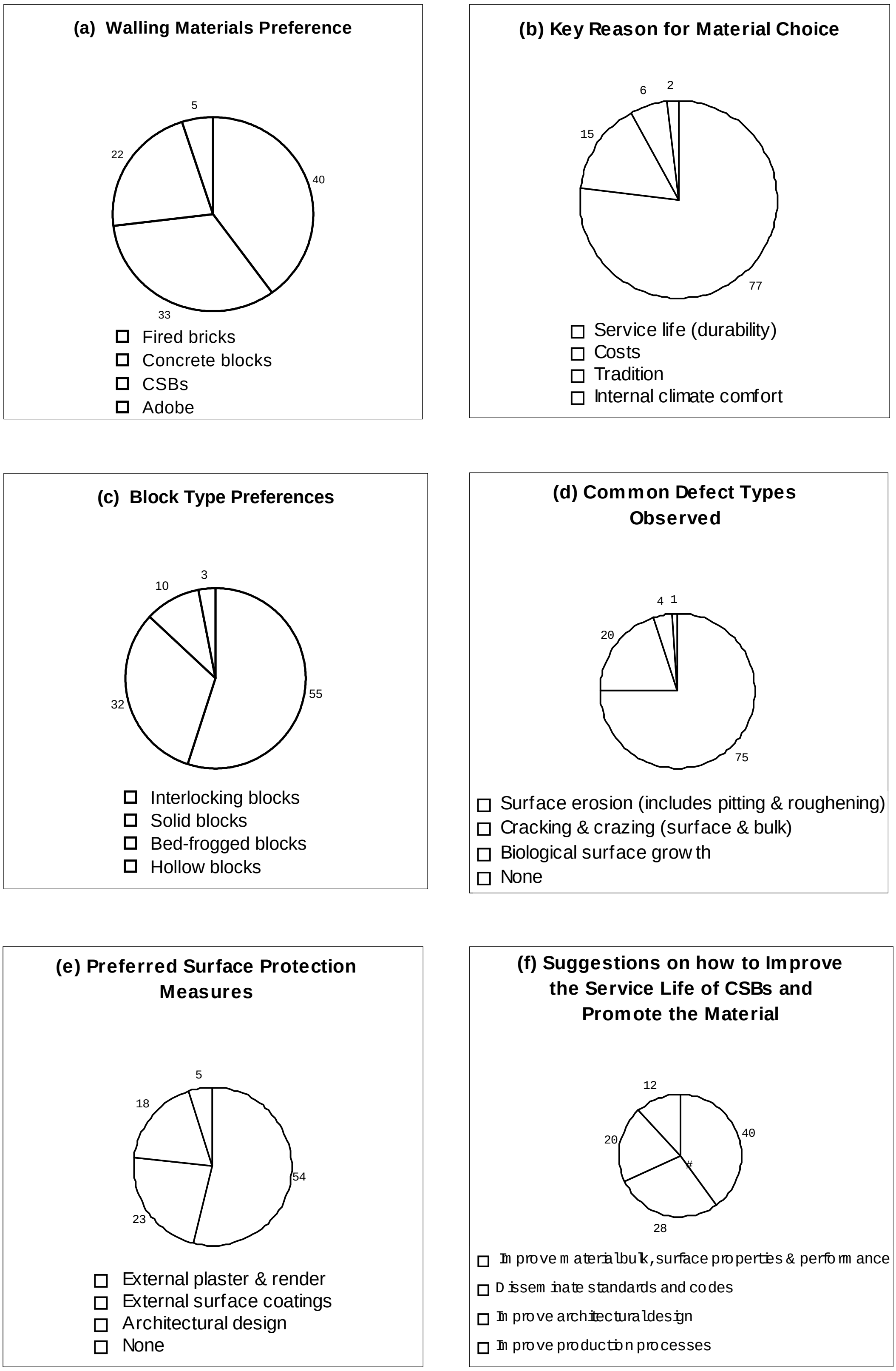


Figure 5: Results of findings from interview and questionnaire surveys conducted in Uganda (January – March, 2000)

Each of the outcomes shown in figure 5 (a) to (f) are now discussed in turn.

Preferences of walling materials to use were almost equally divided between fired bricks, concrete blocks and CSBs. However, fired bricks remain the material of first choice for most of the respondents (40%). This is followed by concrete blocks (33%). These two materials have been in use for generations and most respondents still have a high regard for them. The results for CSBs are encouraging. Having been introduced in the country only as recently as 1987, the fact that up to 22% of the respondents preferred the material over the otherwise cost free adobes (5%) represents a major sign of approval. The substantially better performance of CSBs as compared to adobes appears to account for the immediate popularity of the former over the latter (Games, 1981).

Key reasons for materials choice reveals the main basis for the selection of a particular walling unit. The key reason given by up to 77% of the respondents for choosing a particular walling unit is its in-service record (durability of the material). Only 23% of the respondents considered issues such as costs, tradition and internal climate as being more important. At the time of the survey, fired bricks were the most highly regarded walling units mainly due to their long service records requiring minimum or no maintenance. In future however, as fuel resources for firing bricks get depleted, the use of CSBs is likely to overtake that of fired bricks. According to the respondents interviewed, CSBs have gained prominence in a relatively short time because the material does not require to be fired, or burnt. Moreover, since the soil can be obtained on or near the site, where processing of the block is taking place, transportation costs are significantly reduced.

Block type preferences by respondents reveal a significant shift from solid blocks (33%) to interlocking blocks (55%). While solid blocks have been used since the introduction of CSBs in the country, interlocking blocks have only recently appeared on the scene (late 1990s). The dry-stacked-interlocking blocks have gained prominence over other types of blocks mainly because the use of mortar is not required. Other advantages mentioned are that walls from interlocking blocks are quick to build and are easy to align. Moreover, even if plaster is to be applied, the straight walls achieved require much less render than the traditional mortar bedded blocks (Van Den Branden & Hartsell, 1971). The production of interlocking blocks however, requires very sophisticated presses. At the time of the survey, these blocks were being produced by communally hired and donor subsidised motorised presses (Hydraform press, M 5 Mark I & II from South Africa). The quality of the green blocks appeared to be very high indeed (in terms of surface appearance, parallelism, edge straightness, etc.).

The *most common defects types* observed by respondents were surface erosion (75%) and cracking and crazing (20%). According to the respondents, not only were these two defect types common, but they were also clearly visible and discernible even by the casual observer. The older the building the more visible the defects became. No similar symptoms were observed in walls made from fired bricks and concrete blocks. It was therefore not surprising that 75% of the respondents had detected premature deterioration on unprotected CSB walling in the form of surface erosion (including pitting and roughening). Only 4% of the respondents had observed other, less common defect types like the peeling off of render, plant and surface growth, insect boring, etc. The majority of the respondents interviewed did not find any difficulty in linking some of the major defect types to seasonal variations. The author was informed that it was generally observed that most of the surface erosion in CSB walls occurred during the two main rainy seasons in the country (March-May and

September-November). The symptoms were reportedly similar to those that occurred on adobe walls at the same seasons of the year. The only difference was in the degree of severity. Cracking of exposed CSB walls was also a phenomena more noticeable during the long-dry seasons than during the rainy seasons. A casual link between environmental action and deterioration in blocks was therefore being strongly suggested. The author could not see any reason to disagree with the general hypothesis. When asked why they thought surface deterioration of CSBs occurred too prematurely, most of the respondents were of the view that use of low amounts of cement (4-6%) might be responsible. The reasoning was that due to the low stabiliser levels, poorly bonded soil particles at the surface of the block could easily be dislodged by the mechanical energy from rainfall impact (Laws, 1941, Herbert, 1974). The premature appearance of such defects when no similar defects could be observed in like materials of the same age serving under similar conditions was seen by many as a major cause for concern. It partly explains why fired bricks and concrete blocks, despite their higher costs, still remain preferable to most of the respondents.

Preferred surface protection measures for exterior wall faces were varied. Up to 95% of the respondents considered one form of exterior wall surface protection method or another. The most preferred option was external surface render (54%), followed by surface coating (23%) and architectural design (low roof overhang) (18%). Only 5% of the respondents were not bothered and preferred to let events take their course. Part of this latter group thought external render was expensive since it involved remedial surface repairs being done first on the walls before application of the render proper could follow. Some even considered replacing defective CSBs with equivalent sized fired bricks. Where surface render was used, incidences of plaster peeling off from the blocks were also observed. The reason for this could be attributed to inadequate curing, improper workmanship during rendering, poor choice of soils (too much clay), or even poor choice of stabilisers (high clay content, lime is preferable; low clay content, OPC is preferable). Most respondents preferred to use higher amounts of OPC to improve the overall quality and durability of the block than to have to use lower amounts and still plaster the wall in addition. They thought the former option would work out to be cheaper than the latter in terms of overall costs. The findings appear to support strategies based on enrichment of block surface layers to offer additional resistance to deleterious environmental actions. Use of enriched surface layered material is investigated experimentally in Chapters 6 and 7 of this thesis.

Proposals on *quality and durability improvement* strategies for CSBs came in many forms. The strategy that emerged as the most prominent was the improvement in the overall bulk and surface properties and performance of CSBs (40%). This was followed by the need to disseminate and comply with written standards and codes of practice on the production and use of CSBs (28%). Improved architectural design of CSB buildings and improved block production processing methods were considered by 20% and 12% respectively of the respondents as being the best ways of protecting blocks and achieving higher quality. As stated earlier in the thesis, it is the considered opinion of the author, now confirmed by these findings, that it is the durability of the block, rather than any other consideration, which will ensure its widespread demand and use in developing countries. Current research into the durability of the material therefore appears to be timely. Even where research findings may lead to the production of improved blocks, codes of practice and standards are still needed to ensure that compliance with minimum standards and better methods of work are upheld. At the time of the survey, no approved CSB standards were available in the country yet over 400 CSB structures had been built. And more CSB buildings are being planned for (DoH, 1992).

4.6 CONCLUSION

From the results and findings discussed in this Chapter, the following conclusions can be made.

CSBs are likely to remain in high demand in developing countries such as Uganda where the housing backlog is still very high. The increased use of CSBs for walling in high density, low income urban areas appears to represent the best way forward in redressing the imbalance. The current approach using community hired or centrally used motorised block presses also appears to be the best practical housing delivery method.

In humid tropical areas, rainfall and temperature variations can adversely affect the performance of a block exposed to the elements. These variations can also catalyse chemical reactions between the constituent materials forming the block. More research is needed to understand the mechanisms involved, and to explain how individual rainfall parameters such as drop size, drop size distribution, fall velocity and impact kinetic energy, etc., affect the rate of surface deterioration in blocks.

Visual inspection of a sample population of officially documented CSB structures revealed that in the absence of protective render, premature deterioration can take place. The most common defects observed included: surface erosion, surface roughening, surface pitting, surface cracking, surface crazing, bulk cracking, chipped edges and corners, and loose material residuals. Since weathering conditions were genuine and since the blocks were inspected at full scale, a direct link can be said to exist between the symptoms observed and the exposure conditions. Moreover, as a fairly sufficient number of CSB structures were inspected (more than 10% of the total number), this conclusion is likely to be fairly reliable (cause and effect link).

The amount of loose material lost from the original mass of a block was estimated by directly measuring the recessed volume of the material. The loss in volume from unroofed and unrendered blocks over a period of 12 years was about 38%. Losses were higher on the east-west facades than on the north-south facades. Lower courses of walls also experienced more losses than the middle and upper courses of the same wall (about 8-15% more). The increased amount of rainwater and splash experienced by the lower courses, and the increased amount of solar radiation absorbed by east-west facades appear to be responsible for the differences. Surface protection measures are strongly recommended for blocks that are to be used under similar conditions.

It was also found that crack patterns and dimensions followed the above trends. The widest cracks recorded (2.9 mm) were found on the east-west facades of CSB walls. These cracks are much wider than the normal permissible crack widths in concrete structures. Cracking is undesirable as it makes the block vulnerable to ingress of moisture. The crack patterns observed indicate that drying shrinkage, expanded product formation, thermal expansion and contraction and improper curing can all lead to disruption in bonding. More research is still required to explain the mechanism involved in each of these phenomena.

It was found that field indicator soil testing was a valuable tool for early identification and selection of soils for CSB production. Although at the time of the fieldwork no clear order for conducting the several available tests existed, a more planned approach is recommended. Since the results of the field indicator tests showed considerable convergence with documented laboratory test records for the soils in the same location, the former are recommended as the first step in determining the suitability of a soil for CSB production. For large CSB production sites, laboratory tests should be extended to analyse the chemical composition of the soil to be used. This should be done even after the soils have passed all basic suitability tests.

Findings from the observation of site practice at block production sites visited confirmed that the level of process management can indeed influence the quality of the block produced. Shortcomings were observed right through the production process from soil extraction and preparation to curing of green blocks. Most of the shortcomings could be corrected by better supervision and proper guidance. A balance of emphasis is therefore required between design quality of blocks and their actual quality.

Impromptu quality checks conducted on OPC and water used for production of CSBs confirmed that significant differences can exist between the required minimum standards for each material and the values obtained on site. The wet compressive strength of a sand-cement mortar cube tested at 28 days using the OPC found on site was about 15% lower than the minimum recommended value for the same brand of cement. Tensile force tests also showed similar trends. Similar cubes made and tested in exactly the same way as before but this time using the site water of unknown quality was found to be about 23% lower, well outside the allowable variation. These two findings further confirm that CSBs are likely to be adversely affected not only by variations in the processing methods or exposure to environmental agents, but also by the quality of each of the constituents used in producing them. It is therefore strongly recommended that regular quality checks, inspections, tests and certification be introduced at all key stages in the block production process.

Surveys conducted using interviews and questionnaires revealed a number of wide ranging issues. It was found that a good service record (durability) of a material would ensure its widespread use (as underlined by 77% of the respondents contacted). It was noted that interlocking blocks that do not require use of mortar were the most highly demanded block type. These blocks were considered to require less time and money to use for building than comparable solid, hollow and frog-bedded blocks. It was established that most respondents were quite familiar with causes of premature defects in CSBs, citing surface erosion and cracking as the two most common defect types. To improve the service life of CSBs, various surface protection measures were regarded as the most economic way of achieving the goal (by 90% of the respondents). Other approaches considered included improved intergranular strength (40%), dissemination of standards and codes (28%), better architectural design (20%) and better processing methods.

With the preceding conclusions, the objectives of Chapter 4 were met.

CHAPTER 5

EXPERIMENTAL DESIGN AND PREPARATION OF SAMPLES

5.1 INTRODUCTION

In this Chapter, experimental design for the main laboratory based tests are described. The preparation of CSB specimen samples for further tests are also described. Laboratory based experiments were planned for in the research methodology mainly to test ideas, theories and designs that had been formulated. The scope of this Chapter is limited to the description of the experimental design adopted and the methods used to produce CSB specimen samples. Surface and bulk property tests for which the specimens are fabricated are discussed separately in Chapters 6 and 7.

The objectives for which CSB specimen samples were produced were:

- to obtain a sufficient number of CSB samples from which statistical generalisations can be made.
- to obtain quantitative experimental results from samples with various input variables (response experiment).
- to monitor the effects of the main input variables in CSB production.
- to compare the experimental data obtained with theoretical predictions and with other available data on CSBs.
- to facilitate the explanation of discrepancies between predicted and measured performance.

The laboratory tests had to be completed within a limited period of time. For this reason, the samples produced as described in this Chapter were meant to satisfy only a limited number of tests. For all laboratory tests attempts were made to ensure that the results obtained satisfied three basic conditions: accuracy, reliability and reproducibility. Only standard methods were used in the production of CSB samples. This chapter only describes the measurements made on green blocks and just cured blocks. Measurement on block samples were limited to: dimensions, weight, shape and appearance. The specimens were then marked and labelled for further extensive tests (reported in Chapters 6 and 7).

The rest of this Chapter is presented in four sections, namely: experimental design, results of soil classification tests, preparation of CSB specimen samples, and conclusion.

5.2 EXPERIMENTAL DESIGN

Variation of any of the several production input variables can influence the quality and performance of blocks (Chapter 3). These variables include:

- Soil (type and proportions of main fractions)
- Stabiliser (type and content)
- Mix-water (amount)
- Compaction pressure
- Curing conditions

For any meaningful experiment, it is unhelpful to vary all the input variables at the same time. The experimental design was therefore based on fixing some of the variables while varying others. The control (independent) variables were taken as the composition variables (soil type, stabiliser, water) and process variables (compaction pressure, curing conditions). The main variable fixed was the soil type. All block samples were made using soil of a fixed composition. In this way the effect of varying the stabiliser type and content, compaction pressure, mix-water content and curing conditions on the properties (response) of the block could then be easily monitored. It was also considered necessary to specify the number of observations, the values of the control variables at every observation and the order of observations (Ray, 1992; Greenfield et al, 1996).

The main approach adopted was to compare the properties and performance of two categories of blocks, namely: traditional blocks (TDB) and improved blocks (IPD). While the former were made in the conventional way using OPC and/or lime as the stabiliser, the latter were made using partial replacement of OPC with condensed silica fume (microsilica). The amount used was fixed at 10% of the OPC content (Neville, 1995). The blocks were regarded as being improved because of the theoretical expectation of enhanced performance due to the inclusion of microsilica (Chapter 3).

The stabiliser type and content, rather than any other variable, was used as the main categorisation parameter for several reasons. To start with, it is the stabiliser content which is responsible for most of the improvement in CSB strength, dimensional stability and durability (Spence & Cook, 1983). Compaction pressure could have also been used as the main parameter for categorising block types. Unfortunately, although compaction pressure contributes towards reducing voids and thereby increasing density in blocks, its effects can be easily reversed in the absence of a stabiliser (Chapter 3). It is the stabiliser content alone which is responsible for binding the block particles together on a more permanent basis. It was reported in Chapter 3 that densification alone without the addition of a chemical stabiliser has no permanent effect on soils. However, the effect of varying the compaction pressure, mix water content and curing conditions were also investigated for both improved and traditional blocks. The summary of the actual input variables used in the design of the experimental samples are shown in table 3.

S/N	INPUT VARIABLE	UNITS	AMOUNT	EXPERIMENTAL DESIGN	
				FIXED	VARIED
A	SOIL 'S' (Laboratory soil)			•	
	Gravel	%	2	•	
	Sand	%	75	•	
	Silt	%	8	•	
	Clay	%	15	•	
B	STABILISER				
	OPC	%	3,5,7,9,11		•
	Lime	%	5	•	
	Microsilica	%	10 (of OPC)		•
C	MIX-WATER				
	Highest	%	9.0		•
	Medium	%	8.5		•
	Lowest	%	7.0		•
D	COMPACTION PRESSURE				
	High	MPa	10		•
	Medium/Normal	MPa	6		•
E	CURING				
	Time	Days	28, 56	•	
	Humidity	%	0, 100		•
	Temperature	°C	22-24	•	

Table 3: Summary list of the main constituent materials and input variables used in the production of block specimens.

Each of the variables listed in table 3 are discussed in turn.

The *soil type* was kept fixed, with approximate composition: gravel (2%), sand (75%), silt (8%) and clay (15%). As several block types of nominal dimensions 290 x 140 x 100 mm were required for the experiments, keeping the soil type the same for all specimens would help increase reliability in the tests. It was from the full block sizes that smaller specimen samples were obtained for further experimentation. By keeping the soil type the same at all times for all specimens, better consistency, repeatability and controllability could be achieved. The selected soil composition had to comply with the suitability criteria earlier discussed for soils for CSB production. An optimum composition of soil fractions for more effective stabilisation with OPC rather than lime was chosen. The criteria used for soil classification was particle size distribution. According to literature sources, an ideal soil for effective stabilisation with OPC has the following composition: coarse fraction (gravel and sand) 75% and fines fraction (silt and clay) 25% (Fitzmaurice 1958; United Nations, 1964; Houben & Guillaud, 1994). Following from this, an artificial soil was blended in the laboratory for repeatable use.

The mock soil was made by controlled mixing of ordinary buildings sand (OBS) and ordinary potters clay. The soil was from then on referred to as soil 'S'. The clay type was of the Kaolinite group, chosen due to its known stability and non-expansive nature when in contact with water (Scot, 1963; ILO, 1987; Webb, 1988). The importation of representative soils from the humid tropics was considered not to be necessary. Even if this had been done, not much would have been achieved. This is because soil remains a highly variable material even within each country and moreover, even within regions of the same country. A further advantage in using the artificial laboratory blended soil was that soil properties such as particle size distribution, plasticity, bulk density, moisture content, etc., could all be easily controlled. These soil properties could be kept consistent for all block samples. Any variations in soil properties would not only influence the choice of stabiliser, but also the properties of the blocks produced from it. By keeping the soil type consistent, a more logical interpretation of the effects of other production variables on the performance of the block could be achieved. Further, any variations detected in the performance of blocks could be linked to the method of investigation used instead of attributing it to variations in soil composition. For the soil 'S', key soil properties such as particle size distribution, linear shrinkage, moisture content, etc., were tested using standard test methods. Test methods are described in Webb (1988) and in Webb and Lockwood (1987). The key test results are presented and discussed in Section 5.3 of this Chapter.

The *stabiliser type and amounts* were varied as discussed earlier (table 3). The predominant stabiliser type used was OPC of class 42.5N, supplied from Rugby Cement (BS 12, 1996). The other stabilisers used in combination with OPC were lime (BS 890: 1995) and condensed silica fume (Illston, 1994; Neville, 1995). OPC was selected as the main stabiliser for a number of reasons (Chapter 3). Of all the common stabilisers OPC is widely available in most parts of the world. Lime was also used in combination with OPC for a limited number of specimens. The objective of such combinations was to evaluate the effect of lime on the clay fraction of the soil (Hilt & Davidson, 1960). The lime type used belonged to the Limbux brand, a high quality hydrated lime of typical assay 96.5% calcium hydroxide. The neutralising value was 7.4% CaO. Each required amount of lime was accurately weighed in self-sealing bags on an electronic scale. Microsilica (non-combustible amorphous S_1O_2: CA_5 No. 69012-64-2) in controlled amounts of 10% of the OPC content, was added to a selected number of blocks. The microsilica used was grade 940-4 (Elkem microsilica from Norway). The objective was to assess the effect of such partial cement replacement materials on the improvement of strength and quality in blocks. The material was known to have been employed in the production of high-strength concrete (Neville, 1995). By progressively altering the stabiliser content and type, variations in the performance of blocks produced under each category were monitored. The extent and significance of changes in properties were of great interest to the research. The amounts of OPC used is shown in table 3. OPC content varied from 3% to 11% by weight in increments of 2%.

Theoretically, when OPC is partially replaced with a CRM such as microsilica, the latter acts as a nucleic centre, thus reducing the water-cement ratio (Chapter 3). With proper wet curing, a maximum degree of hydration can also be achieved. Moreover, the microsilica also reacts with the lime that is released during the hydration of OPC to create a secondary binder in the block. Under such circumstances, it can be expected that such a block would have a much higher inter-granular strength, higher density and more resistance to surface abrasion. It is for this reason that such blocks have been referred to here as 'improved blocks' (IPD).

Mix-water content was varied (from 7% to 9%) for a select number of blocks (those made with 5% OPC content). For all other blocks, the mix water content was maintained at 8.5% by weight of the soil plus stabiliser mix. Variation of mix-water content was not originally planned. After accidentally adding more water than was originally intended and obtaining a much higher value of wet compressive strength for the block, it was decided that the variable be investigated further experimentally. The effect of changing the mix-water type was also investigated: ordinary laboratory tap water and distilled water. The results from three samples showed that there was no significant difference in the performance of blocks made from either type of water. Consequently, Coventry tap water was used to produce all block specimen samples used in the experiments. The water temperature was approximately 23°C.

Compaction pressure was maintained at 6 MPa, but only varied to 10 MPa for a select number of blocks. The latter was used purely for comparison purposes only since such high values are rarely used in practice. It is common to compact CSBs at compaction pressures between 4 MPa and 8 MPa (Houben & Guillaud, 1994). The BREPAK press that was used to make all the blocks was equipped with a pressure monitoring gauge (Webb & Lockwood, 1987).

Curing conditions were maintained for all blocks according to the specifications for the binder type used. For a select number of blocks, curing conditions were varied by curing the blocks under exposed and wet conditions throughout (immersion after 24 hours of demoulding). Otherwise normal curing conditions were applied to the majority of blocks produced. Primary curing taking 3-7 days, followed by secondary curing for 28 days was the general format adopted. Where lime was included in a block, the above periods were doubled. After curing, the blocks were then cut to the required sizes (Section 5.4).

5.3 CHARACTERISATION OF SOIL 'S'

Soil classification tests were performed on soil 'S' in order to confirm its category amongst other soils. The main tests conducted included the following:

- Particle size distribution test (Vickers, 1983; BS 1377: Parts 1 and 2, 1990)
- Sedimentation test (Appendix J)
- Linear shrinkage test (Appendix K)
- Moisture content test (BS 1377: Parts 1 and 2, 1990)

The procedures involved in each of the above tests are described in the references shown while some are discussed in Appendices J and K. The soil tests were conducted before and after the manufacture of the several specimens. These showed that no significant changes in soil composition had occurred during the entire testing period. Summary of the average values obtained in the above tests are presented in table 4.

S/N	TEST	UNITS	TEST RESULTS	RECOMMENDED VALUES
1	PARTICLE SIZE DISTRIBUTION			
	Gravel	%	1.3	< 40
	Sand	%	75.4	25-80
	Silt	%	8.1	10-25
	Clay	%	15.2	8-30
2	SEDIMENTATION (JAR)			
	Gravel and sand	%	73.9	75
	Silt and clay	%	26.1	25
3	LINEAR SHRINKAGE	mm	17.6	15-30
4	MOISTURE CONTENT	%	0.9	<3
	SOIL TYPE		SANDY SOIL	

Table 4: Summary of soil classification test results for soil 'S'. (Recommended values: ILO, 1987; Houben & Guillaud, 1994; Rigassi, 1995)

The *particle size distribution* test results for soil 'S' show that the soil type is predominantly sandy (Appendix L). The proportions of the main soil fractions present fall within the recommended ranges. Soil 'S' was therefore found to be suitable for stabilisation with OPC. The soil has sufficient proportions of coarse fraction (fine gravel and sand) for the skeletal frame and body of the block, as well as an adequate proportion of fines (silt and clay). The test method used is fully described in Vickers (1983) and in BS 1377: Part 2 (1990).

The *sedimentation (jar)* test results also confirm the presence of sufficient quantities of coarse soil fraction and fines. This result also shows convergence with the previous test. According to the results, the amount of coarse soil fraction was about 73.9% and fines fraction about 26.1%. As explained in the earlier parts of the thesis, the main advantage of having a sufficient amount of fines is to make sure that the block remains intact on demoulding. On ejecting a block, the hydration reaction of OPC is still at a very early stage and the cement will require more time before it begins to set and harden. The presence of a natural binder like clay in the block is therefore advantageous. The sedimentation test is however, quite slow (over 48 hours) and of medium accuracy. The values of silt and clay can be slightly distorted due to swelling and expansion in water. It was also found difficult to differentiate the silt from the clay as both appeared to be well intertwined. The test method is described in Appendix J.

The *linear shrinkage test* (LST) result of 17.6 mm (mean value) confirmed that soil 'S' had just enough clay in its composition (Webb & Lockwood, 1997). There is therefore no need to add more clay than the amount already added (15%). The results also confirm that the use of OPC, rather than lime for stabilisation would be more effective in this case. Lime would have been required if the shrinkage value had been higher, signifying a high clay content in the soil. The LST method is described in Appendix K.

The *moisture content* value of 0.9% shows that soil 'S' is in a near dry state. The term 'dry' as used here might not be strictly accurate since there is still some water present in the soil in the form of adsorbed water which surrounds the solid soil particles (Chapter 3). The term 'dry' has been used here to indicate that soil 'S' attained constant weight on being heated to 105°C - 110°C (BS 1377: Part 2: 1990). The dry state of soil 'S' is quite important since mixing of the soil with the stabiliser has to be done with both materials in a similar state. If the soil had been wet, then its specific surface area would have varied unnecessarily. The size and density of blocks obtained would not have remained consistent (Chapter 3). By determining the dry state moisture content, it was also then possible to determine the total amount of water added to achieve the optimum moisture content of the soil. The test for the optimum moisture content in this case is done using the drop test. Soils are compacted at the optimum moisture content because it is difficult to compact them at lower moisture contents. An increase in moisture content lubricates the soil, making it more workable. Dry density increases and air voids are reduced. The optimum moisture content of the soil is however, not a parameter dependent on the soil type alone. It also depends on the type of grading and on the compaction effort used (ILO, 1987). The test method used to obtain the moisture content value for soil 'S' is based on BS 1377: Part 2: 1990.

5.4 PREPARATION OF CSB SPECIMENS

In this Section, the design and production of CSB specimen samples used for subsequent tests and experiments are described. The summary list of the total number of samples made during the experimental stages is also provided. The sample size for each test was based on earlier exploratory tests where the coefficient of variability for each test type was determined (5.4.2).

5.4.1 LABORATORY PRODUCTION OF CSBs

The planned experiments demanded a large number of specimens prepared to a high degree of accuracy, reliability and consistency. Extra care had to be taken at all stages of the block production process: soil preparation, mixing, compression, and curing of the samples. After curing, the block specimens were cut to conform with the sizes required for each test. Apart from the mix-proportioning stage that distinguished the block types by amount and type of stabiliser used (improved and traditional blocks), the rest of the procedures remained the same. Specimen design and preparation describes the procedures adopted and the precautions taken to produce the required number of block specimens for the various tests planned. The description is based on the four main stages of CSB production:

- Soil preparation
- Mixing
- Moulding
- Curing (and sizing)

Soil preparation involved the mixing and storing of soil 'S'. Before this was done however, the ordinary builders sand (OBS) was dried and screened prior to mixing with clay. The sand was supplied in 500 kg bags and placed in bins outside the laboratory. The sand had been supplied clean, i.e. after washing out the clay fraction from the sand. To dry out the material, the sand was removed from the yard bins and spread out on the hard, flat concrete laboratory floor. About 100 kg of the OBS was weighed and spread out each time. The weighing was done using the Avery Weighing Scale: type 3202/CLE No. B672521 (capable of weighing up to 50 kg at a time). The objective of drying out the sand was to ensure that a material of an almost even moisture content was obtained. The spread out sand was regularly and repeatedly raked to turn it over every four hours for about three days. When both the bottom and top layers achieved uniform light colouration, the sand was considered to be dry enough. The dry sand was then screened by pouring portions of it at a time onto a circular framed screen placed tightly over a laboratory soil storage bin. The square sieve aperture used was 5 mm (BS 410) to allow only fine gravels and sand to pass through (sieves made by Endcotts Test Sieves Limited). In this way all medium to coarse gravel present in the supplied sand was eliminated. Even then, it was still found necessary to use the hand to occasionally remove soil fraction sizes larger than 5 mm that may have accidentally gone through. The screened material was then stored in sealed bins within dry areas of the laboratory.

Mixing was then done to improve the grading of the sand. Controlled mixing was done by adding about 15% by weight pure grade E Kaolin clay. The characteristics of the clay used were: ECC International Grade E potters pure clay (quality China clay made in England); specific gravity 2.6; specific surface area 8.0 m^2/g; water soluble salts 0.15%; silica (S_iO_2) 50%; alumina (Al_2O_3) 35%; and pH 5± 0.5. The Kaolin clay was supplied in 25 kg bags. The OBS and clay were mixed mechanically using the Hobart machine mixer. Mixing was done for each batch of about 30 kg for about 4 to five minutes till a uniform colouration was achieved each time. After a homogeneous mix was obtained, soil classification tests were performed for every other five batches (Section 5.3). Soil 'S' was then stored in laboratory bins, covered and sealed. Covering of soil 'S' was done to minimise risks of contamination and to ensure that the moisture content remained uniform throughout. This procedure was repeated until enough soil to make about 60 blocks of nominal dimensions 290 x 140 x 100 mm was obtained. Each block required about 8.0 kg of soil 'S'. The amount of OBS and clay supplied was sufficient for the required number of experimental samples.

Mixing of soil 'S' with stabilisers (OPC, lime and microsilica) and water, was done in four stages for each batch. Proportions for the various stabilisers and soil 'S' used are shown in Appendix M. The key objective during the mixing stage was to ensure a good distribution of the stabiliser and water throughout the mix. Consistent proportioning out, dry mixing and wet mixing were required to obtain proper samples.

The proportioning out of soil and stabiliser was done by weight, not by volume. An electronic weighing scale capable of weighing up to 20 kg to an accuracy of 0.05 grams was used each time. All materials were weighed inside a plastic bag which was then sealed and clearly marked. The bags were carefully labelled to show the exact weight, type of material and date of weighing. By sealing the bags, variations of moisture content and contamination of the weighed out material were avoided. In all cases, dry mixing was done first before wet mixing with water. All mixing (wet and dry) was done in the Hobart Machine mixer as described earlier. Dry mixing was done for about three to four minutes. After this, water was then uniformly added to the dry soil and stabiliser mix and the process repeated. Amounts of water varied between 7.0% and 9.0%. The amount used was determined to give the soil its approximate optimum moisture content. The water was also meant to be sufficient for hydration of the stabiliser(s).

After uniform colouration was achieved, a consistency test was done for each mix (Chapter 3). Soil and stabiliser mixes which passed the drop test were immediately separated into three equal amounts sealed in polythene bags. Separation was necessary in order to ensure that mould filling could be done in three equal layers. Except where it was done deliberately, no delay between mixing and moulding was allowed.

Compression of the damp soil and stabiliser mix was done using the pre-installed BREPAK block making machine (SN BQ 038074, originating from Bristol, United Kingdom). The block making machine was designed on the quasi-static compression principle. The same machine was used for all block specimens produced. The main characteristics of the machine were: maximum nominal block size: 240 x 140 x 100 mm; maximum daily output, 300 blocks; maximum moulding pressure, 2 to 10 MPa.

Instructions contained in the operators manual for the machine were followed while making blocks (Webb & Lockwood, 1987). The compression procedures were done in three stages: mould filling, moulding, and demoulding. Mould filling was done after first cleaning the mould using release oils. This was repeated after every four to six blocks were made. Filling was done in three equal layers as described before, using the pre-weighed and separated mixes. By accurately weighing the mixes, it could then be expected that blocks of the same size and of consistent density could be produced. On placing each layer into the mould, the operation was checked by using fingers to press the mix into the corners of the mould. After the last layer was levelled, the mould cover was turned into position to cover the mix. The pressure monitoring gauge attached to the machine was used to determine the amount of force applied as required. The procedures were repeated till the required number of blocks were produced. Three blocks were produced for each specific mix type.

After the blocks were made, demoulding and handling followed, (done with great care as the blocks were still weak). Plywood sheets of about 20 mm thickness were used to remove the blocks from the elevated mould base plate. The sheets over which the green blocks were carried were each pre-weighed. The removal procedure was the same for all green blocks demoulded. While holding the pre-weighed plywood sheet level with the top of the elevated mould base plate, the green block was gently moved onto it using a second plywood sheet. The removed green block was then weighed together with the plywood sheet on which it was carried. Weighing was done using the electronic scale described earlier. External dimensions of the demoulded blocks were also taken. Dimensions were taken using a Mitutoyo shockproof dial calliper accurate to 0.05 mm. Measurements were taken at several locations on the block edges and mid-sections as specified in BS 6073: Parts 1 and 2, 1981 and BS 3921,

1985. The blocks were then carefully labelled using a soft-nib permanent marker. This was done to identify each block by date of manufacture, serial number, stabiliser content and moulding pressure used. The blocks were then covered with polythene sheets, which were also marked externally as before.

Curing of green blocks was done according to the specifications for each type of stabiliser used. A selected number of blocks were however cured under different conditions to evaluate the effect of varying this parameter on the properties of the block. For all other blocks, normal curing procedures were followed. Primary curing periods varying between three and seven days, followed by secondary curing periods lasting up to 28 days for OPC stabilised blocks, were maintained. Where lime was included in the mix, these periods were doubled. Secondary curing temperatures were maintained at the laboratory levels (22-24°C). After curing, the blocks were again marked to indicate the time and curing conditions followed for each block. The blocks were then cut down to smaller sizes as required for each test category (Section 5.4.2).

5.4.2 NUMBER OF SPECIMENS PRODUCED

A sufficient number of CSB specimens were required for all the planned laboratory experiments. Initial performance tests and accelerated tests for surface and bulk properties of blocks required specimens of different sizes. For these reasons, the CSBs that had been produced in full scale had to be cut to smaller dimensions.

Blocks were cut mechanically using a concrete lathe machine (masonry saw machine: Clipper, model EN 2-40-3). The lathe was driven electrically with a powered circular saw complete with a water sprinkler. Each block was accurately pre-demarcated with the required dimensions before the lathe was used to cut through. The machine was so effective that the cut surfaces were neat and straight. In this manner, blocks of nominal dimension 290 x 140 x 100 mm were cut down to the following major sizes: 100 x 100 x 100 mm (two per block); 100 x 100 x 40 mm (two per block); 100 x 100 x 90 mm (one per block); 100 x 90 x 40 mm (one per block). Even during the cutting, differences in the resistance of the block bulk to cutting could be felt. The blocks made using partial replacement of OPC by microsilica were the hardest to cut while blocks made with lime inclusion were the weakest to cut. Blocks compacted at 10 MPa were also harder to cut than blocks compacted at 6 MPa but consisting of the same stabiliser and soil mix.

For each test, three specimen samples made in exactly the same manner and composition were required (reasons explained later in this section). The total number of full size blocks made in this way were three per stabiliser and soil mix. The grand total number of blocks made was 51. From this grand total, over 306 smaller specimen samples of different dimensions were obtained. The specimens were then used for various bulk and surface property tests as described in Chapters 6 and 7. Appendix N shows the list of the various types of blocks produced as well as the different specimen samples obtained from them. Specimen samples for comparable materials such as concrete blocks (CBS), fired brick samples (FBS) and rock block samples (RBS) were obtained from the laboratory. These materials were used for the TWA and SDI tests only.

The reasons for testing three specimen samples for each test (then using the mean for interpretation) were based on the following considerations:

- as it is well known that all test results vary, preliminary tests using six specimen samples composed of 5% OPC and compressed at 6 MPa (cured for 28 days) showed the estimated variation from the mean in each case (properly tested) to be consistently low: WCS, 2.49 MPa (variance 0.027): BDD, 2127kg/m^3 (variance 0.023): TWA, 9.8% (variance 0.135) and SDI, 81.4% (variance 0.118). A 95% confidence interval was used in each case. There was no reason to expect that other mixes of differing OPC content would not show similar consistency and trends.

- previous findings by other researchers had arrived at the same conclusion (Webb, 1958; Fitzmaurice, 1958; Gooding, 1994)
- composition variables (soil type, stabiliser content, mix-water-content) and processing variables (moulding pressure, curing conditions) were determined using precision instruments and standard processing methods respectively. The specimens were therefore produced with a high degree of consistency. It is unlikely that the methods used in the laboratory can be repeated in field practice without major departures (Chapter 4).

Moreover, even if more specimens than the determined number of three for each case had been produced, other research constraints such as cost, time and space, had to be taken into account. Time constraints at planning, design and implementation showed that mandatory delays due to curing periods meant that the number of specimens required had to be limited. The time for the actual experimental work and for recording, computation and analysis of the results were also important considerations. Cost considerations relating to ordering of materials, delivery, wages, electricity, water, etc., were other important constraints. From the degree of accuracy, reliability and repeatability achieved in each case, it was later found that the decision made to use three specimens per test was justifiable.

5.5 CONCLUSION

From the preceding discussions in Chapter 5, a number of conclusions can be made regarding the following: experimental design, soil 'S' test results, CSB specimen production, and the total number of specimens provided.

The *experimental design* was based on identifying the main composition and processing variables involved in the production of CSBs: soil type, stabiliser type and content, mix-water content, compaction pressure, and curing conditions. In the experimental design for sample production, the soil type was fixed. Soil 'S' was composed of about 75% fine gravel and sand, and about 25% silt and clay. This was done to ensure that consistent use of the same soil would be possible throughout the testing period.

Stabiliser type and content were varied: cement content was varied from 3% in increments of 2% to 11%. Microsilica amount was fixed at 10% of the cement content. The amount of lime was fixed at 5% by weight of the soil when used in combination with cement. Blocks made using a mixture comprising microsilica and cement were designated as 'improved blocks'. Blocks where microsilica was not used were regarded as 'traditional blocks'. The categorisation was based on the stabiliser type because it is this variable that remains the single most influential factor that can affect the performance of blocks.

The mix-water content, compaction pressure and curing conditions were maintained at similar levels for the majority of blocks produced. For a select few numbers of blocks, these parameters were varied. The majority of blocks were made with a mix-water content of 8.5%, while the select few referred to were made using 7.0% and 9.0% by weight respectively. Blocks made using the 9.0% mix-water content were later found to perform better than those made using 8.5%. Compaction pressure was fixed at 6 MPa for most blocks. A few blocks were produced using a compaction pressure of 10 MPa mainly for comparison purposes only. Curing time and conditions were maintained at the specified levels required for each stabiliser type. All wet curing (100% humidity) was done for a select number of blocks to evaluate the effects of such conditions on the performance of blocks. The effects of these variables on the bulk and surface properties and performance of blocks are discussed in Chapters 6 and 7 respectively.

The *artificial experimental soil* blended in the laboratory (soil 'S') was found to meet critical requirements for suitability for stabilisation with OPC. The mean linear shrinkage value of 17.6 mm was within the range (15-30 mm) indicating the presence of a sufficient amount of clay. If the shrinkage value had been less than 15 mm, then the soil would have been regarded as having an insufficient amount of clay in it. The glass jar sedimentation test results confirmed that the coarse soil fraction (fine gravel and sand) was about 73.9%, while the fines fraction (silt and clay) was about 26.1%. Both values are within the recommended ranges for soils suitable for stabilisation with OPC. The laboratory dry moisture content value of 0.9% showed that the soil used was in a near-dry state, and of uniform moisture distribution. Most soils have moisture content well above 3% in the 'dry' natural state. The particle size distribution test results confirmed that soil 'S' was composed of all the four main soil fractions: fine gravel (1.3%), sand (75.4%), silt (8.1%), and clay (15.2%). The amount of silt was however lower than the recommended minimum of 10%. The values obtained still fall within the range for suitable soils for CSB production.

Specimen design and production of CSBs for further testing were done with the main objective of obtaining an adequate number of samples for all the planned experiments.

For each mix type, at least three blocks were obtained. From these blocks, smaller specimen sizes were cut. A total of 51 blocks of nominal dimension 290 x 140 x 100 mm were produced. Out of this number of blocks, over 306 smaller specimen sizes were obtained. This number was considered to be adequate for all the bulk and surface property tests planned for. The decision to use three specimens per test was based on earlier findings by other researchers, and the low variance calculated during preliminary tests. Careful attention was paid to the block production process: preparation, mixing, compression, and curing. The blocks produced were all found to be of high quality and fit for further testing. Each of the block samples produced and specimens obtained were carefully labelled for easy identification.

From the preceding conclusions, the objectives of Chapter 5 were fully met.

CHAPTER 6

BULK PROPERTIES AND PERFORMANCE

6.1 INTRODUCTION

As mentioned earlier in this thesis, CSB bulk properties can be influenced by the proportions of the main constituents that form the block and by the processing methods used to produce them (moulding pressure, curing conditions, etc.). The objectives of this chapter are twofold, namely: firstly, to identify the main bulk properties likely to affect the durability of a block, and secondly, to test experimentally the performance of blocks made using differing input variables (stabiliser content, mix-water content, moulding pressure, curing conditions, etc.). The bulk properties identified as likely to influence its durability include (Lunt, 1980; Baker et al, 1991; Illston, 1994; Rigassi, 1995):

- Wet compressive strength (WCS)
- Block dry density (BDD)
- Total water absorption (TWA)
- Total volume porosity (TVP)

For each of these properties, the effect of varying some of the input variables described before are investigated (Chapter 3). The results obtained from the tests are analysed with a view to identifying general trends as well as comparing the performance of traditional blocks and improved blocks. Current standards and initial performance characteristics of like materials such as fired bricks and concrete blocks are also compared. Finally, the results are used to validate or query theoretical assumptions made in the earlier chapters of the thesis. The implications of the findings on future methods of design and production are discussed.

The coverage in Chapter 6 is limited to the discussion of experimental findings related to the above properties. All experiments were conducted following standard procedures to ensure accuracy, repeatability and reproducibility. Chapter 6 is presented in six sections. After this introductory section, the others include discussions of the findings relating to compressive strength, dry density, water absorption, volume porosity, and conclusion.

6.2 THE COMPRESSIVE STRENGTH OF BLOCKS

The compressive strength of a block is perhaps one of its most important engineering properties. It was established from the literature that the durability of CSBs increases with increase in its strength (Stulz & Mukerji 1988; Houben & Guillaud, 1994). Indeed a stronger block which has been well cured is usually better resistant to deleterious environmental agents (Chapter 2 and 3).

It is on the basis of the value of the strength of a block that its mechanical and other valuable qualities are judged (Rigassi, 1995; Young, 1998). Knowledge of the compressive strength value of a block can be used in a number of ways. They include:

- to check the uniformity of block quality
- to compare a given block sample with a specified requirement
- to approximate the degree of hydration achieved by OPC (through the strength of bonding)

- to classify a block in terms of its resistance to abrasive durability

Just as is the case with concrete, CSBs are composite materials. Such materials are known to be brittle and are therefore more accommodating of compressive stresses than tensile ones. The tensile strength of a block is about 90% lower than its compressive strength (Fitzmaurice, 1958). For this reason, the discussion in this section is confined to the behaviour of a block under compression only. The discussion is presented under the following three sub-headings:

- Type of inter-particle bonding in CSBs
- Factors influencing strength in CSBs
- Test methods used to investigate the compressive strength in blocks

Type of inter-particle bonding in CSBs

As a heterogeneous mixture of fine gravel, sand, silt, clay and stabiliser, the type of bonding between the different particles in a CSB is believed to be complex (Ingles, 1962; PCA, 1971). The nature of the bond is known to greatly influence its compressive strength. Unfortunately, determination of the quality of bonding is difficult to assess as no accepted test exists at the moment. However, most of the strength of a block is said to depend on the bond between the cementitious matrix and the coarse soil fraction (fine gravel and sand) (Houben & Guillaud, 1994). Physical mechanical interlock takes place between OPC hydrates and the mainly sandy fraction of a soil, with the bond strength varying from point to point (Mitchell & El Jack, 1978). The bond strength also varies according to the type and texture of the coarse soil fraction. It is generally held that characteristics of sand which do not permit penetration of its surface by the hardened cement paste cannot be conducive to good bonding. Soft, porous and mineralogically heterogeneous sand particles are likely to result in better bonding with cement paste. This consideration is often mentioned in concrete research (Glanville & Neville, 1997; Young, 1998).

As sand particles form the bulk of a block, by preserving their own integrity through their own high internal bonds, they constitute the strongest component within the block. Such high internal strength surrounded by 'weaker' contact strength can influence the path lines of failure in a block (cracks). The compressive strength of a block cannot therefore be expected to exceed that of its constituent sand particles. This theory is easier to assume than to test experimentally.

The cement hydrates that intertwine sand particles in a block are known to be porous aggregation of interlocking fibres (Hertzog & Mitchell, 1963). Bundles of these fibres form a cross-linked anisotropic network that effectively limits and opposes movement within the block fabric. The bonds between OPC hydrates are reported to be of the van der Waal type (Weidemann et al, 1990; Young, 1998). Such bonds are known to be physical in nature arising from the large energy available at the surface of gel particles. The forces at the surface of these gels can be large in comparison with their body forces. The bonds within OPC hydrate fibres are however chemical in nature (of the ionic and covalent types) (Taylor, 1998). Such bonds are stronger than the physical ones. These bonds are strong enough to resist any unlimited thixotropic expansion that might normally occur. Lastly, the bond between clay particles in a soil and the OPC hydrates is thought to be of the chemical type (Herzog & Mitchell, 1963; Ingles & Metcalfe, 1972). Through linkage due to the presence of water, a fairly stable chemical bond occurs between the clay minerals and the freed lime from the hydration reaction of cement.

In summary therefore, the strength of a block is governed by the strength of its cement paste, the strength of bonding between the cement paste and sand particles, and the internal strength of the sand particles (Uzomaka, 1978).

Factors likely to influence the strength of CSBs

The strength of CSBs can be influenced by a number of factors (BRE, 1980; Hughes, 1983). The main ones are the:

- water-cement ratio and degree of hydration
- degree of compaction
- state of moisture in a block
- temperature of a block
- age of a block
- type of coarse fraction present

The above factors are briefly discussed each in turn.

The *water-cement ratio and the degree of hydration* are known to determine the strength of a cement matrix (Neville, 1995). It can be expected that the lower the effective water cement ratio and the higher the degree of hydration, the lower the capillary porosity and the stronger the block. This can be achieved through accurate determination of the water cement ratio (proportioning and consistency testing) and proper curing (to maximise the degree of hydration). The degree of hydration can increase as long as moisture continues to be available for hydration. Wet curing of green blocks soon after demoulding is therefore a critical factor in this respect. This phenomenon is investigated experimentally in this research. The total volume porosity of a block and its correlation to strength are also investigated experimentally in this thesis (Section 6.5).

The *degree of compaction* can also affect block strength (Chapter 3). Compression reduces the amount of voids and increases inter-particle contact within a block.

Higher density has always been associated with higher strength (Spence, 1975; Gooding, 1993). This phenomenon is also investigated experimentally in this thesis.

The *moisture state* of a block can also influence its strength. Saturated blocks are weaker than dry blocks (Fitzmaurice, 1948; Houben et al, 1996). The difference in strength can be explained in a number of ways. Firstly, the presence of moisture in a block lowers the weak van der Waals bonds between the surfaces of the cement hydrates and the surface of the sand particles in the material. Secondly, since CSBs contain clay minerals, their high affinity for water leads to absorption and subsequent dispersal of any unstabilised grains. This can have the undesirable effect of weakening the state of bonding in the block. Thirdly, in a saturated state, as a block is subjected to loading, internal pore pressures can build up within it. Such pressure build-up can lead to the type of stress relief normally associated with disruption of inter-particle and inter-phase bonding in cement-based materials (Lea, 1970; Newman, 1986). The difference between the wet and dry compressive strength of a block is likely to be a valuable indicator of the strength of bonding achieved within it. The smaller the gap between the two, the higher can the bond strength be expected to be. This difference is also investigated experimentally in this thesis (Section 6.2.2).

The *temperature* of a block can also influence its strength, and by implication its durability. The effect on strength is likely to be more pronounced during the early age of a green block. It is known that the hydration reaction between OPC and water depends on temperature (Weidemann et al, 1990; Illston, 1994; Young, 1998). The rate of hydration increases as the temperature increases. At later periods in the life of a block, higher temperatures can still be counterproductive. A higher temperature maintained during the service life of a block is likely to result in short-term strength gains but, lower long-term strengths. This phenomenon is not investigated experimentally in this thesis.

The *age* of a green block influences its strength since the degree of hydration of the OPC stabiliser is known to increase with time. During the early stages of production, the degree of hydration within a block increases with curing age, and so does its strength. This phenomenon is investigated experimentally in this thesis. It is also possible that the hydration reaction of OPC might never really become complete (Taylor, 1998). CSBs are therefore likely to continue to gain strength for many years. The rate of increase in strength is however known to decrease after some years.

The *type of the coarse-soil fraction* (fine gravel and sand particles) used to produce blocks can also influence their strength. Increased surface roughness of sand particles is thought to be beneficial in improving bonding, mainly due to improved mechanical interlock between the sand particles and the OPC hydrates. Moreover, improved grading of sand particles can also improve the degree of interlocking due to closer packing of the grains within a block (Chapter 3). Conversely, the use of larger soil grain particles is likely to be disadvantageous. This is because larger soil fractions have a lower overall surface area with a corresponding weaker transition zone. Limits on the size and proportion of the maximum soil fraction can therefore lead to improved bonding, and thus strength of blocks. During the block production stage for this research, the maximum allowable coarse fraction size was limited to 5 mm (by screening with a 5 mm aperture sieve, Chapter 5).

Compressive strength test methods and factors considered

In this sub-section, highlights of the test method and factors considered during compressive strength evaluation of CSB specimens are discussed. Block specimens were produced as discussed in Chapter 5.

The compressive strength of a block is the failure stress measured normal to its face. For all CSB specimens tested, standard methods of test were used throughout (BS 6073: Parts 1 and 2: 1981; BS 3921: 1985). For research purposes only, some similar block specimens were tested both in their wet and dry state. Current standards only recommend testing of samples in the wet state. The reason why dry state testing was also done was explained earlier in this section.

Standard procedures which were consistently followed with no departures allowed are presented in Appendix O. This was done because even with standard procedures, a slight variation in one of a number of test conditions can easily affect the outcome. The most important test factors considered included the:

- block specimen size
- sample moisture condition
- specimen curing age
- specimen end-surface preparation
- rate of application of loading
- rigidity of the testing machine

Each of the above factors are now briefly discussed in turn.

The *block specimen size* was kept uniform as 100 mm cubes for all samples tested. Although standards permit use of cylinders as well, it was found to be a lot more convenient to use cube prisms. The decision was mainly dictated by the method of manufacture used to produce full-scale block samples. Cutting out smaller specimens from the full-scale sizes had several advantages. The 100 mm cube specimens were:

- easy to cut out
- expedient to protect from damage

- cheaper to make
- convenient to test with a lower capacity machine
- economic to test as less material was wasted (the test was destructive)
- small enough to be less likely to contain elements of weak points

It is well established that the strength of a cement based specimen decreases with size (Neville & Brooks, 1994). So does the *variability* in strength of geometrically similar block specimens, because smaller specimens are more homogenous. The results obtained were satisfactory with a high degree of consistency (Section 6.2.1).

The choice of *moisture condition* in the test samples was considered an important factor. Testing in the 'wet' state is advantageous in that it is more reproducible than testing in the 'dry' state. Testing in the latter state can be unhelpful since it includes a widely fluctuating degree of dryness in a block sample used. The outcome from such a test would really not be that accurate to compare. Testing in the dry state can also lead to higher strength values being recorded (Fitzmaurice, 1958). This can be misleading when related to actual service conditions where blocks are likely to be continually subjected to moist conditions. Testing in a wet condition therefore relates better to real life applications of the block. For purely research purposes, compressive strength tests on a select few number of blocks were conducted in both states.

The choice of *specimen curing age* was based on the specifications of the stabiliser type used. As stated earlier, curing is associated with the rate of hydration of the stabiliser used. Indeed it is with age of hydration that OPC gains strength, especially at the early stages of the process (Weidemann et al, 1990). For practical purposes the hydration of OPC is generally regarded as being substantially complete at 28 days (Illston, 1994). Compressive strength test values obtained around this time ought to reflect the full strength of a cured block. No significant increase in strength is likely to be recorded after 28 days (Lea, 1970). For these reasons, all cement based block specimens were tested at 28 days. Lime based blocks were tested at 56 days (BS 890, 1972; Bessey, 1975; Coad, 1979).

The *end surface preparation* for each test specimen was considered a critical factor likely to affect the results. As during the test two dissimilar types of surfaces (block surface and testing machine platen) would be coming into intimate contact with one another a special precaution was required. While the surface of the end platen of the machine might be smooth, the surface of the CSB specimen is rougher, uneven and not really plane (ILO, 1987). Such dissimilarities can give rise to undesirable stress concentrations on the block specimen. This effect can lead to variations in test results to the extent that the outward compressive strength of a block specimen would appear to be diminished (Bungey & Millard, 1996). In order to overcome this problem, the surfaces of all block specimens tested were capped using plywood pieces of dimensions 105 x 105 x 12 mm (BS 6073: Part 1 and 2, 1981). The size was chosen to be about the same as the top of the block specimen. In this way, the influence of any surface defects in planeness that could have created significant variations in test results were removed, or at the very least minimised. The narrow scatter of test results later showed that this decision had been the correct one (Section 6.2.1).

The *rate of application of loading* (compression testing machine) was also regarded as an important factor likely to affect the seeming strength of test specimens. It is well established that the lower the rate at which the stress is increased onto the block, the lower will the eventual recorded compressive strength be (BS 6073: Parts 1 and 2, 1981). Conversely, if the load is applied rapidly, higher strength values can be recorded. The reasons behind such outcomes is based on the rate of increase in strain over time. It is widely reported in concrete studies that when the limiting strain is reached too soon, failure also takes place prematurely (Lea, 1970; Neville & Brookes, 1994; Jackson & Dhir, 1995). It is therefore important that the stress on block specimens is applied at a uniform and consistent rate for all samples tested. In this way comparable results can be obtained. The selected rate of loading applied gradually without shock for all specimens tested was 15 KN/minute (BS 6073: Parts 1 and 2, 1981; BS 3921: 1985; ILO, 1987). It was generally observed that stronger blocks (higher stabiliser content 7%-11%) exhibited lower severity to the strain rate. The required rate of loading was selected using an electronic input board attached to the testing machine. Failure in most blocks samples occurred within about 2 to 4 minutes.

The *rigidity of the testing machine* was the last factor considered. A less rigid testing machine can store up energy leading to explosive fractures occurring in the test specimen. The machine used for the tests had just been serviced and recalibrated only a few months earlier. This problem cannot therefore be a source of errors for the tests conducted during that period.

After failure was achieved, the crushing strength of each block specimen was calculated by dividing the maximum recorded load carried (in KN) by the cross-section area of the specimen (mm^2). The crushing strength was expressed to the nearest 0.05 KN/mm^2 (MPa). The results are discussed in Sections 6.2.1 to 6.2.4.

6.2.1 EFFECT OF VARYING THE STABILISER CONTENT AND MOULDING PRESSURE ON THE WCS OF CSBs

The values of the 28-day mean wet compressive strength for both traditional and improved blocks are shown in Appendix P(1) to P(5). A plot of these values against the range of cement contents used is given in figure 6. Each of the data points shown plotted in figure 6 is an average of three separate experimental results (Chapter 5). The key to the symbols used on the graph is also given. All subsequent graphs are presented following the same format.

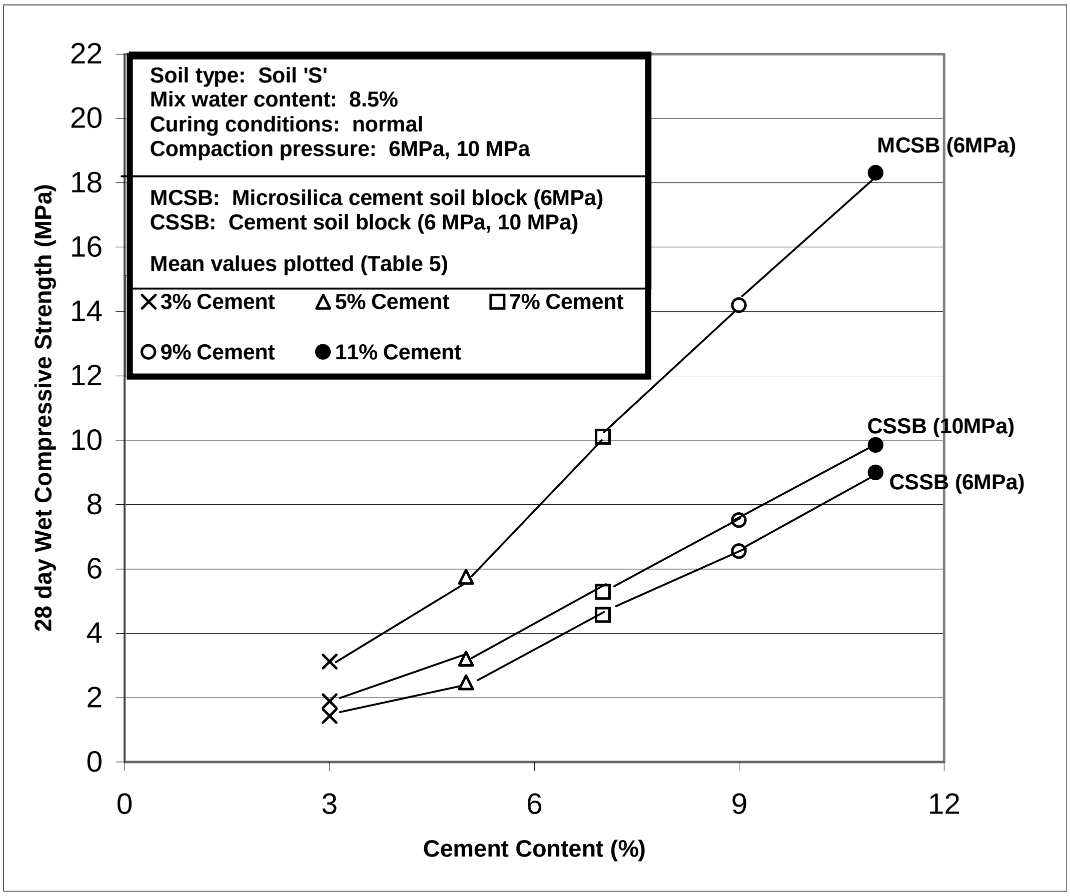


Figure 6: Effect of varying the stabiliser content and compaction pressure on the wet compressive strength of CSBs. (University of Warwick, 2000)

The discussion of figure 6 is conducted along three lines: the range of values obtained, comparison of these values to existing standards, and analysis of the trends shown by the results. This approach is used for all other subsequent results presented in this thesis.

Table 5 shows a summary of the plotted values in figure 6.

Cement content	28-day Wet Compressive Strength		
%	MPa		
	MCSB (6MPa)	CSSB (10MPa)	CSSB (6 MPa)
3	3.12	1.89	1.43
5	5.76	3.21	2.48
7	10.11	5.29	4.57
9	14.19	7.51	6.54
11	18.3	9.84	8.99

Table 5. Mean wet compressive strength values (28-day) for MCSB and CSSBs.

The values of the average 28-day wet compressive strength for both traditional (CSSB) and improved blocks (MCSB) were satisfactory. The values ranged between 1.43 MPa and 8.99 MPa in the case of the former and between 3.12 MPa and 18.3 MPa in the case of the latter. The lower values in either case correspond to the cement content of 3%, while the higher ones to 11%. As can be seen, the WCS values in improved blocks were found to be considerably higher than in traditional blocks made in exactly the same manner but without the addition of microsilica. On average the addition of microsilica resulted in the doubling of strength in blocks. Although some improvement had been expected, the magnitude of the strength gain achieved was surprising. Such high values had not been previously obtained with the corresponding amounts of OPC according to current CSB literature (Rigassi, 1995). The inclusion of a partial cement replacement materials (CRM) such as microsilica therefore appears to be an effective way of increasing the WCS of blocks. These results also confirm the earlier theoretical assumptions described in Chapter 3. This approach represents a new way forward in terms of strengthening CSB fabrics for wider engineering applications. It is also likely to be particularly useful for blocks exposed to severe environmental conditions.

According to literature sources, recommended WCS values for CSBs are quite wide-ranging, varying from country to country, and from author to author. The experimental values obtained here however, compare well with most current CSB standards. Some recommended minimum values are: 1.2 MPa (Lunt, 1980),1.4 MPa (Fitzmaurice, 1958) and 2.8 MPa (ILO, 1987). The value of 1.2 MPa is now more widely used (Houben & Guillaud, 1994). The lowest experimental value obtained for traditional blocks (1.43 MPa), is about 20% higher than this. For improved blocks the 3.12 MPa value is about 62% higher. Both values correspond to blocks stabilised with 3% OPC. The blocks made with OPC contents of 5-7% were all significantly stronger than the 1.2 MPa standard (5 to 8 fold stronger). Moreover, by interpolating the plotted values for IPD blocks below the 3% cement content point, it can be estimated that only about 1% of the binder content would be required to achieve the minimum recommended WCS value of 1.2 MPa. The approach established from these results constitutes a significant new finding.

The preceding discussions concerned variation in stabiliser content only. The results of varying compaction pressure from 6 MPa to 10 MPa over the same range of cement contents for TDB blocks are also shown in figure 6. No improved block samples were subjected to similar variations in compaction pressure. The results show that for the same stabiliser content, increase in compaction pressure leads to an increase in WCS. It was found that at lower cement contents, increase in compaction pressure from 6 MPa to 10 MPa (about 70%) resulted in increase in WCS of about 32%. A similar increase in compaction pressure resulted in a corresponding increase of only 9% at the higher cement contents (11%). Within the range of interest (5-7% cement content), the increase in WCS was between 16 and 30%.

These values are much lower than the dramatic increases witnessed by varying the stabiliser content. The findings confirm earlier work by other researchers that increase in stabiliser content is a more economic way of increasing the wet compressive strength in blocks (Lunt, 1980). Blocks stabilised at high stabiliser contents but compacted at low compaction pressures were found to perform satisfactorily. The final wet strength of a block appears to be more sensitive to changes in cement content than compaction pressure. The results also show that although improved performance can be achieved by increasing compaction pressure, the degree of improvement diminishes as this pressure is increased. Block making machines operating within the range of 4 to 8 MPa should therefore be adequate to give satisfactory results (Houben & Guillaud, 1994).

6.2.2 Comparison of the ratio between Mean dry and wet Compressive Strength

The effect of varying the stabiliser type and content on the gap between mean dry and wet compressive strength was investigated experimentally. The values obtained are plotted as shown in figure 7.

The range of the plotted values shown in figure 7 are summarised in table 6.

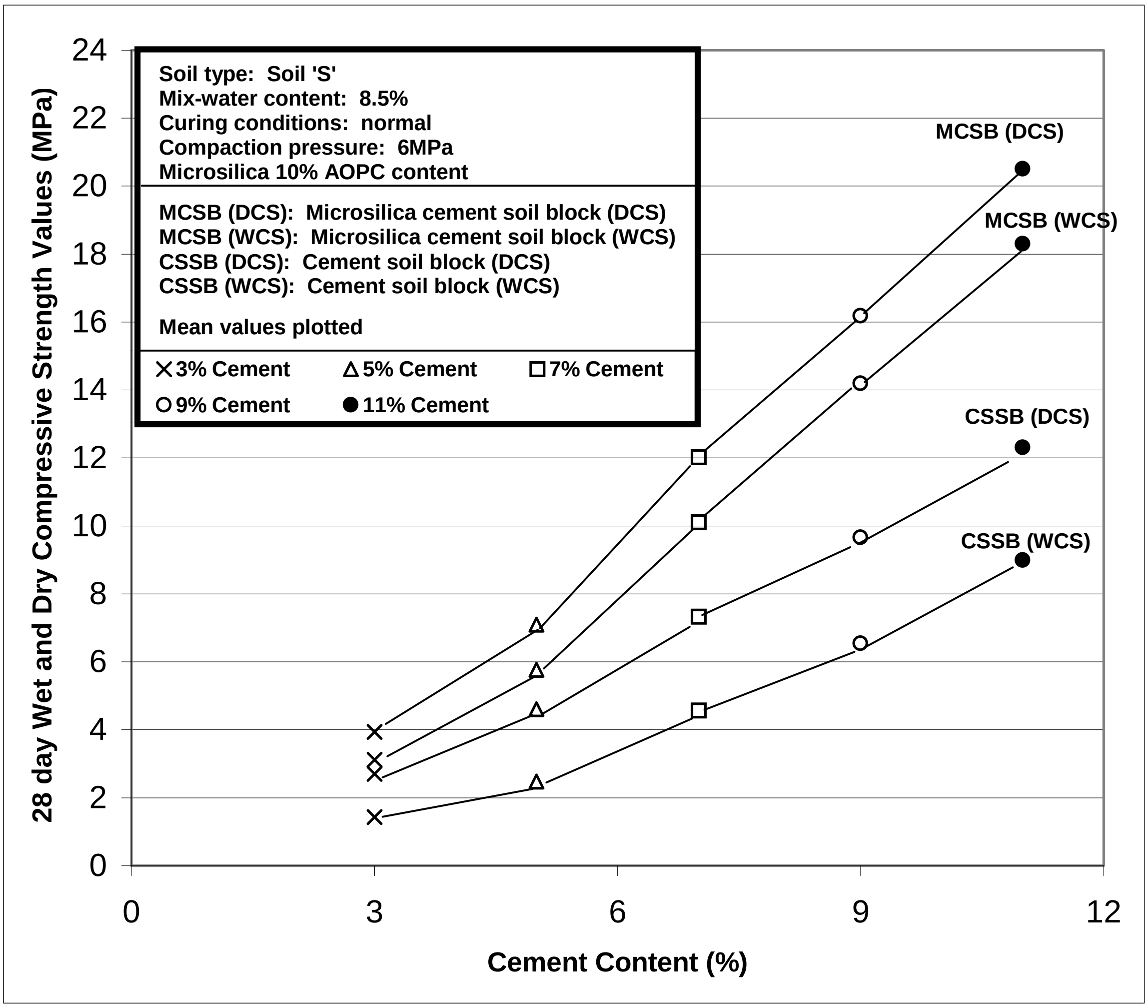


Figure 7: Comparison of the mean wet and dry compressive strengths in both traditional and improved blocks (University of Warwick, 2000).

Cement content	Mean Compressive Strengths					
%	MPa					
	MCSB			CSSB		
	WCS	DCS	Ratio	WCS	DCS	Ratio
3	3.12	3.94	1.3	1.43	2.70	1.9
5	5.76	7.09	1.2	2.48	4.61	1.9
7	10.11	12.02	1.2	4.57	7.33	1.6
9	14.19	16.18	1.1	6.54	9.66	1.5
11	18.30	20.50	1.1	8.99	12.30	1.3

Table 6. Values of the 28-day mean WCS and DCS of TDB and IPD blocks: Ratio (DCS/WCS).

The values of the mean WCS in traditional blocks ranged between 1.43 MPa and 8.99 MPa. The equivalent values of their dry compressive strengths ranged between 2.70 MPa and 12.3 MPa. The difference between mean DCS and WCS ranged between about 40% (for 11% cc) and 90% (for 3% cc). This shows that the higher the cement content, the lower the fractional difference between mean wet and dry strength in a block.

A similar trend emerged with results obtained for improved blocks. The mean WCS in these blocks ranged between 3.12 MPa and 18.3 MPa, while the matching dry strength ranged between 3.94 MPa and 20.5 MPa. Apart from the inclusion of microsilica (10% of the cement content), all other production variables remained the same. As before, the magnitude of the gap ranged between 12% and 26% only (for 11% and 3% cc blocks respectively).

The results for improved blocks compare well with values reported in concrete research where the difference between mean wet and dry compressive strength ranges between 9% and 21% (Neville, 1995). The results for traditional blocks similarly compare well with results obtained by earlier researchers. It was found that the difference between the two strength values in stabilised blocks varied between 35% and 120% (Fitzmaurice, 1958). It has also recently been recommended that the ratio of the mean dry and wet compressive strength in CSBs should not be greater than 2 (Houben et al, 1996). The experimental results obtained here for both traditional and improved blocks fall well within this limit. However, the ratio in improved blocks (1.1 to 1.3) is much lower than in corresponding traditional blocks (1.3-1.9). The variation in ratios for IPD blocks is also less wide-ranging than the case with TDB blocks. The considerable reduction in the gap between the mean dry and wet compressive strengths achieved in improved block represents a major breakthrough in

CSB development. It is well established that the higher the gap between the two, the lower can the strength of bonding between the particles and phases in a block be expected to be (Houben et al, 1996).

The marked increase in strength witnessed in improved blocks as opposed to traditional blocks can be linked to an increase in the degree of bonding within the block. In this case, the improvement can solely be attributed to the inclusion of microsilica in the mix (Chapter 3). The use of this CRM in moderate amounts (5 to 10% of the OPC content) for particular applications is recommended in preference over OPC-only mixes. The general pattern of improvement in strength and other properties of the block are undeniable. The upturn in strength is a consequence of its pozzolanic reaction with the freed lime from the hydration reaction of OPC with water, and also due to its ability to effectively 'fit in' between the OPC grains (Weidemann et al, 1990; Illston, 1994; Young, 1998; Taylor, 1998).

According to cement literature, in traditional blocks, the cement hydrates produced from the hydration reaction grow away from the OPC grains (Weidemann et al, 1990; Taylor, 1998). Even when hydration is deemed complete, the extended hydrate structure is likely to remain sparse, weak and permeable, given the low amounts of OPC used. When microsilica is added, the situation is markedly different. Being an almost pure silicon dioxide, when well dispersed in the cement/block, it can surround every OPC grain with about 100,000 microspheres (surface area 15,000 m^2/kg compared to 350 m^2/kg in OPC) (Illston, 1994). Finer and stronger hydrates can then grow from both the OPC and the 'nucleation centres' of the well dispersed CRM. As can be expected, this action can transform the relatively weak and porous structure into a far denser, more homogenous and impermeable matrix than hitherto possible. As the results in this thesis have shown, dramatic improvement in block properties are evident. Higher strength, density and hardness, as well as effective abrasion resistance and possibly longer service life, are some of the beneficial outcomes. The fact that the block attains superstrength in a matter of weeks shows that improved blocks are likely to outlast traditional blocks in whatever condition they are used in. The wear resistance of both categories of blocks are investigated experimentally in Chapter 7.

In summary, the inclusion of a CRM such as microsilica can be expected to have the following beneficial effects on CSBs:

- Rapid strength development
- Rapid surface drying (to below 75% relative humidity in one to five days)
- No bleeding or segregation at the surface
- Very low surface permeability
- Higher wear and abrasion resistance
- Extreme durability
- Reduced life cycle maintenance costs
- High strengths (compressive, tensile, flexural)
- Very low bulk permeability and sorptivity
- Lower shrinkage and creep
- Extended use of CSBs for flooring, foundations, pathways, underwater applications, etc.

It has also been reported in the literature that concrete materials achieve about 80 to 90% of their ultimate strength within 28 days of production (Neville, 1995). The comparable value attained by CSBs within the same period is lower, about 60-70% (Houben & Guillaud, 1994). The use of a rapid strength developing material such as microsilica is therefore a major advantage where early high strength is required. The problem of increased costs due to its inclusion can be overcome in either of a number of ways. These include the use of hollow blocks, thin surface layered blocks, frog-bedded blocks and interlocking blocks. All these simple measures can effectively reduce the amount of microsilica actually used and thus enable cost reduction to be achieved at no extra expense. The use of microsilica in improving block strength, dimensional stability and durability is therefore highly recommended.

6.2.3 THE EFFECT OF MIX HOLD-BACK TIME ON THE WET COMPRESSIVE STRENGTH OF BLOCKS

The ultimate cured wet strength of a block can be affected by the manner in which it is produced. At stated earlier in the thesis, despite its importance this is an area which has previously received very little attention in CSB literature. The effect of mix hold-back time was investigated experimentally to establish the extent to which WCS can be affected when this variable is introduced. The investigation was limited to traditional blocks stabilised with 5% OPC and compressed at 6 MPa. A similar effect is likely to apply in the case of improved blocks. The phenomenon was also investigated experimentally because it was found during the fieldwork (Chapter 4) that batches too large to be moulded within the hour were being widely used. Very little concern was being shown in the field regarding the potential adverse effects of this variable on the quality of blocks produced.

The mean values of the experimental results obtained are shown in table 7 and figure 8. Each point represents the average of three block specimen samples.

Time	WCS 28-day	Ratio
minutes	MPa	
5	2.53	1.0
30	2.08	1.2
60	1.73	1.5
90	1.58	1.6
120	1.49	1.7

Table 7. Wet compressive strength values (28-day) of CSSBs compacted at various hold-back times.

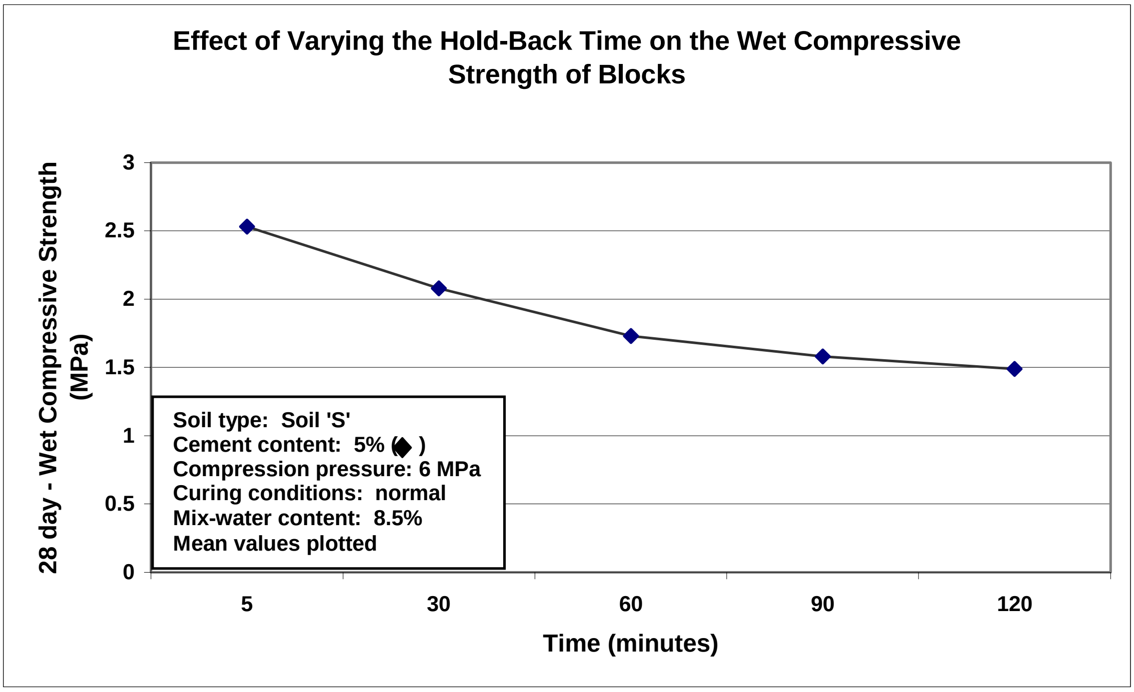


Figure 8. Effect of mix-hold back time on the 28-day WCS of CSSBs (University of Warwick, 2001)

Figure 8 and table 7 show that the average 28-day wet compressive strength fell from 2.53 MPa to 1.49 MPa as the hold-back time was increased from 5 to 120 minutes (a 41% loss).

It was found that blocks compacted within 20 minutes after damp mixing were about 27% stronger than those compacted after 45 minutes of delay. These findings confirm earlier results by other researchers. For example, it was found by Rigassi (1995) that loss of strength after two hours delay was about 50%. It was also found that blocks moulded within 20 minutes of damp mixing were between about 30 and 40% stronger than those compacted after 45 minutes (Houben & Guillaud, 1994). The time of 45 minutes is used as a yardstick because it approximates the early setting time of OPC (Weidemann et al, 1990). The findings here confirm that a general downward trend of loss of strength due to long hold-back times should be expected when OPC is used as the stabiliser. The opposite is true for lime (Bessey, 1975; Coad, 1979). OPC stabilised blocks should therefore be compacted within 20 minutes of mixing, but certainly not after 45 minutes. It is still common field practice to mix batches for hourly production which end up not being used up within the hour (Chapter 4). This discussion confirms earlier statements made in this thesis that the processing method employed during production can significantly affect the ultimate quality of a block (Chapter 3). It is therefore recommended that all CSB production stages should be treated with the same level of skill, competence and supervision.

6.2.4 The Effect of Varying Curing Conditions on the Wet Compressive Strength of Blocks

The effect of varying curing conditions on block strength was investigated experimentally for a limited range of blocks (Chapter 3). CSB specimens stabilised with 5% OPC and compacted at 6 MPa were used. As the test was meant to be indicative only, the OPC content was not varied as before. The investigation was also restricted to traditional blocks only on the assumption that similar effects would be applicable in improved blocks of matching cement content. The curing conditions were varied to approximate common practice in the field on the one hand, and to test theoretical predictions that full moist curing would be beneficial on the other. Curing time of 28 days was maintained for all blocks. The conditions were varied as follows:

Condition A: Open exposure (within a well lit area in the laboratory)

Condition B: Normal curing (7 days wet curing, 21 days dry curing)

Condition C: Complete cover (with polythene sheeting material)

Condition D: Wet curing throughout (by immersion in water 24 hours after moulding until testing time)

After 28 days, three specimen samples from each curing condition category were tested as before. The results are shown in table 8 and are also plotted in the form of a histogram (figure 9).

Condition	28-day WCS	Ratio
	MPa	-
A	1.13	1.0
B	2.54	2.3
C	3.28	2.9
D	6.85	6.1

Table 8. 28-day WCS values of CSSBs cured under varied conditions

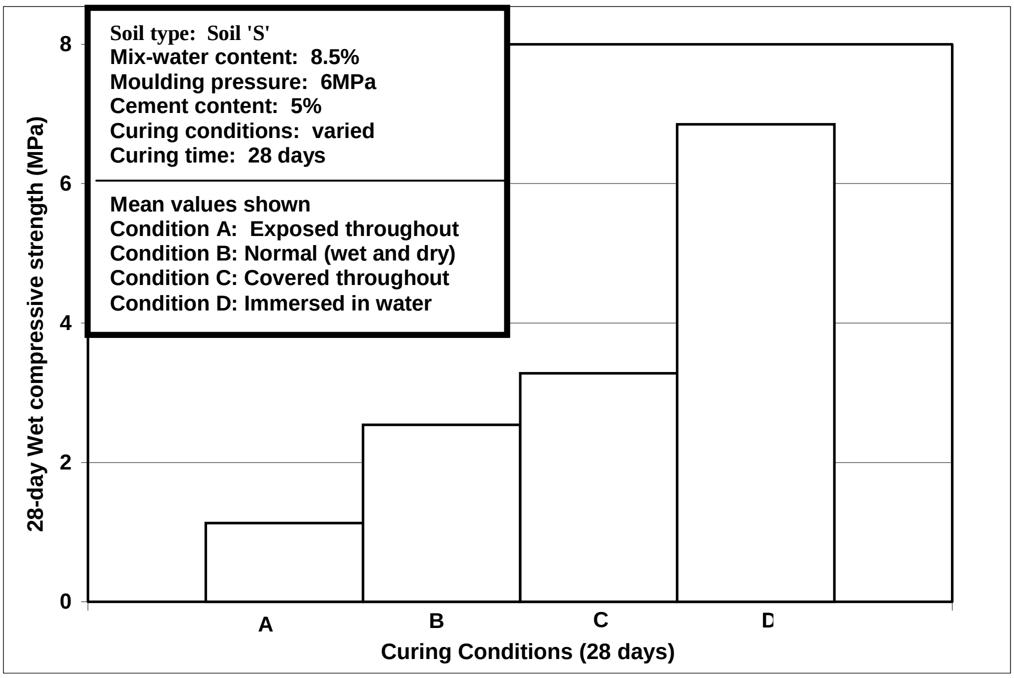


Figure 9: Histogram showing the effects of varying curing conditions on the wet compressive strength of CSBs (University of Warwick, 2001)

When condition A blocks are compared with the other blocks, the differences in strength found are considerable. Normal cured blocks were about twice stronger than exposed blocks. If these blocks had been left exposed in the sun, it is likely that the difference in strength could have been even higher. The case actually approximates actual practice on most CSB production sites (Chapter 4). The belief in such places is that the faster the loss of moisture, the stronger the block becomes and thus the faster it can be used. It is a mistaken concept since prolonged retention of moisture can be beneficial in ensuring that the hydration process continues till maximum hydration is achieved (Chapter 3). This finding partly explains the poor performance of CSBs as observed during the fieldwork.

The results also show that block samples covered throughout are about three fold stronger than those cured under exposed conditions. Blocks cured fully immersed in water for 28 days were about six fold stronger than similar ones cured exposed. The immersion in water was done 24 hours after demoulding since initial attempts to immerse them immediately proved futile. The blocks were then left in water till testing time. The difference in wet strength between the fully immersed cured blocks and the exposed blocks is about 506%. This should be a cause for concern since the latter simulates field practice in most developing countries (ILO, 1987). The recommendation here is that CSB standards covering specifications and workmanship during production should lay more emphasis on the need for proper curing. The procedures to be followed should be clear, simple and easy to understand and execute (Lowe, 1998; Schildermann, 1998). A checklist system spelling out all the necessary steps during production ought to be beneficial in this regard. The main emphasis should be on keeping green block surfaces moist for as long as possible. Exposure of green blocks to rainy conditions, direct sunlight and wind conditions should be avoided especially during the first few days of production (Chapters 3 and 4).

6.3 BLOCK DRY DENSITY (BDD)

The density of a block is a valuable indicator of its quality. It can be expressed in a number of different ways, depending on the pre-existing moisture state of the block, thus:

- Block dry density (BDD) (usually indicating the oven-dried value when desiccated to $105 \pm 5°C$ for 26 hours)
- Block bulk density (BBD) (based on the pre-existing state of moisture, e.g. soon after demoulding)
- Saturated block density (SBD) (when soaked in water for between 24 and 48 hours after oven drying as before)

It is the dry density that is commonly used in building specifications (BS 6073: Part 2, 1981) and is the one discussed in this thesis. In addition to the solid phases that exist in a block, the material also contains pore spaces filled partly with air and partly with water (Jackson & Dhir, 1996). The amount of either phase depends on the moisture state of the block (varies from block to block). When both air and water are driven out (by oven drying to constant mass), the block dry density value is obtained. Apart from the state of moisture in a block, its density also depends on the following:

- the degree of compaction used (normally between 4 and 8 MPa)
- the density of the constituent materials (especially the coarse sand fraction) Sand has a dry density value of about 2,200 kg/m^3 while that for clay is about 2000 kg/m^3 (Houben & Guillaud, 1994)
- the size and grading of the soil particles
- the form of the block (solid, hollow, frogged)

Since the structural strength of a block is the result of the friction between the constituent cement hydrates and soil grains, the closer the packing of these solids, the stronger the block can be expected to be. Densification following the stabilisation of soil with OPC can ensure that the close packing achieved is maintained through the mechanical interlock of the grains. It is this interlock which limits excessive movements more than would have been possible if the stabiliser had not been used. Without the binder, either through omission or due to progressive decay, a block is likely to become weak. In such cases, the effects of densification can be progressively reversed (Lola, 1981; Minke, 1983).

The density of a block can have implications on most of its other bulk properties (Markus, 1979). These include compressive strength, permeability, water absorption, porosity, thermal capacity, sound insulation, hardness and durability (Lunt, 1980;

BRE, 1980; Spence & Cook, 1983). The higher the density of a block, the better can its performance be expected to be. For example, density has commonly been closely associated with the strength of a block (UN, 1964; Spence, 1975). The relationship between strength and density is investigated experimentally in this thesis to determine whether density can be a surrogate for the strength of a block. The correlation between density and water absorption and porosity are also investigated experimentally. Similarly, the correlation between density and durability is discussed in Chapter 7.

Determination of the density value of a block is provided for in most standards. The test method used in this thesis is based on the one described in BS 6073: Part 2, 1981. For all block specimens, three samples were tested in each category and the mean value used for subsequent analysis (Chapter 5). The density obtained in each case was expressed to the nearest 10 kg/m^3 (BS 6073: Part 2, 1981; BS 3921, 1985). The full summary list of the results are shown in Appendix Q(1) to Q(4). The findings are discussed in Sections 6.3.1 and 6.3.2. References to the same values obtained are also made in subsequent sections of this chapter as well as in Chapter 7.

6.3.1 EFFECT OF VARYING THE STABILISER CONTENT AND COMPACTION PRESSURE ON DENSITY

The effect of changing the above variables on density were investigated experimentally. The number and type of samples tested were as before. The compaction pressure was varied from 6 MPa to 10 MPa for a limited number of traditional blocks. The plotted results are shown in figure 10.

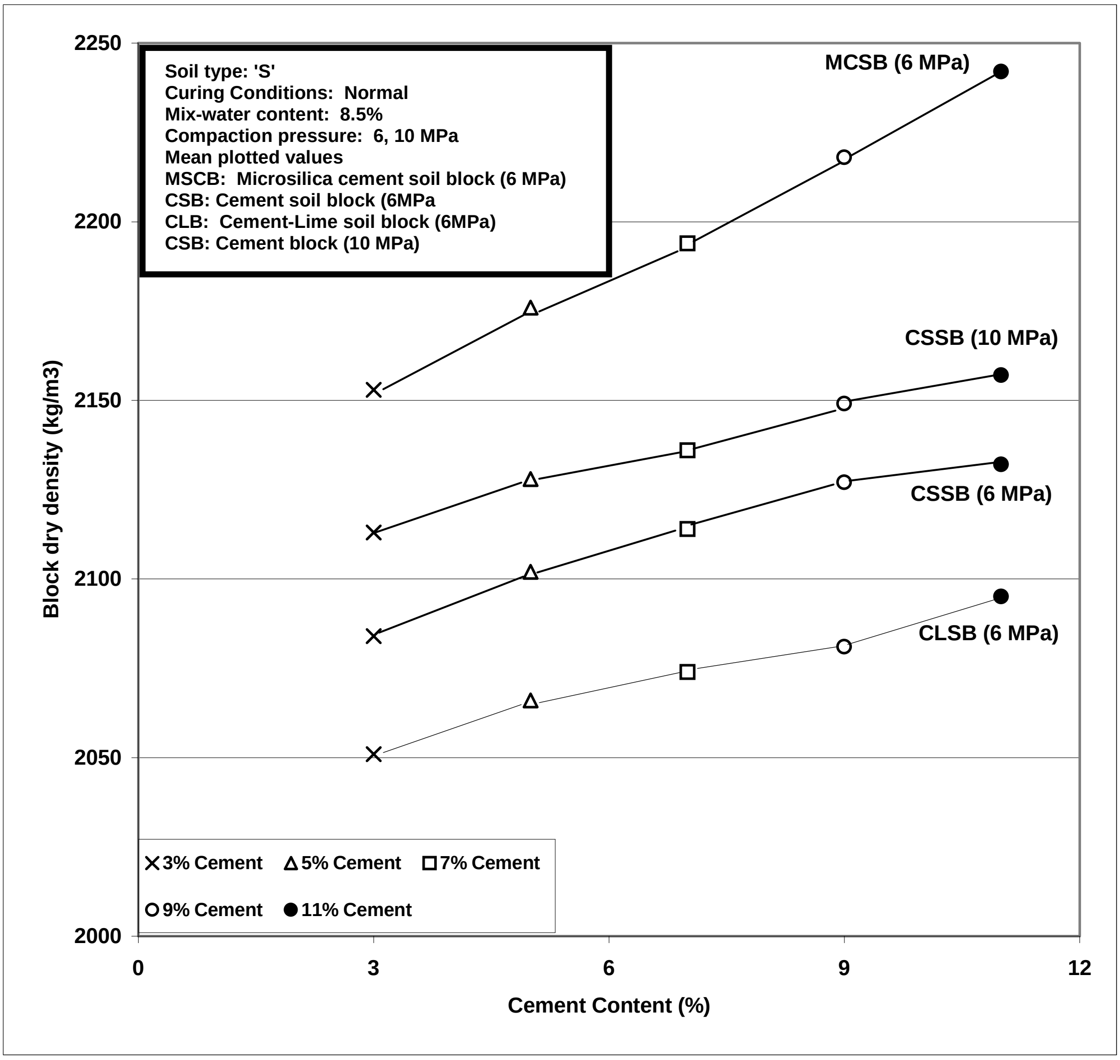


Figure 10: Effect of varying the stabiliser content and compression pressure on BDD

The range of BDD values obtained for both improved and traditional blocks are presented in table 9.

Block type / stabiliser	Compaction Pressure used	Range of BDD values	Density increase with increase in OPC from 3% to 11%
	MPa	Kg/m^3	%
Improved OPC + Microsilica	6	2153 - 2242	4.1
Traditional OPC + Lime	6	2051 – 2095	2.1
Traditional OPC only	6	2084 – 2132	2.3
Traditional OPC only	10	2113 – 2157	2.0

Table 9: Range of BDD values obtained for improved and traditional blocks

The two density values shown in table 9 correspond to blocks stabilised with 3% and 11% OPC respectively. Although the overall increase in density in improved blocks was marginally higher than in traditional blocks, the difference does not appear to be dramatic. However, for matching amounts of OPC content, improved block density was about 4% higher than in traditional blocks. The increase was directly proportional to the increase in cement contact. While the part substitution of OPC by lime resulted in less dense blocks, the partial substitution of OPC by microsilica produced the opposite effect.

Increase in density was also found to occur when compaction pressure was increased. For the OPC only stabilised blocks, increase in moulding pressure from 6 MPa to 10 MPa resulted in a corresponding increase in density of about 1.3% only. So for an increase in compaction of about 70%, increase in density of less than about 2% was achieved. This is considerably lower than the equivalent increase in density due to the inclusion of microsilica (between 3.3% and 5.2%). The addition of a partial cement replacement material appears to be an economic way of achieving higher densities in blocks. The marked increase in density witnessed in improved blocks could have been due to four factors associated with the inclusion of microsilica:

- Pore filling effects
- Increased homogeneity
- Improved bonding
- Reduced voids

The results further confirm the beneficial effects of using a partial cement replacement material such as microsilica (Chapter 3).

6.3.2 *Correlation Between Density and Wet Compressive Strength*

In this section, the correlation between density and strength is discussed. It was stated earlier that for better performance, a denser block would be required. Density was also mentioned as a valuable indicator of strength and durability in a block. The experimental results obtained for BDD are plotted against those for 28-day WCS (shown in figure 11).

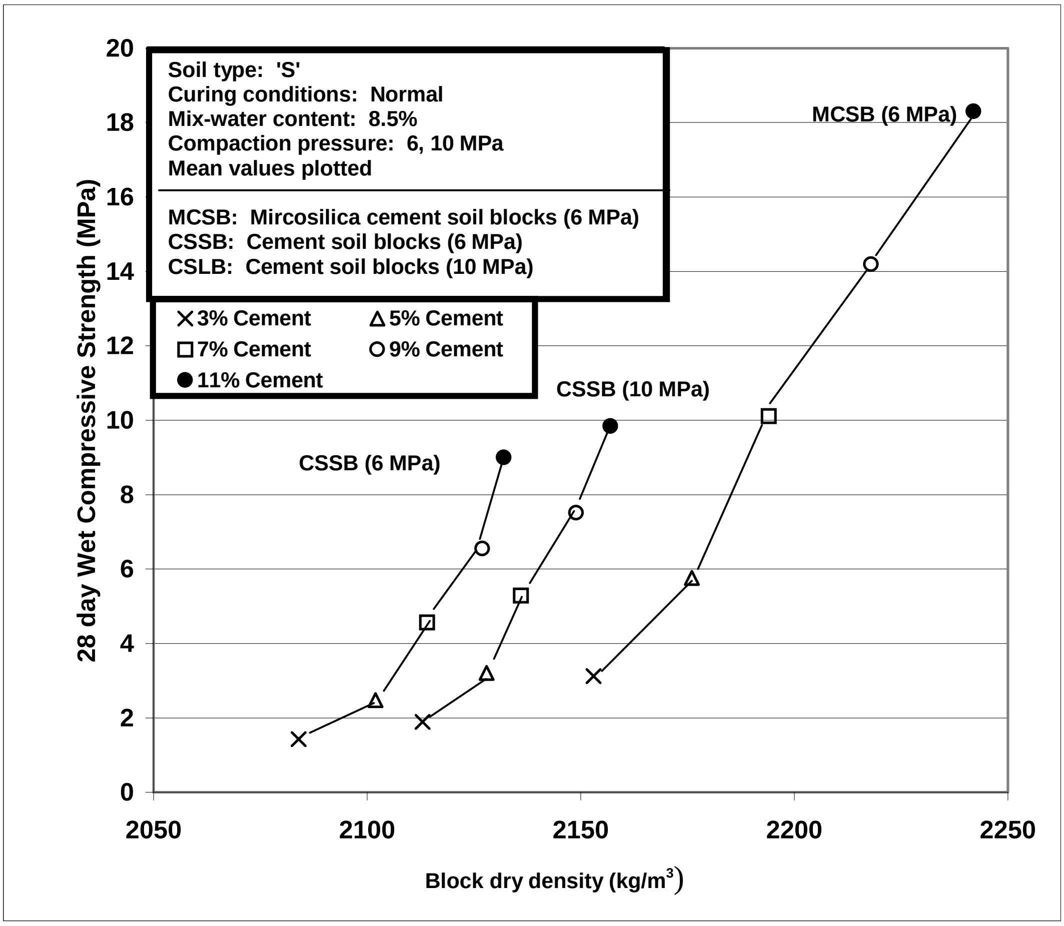


Figure 11: Correlation between BDD and 28-day WCS in CSBs (University of Warwick, 2001)

Figure 11 shows that a general positive correlation exists between BDD and WCS for all the different types of blocks tested. The graph shows that an increase in density is accompanied by a corresponding increase in strength. The coefficient of correlation and P-values are as follows: traditional block (0.971; 0.006), improved block (0.996; 0.000). A strong correlation therefore exists between BDD and WCS.

The correlation between density and strength has also been widely reported in comparable materials (Jackson & Dhir, 1996; Ruskulis, 1997). The dry density values for some like materials are:

- Fired clay bricks: 2250-2800 kg/m^3 (usually 2600 kg/m^3)
- Calcium silicate bricks: 1700-2100 kg/m^3
- Concrete blocks: 500-2100 kg/m^3

These values compare favourably with those obtained experimentally for CSBs. It is widely known that fired clay bricks are the most popular building material in most parts of the world (Parry, 1979; Agarwal, 1981; Spence & Cook, 1983). These blocks are denser, stronger and more durable than comparable materials. The average density of 2,600 kg/m^3 is probably a major contributor to the strength of fired bricks. The comparable density values also show that improved blocks were denser than concrete blocks by about 5%. The only drawbacks likely to result from higher densities are ease of handling and transportation. Blocks which are very heavy can be difficult to lay, and are normally expensive to transport (NASA, 1971; ILO, 1987).

6.4 TOTAL WATER ABSORPTION IN CSBs

Almost all bricks and blocks can absorb water by capillarity (Keddi & Cleghorn, 1980). The existence of pores of varying magnitudes in these materials confers marked capillarity in them. The total amount of water absorbed is a useful measure of bulk quality. The reason for this is that the total volume of voids (or pore space) in a block can be estimated by the amount of water it can absorb. This property is clearly distinct from the ease with which water can penetrate a block and permeate through it (Neville, 1995).

Knowledge of the value of the total water absorption (TWA) of a block is important because it can be used for:

- routine quality checks on blocks (surrogate test for quality)
- comparison purposes with set standards and values for other like materials
- the classification of blocks according to required durability and structural use
- approximation of the voids content of a block (Section 6.5)

Generally, the less water a block absorbs and retains, the better is its performance likely to be (ILO, 1987). Reducing the TWA capacity of a block has often been considered as one of the ways of improving its quality. The deleterious effects of moisture on block properties were discussed in Chapter 2. A block that readily absorbs water is likely to be vulnerable to repeated swelling and shrinkage as moisture and temperature variations take place. Repeated swelling and shrinkage is likely to progressively lead to the weakening of a block fabric (either directly or indirectly). A block that contains absorbed water is often weaker with a less hard surface than when it is dry. The presence of absorbed water can also lead to the creation of conditions suitable for the resumption and acceleration of otherwise dormant chemical activity (BSI, 1950; BS 7543, 1992). The lower the water absorption capacity of a block therefore, the more likely it is to be more durable.

Test methods used

The TWA capacity of a block can usually be measured by determining the amount of water it can take in (ILO, 1987). Since the amount absorbed is influenced by the pre-existing moisture condition of a block, it is advisable that it be first dried to constant mass before further testing (BS 3921, 1985). For this particular research, attainment of constant mass was determined using an electronic weighing scale accurate to 0.01% of the sample mass. Simple immersion of the specimen without prior evacuation can lead to incomplete absorption and saturation. Moreover, the suction exerted by a dry block is usually much higher (PCA, 1970).

Various procedures can be used to determine the TWA capacity of a block (BS 3921: 1985):

- Cold immersion in water (24 to 48 hours) after oven drying to constant mass
- Boiling test method (5 hours)
- Absorption under vacuum test

With the above methods, widely differing results can still be obtained (Bungey & Millard, 1996). It is reported that none of the three methods above can show any precise convergence (BS 3921, 1985). The results obtained from each of the three methods can be different, and neither proportional nor equivalent to one another (Neville, 1995). For this thesis, oven-drying followed by cold-immersion in water was found to be the most convenient (and easy one) to conduct. The method was also found to be fairly accurate and repeatable. It was therefore the only method used throughout for determination of the TWA capacity of both traditional and improved blocks. For comparison purposes, tests were also conducted on samples of other like materials found in the laboratory (fired clay bricks and concrete blocks). Brief details of the test method are described in Appendix R. For each test however, three specimen samples of each material under test were examined (Chapter 5). The TWA was calculated by taking the amount of water absorbed by a dried sample that had been immersed in water for a specified period of time (24 to 48 hours). Mean values obtained were taken as the total water absorption (TWA) of the sample. The result was expressed as a percentage of the original dry mass of the specimen to the nearest 0.1% of the dry mass. Details of all individual measurements recorded are presented in Appendix T(1) to T(5).

6.4.1 EFFECT OF VARYING THE STABILISER CONTENT AND COMPACTION PRESSURE ON THE TWA IN BLOCKS

Both traditional and improved blocks were examined. The mean values obtained are shown in figure 12 and in table 10. The latter shows the range of the extreme values found.

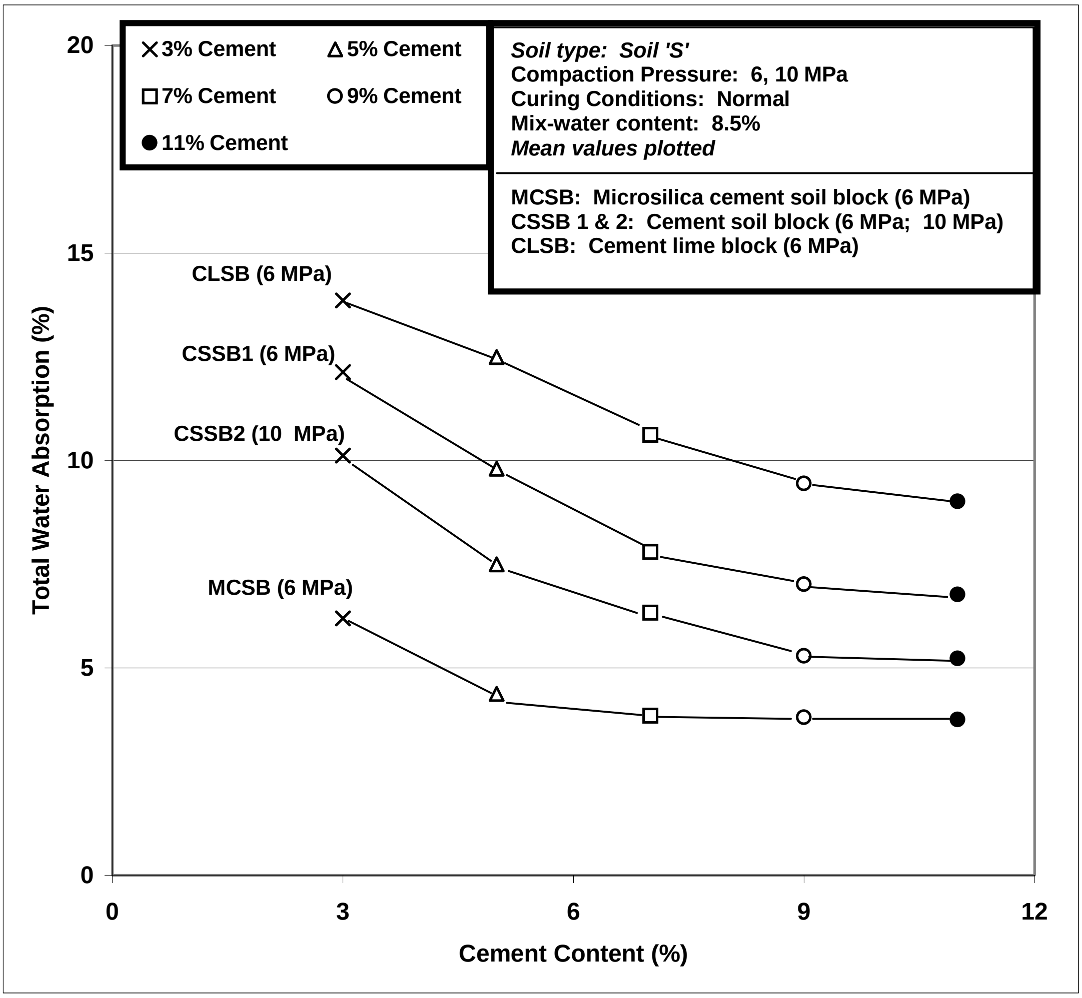


Figure 12: Effect of varying the stabiliser content and type, and compaction pressure on the TWA in CSBs. (University of Warwick, 2001)

Block Type	Compaction	Range of TWA Values		Comparison of values with those for improved blocks		Overall decrease in TWA
		OPC 3%	OPC 11%	OPC 3%	OPC 11%	
	MPa	%	%	%	%	%
Improved (OPC + microsilica)	6	6.19	3.75	-	-	40
Traditional (OPC + lime)	6	13.86	9.01	124	140	35
Traditional (OPC only)	6	12.13	6.76	96	80	44
Traditional (OPC only)	10	10.11	5.22	63	39	48

Table 10: Range of TWA values obtained

Figure 12 shows that a negative correlation exists between increase in cement content and total water absorption: coefficient of correlation values were -0.947 ($P = 0.014$) for traditional blocks, and -0.832 ($P = 0.080$) for improved blocks. From both figure 12 and table 10, improved blocks had the narrowest band of TWA results. They also absorbed considerably less water than traditional blocks. Generally, traditional blocks compacted at the same level absorbed about twice the amount of water absorbed by improved blocks.

There was a general decrease in water absorption with increase in cement content (and compaction pressure). The decrease was generally about 42% with variation in cement content from 3% to 11% (table 10). The trend was the same for all categories of blocks. Blocks of lower stabiliser contents were however found to absorb more water than those with higher ones. The reduction in absorption with increase in stabiliser content is progressive but diminishes. The absorption effectively ceases to reduce any further beyond certain cement content values in both traditional and improved blocks. These limits are 9% and 7% respectively. Beyond these limits, increase in OPC content does not result in any appreciable reduction in TWA. It is also worth noting that the limit of 9% appears to apply to all types of traditional blocks regardless of the moulding pressure used. The lowering of the limit to 7% from 9% in improved blocks can only be attributed to the inclusion of microsilica. Even beyond these points however, the block can still continue to absorb water.

The results also show that the TWA values obtained compare well with those of other like materials and with current recommended maximum values for CSBs. The recommended maximum is 15% (ILO, 1987). Although this value is neither absolute nor widely adopted by other researchers, it still serves a useful purpose. The experimental TWA values for improved blocks were on average considerably lower than this recommended value. The values obtained were favourable when compared with those of like materials (clay bricks 0 to 30%; concrete blocks 4 to 25%; calcium silicate bricks 6 to 16% (Jackson & Dhir, 1996)). According to BS 5628 Part 1, TWA values below 7% are regarded as being low, while those above 12% as high. All improved blocks tested would therefore be regarded as having low TWA values. The values for traditional blocks fall in between the two limits and would therefore be regarded as moderate.

The above results confirm that CSBs have the potential to absorb appreciable amounts of water and possibly retain it too. They also show that the use of CRMs can be a effective way of reducing water absorption. Moreover, they confirm earlier findings that improvement in the quality of a block is easily achieved by variation in stabiliser content and type. The correlation of TWA and other bulk properties are discussed in subsequent sections.

6.4.2 CORRELATION BETWEEN TOTAL WATER ABSORPTION AND DENSITY

The correlation between water absorption and dry density was examined. Figure 13 shows the plotted values from the measured points for both properties.

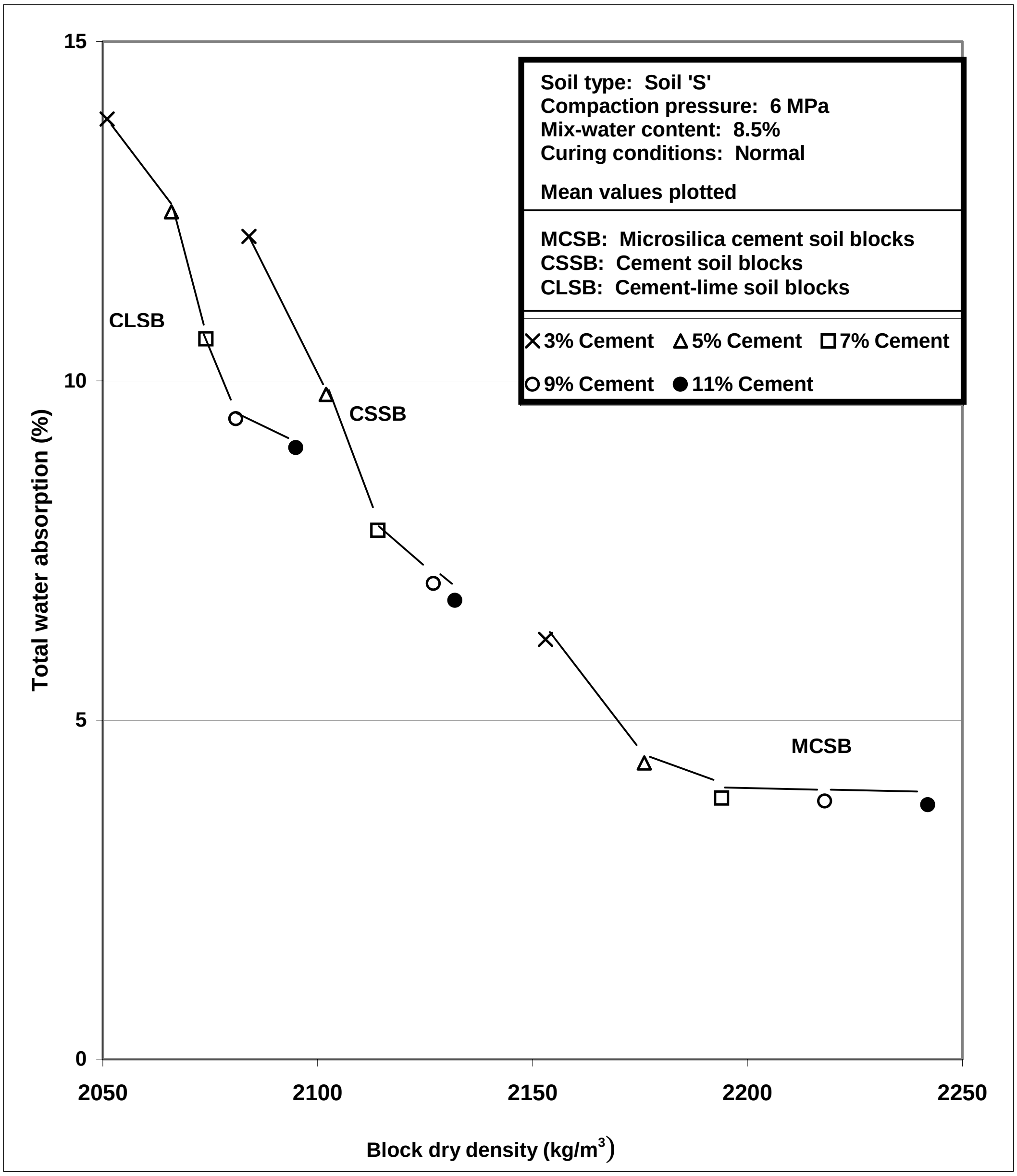


Figure 13: Correlation of TWA and BDD (University of Warwick, 2001)

As shown in figure 13, a negative correlation exists between TWA and BDD. The coefficient of correlation and P-values for both traditional and improved blocks were: -0.985 ($P = 0.002$) and –0.820 ($P = 0.089$) respectively. These values confirm that a strong negative correlation exists between the two bulk properties. Increase in the latter is likely to result in a decrease in the former. For example, in traditional OPC only stabilised blocks, increase in density from 2084 kg/m^3 to 2132 kg/m^3 (2.3% increase), resulted in an overall reduction in water absorption by 44%. Similar increase in density over the same range of cement contents in improved blocks resulted in a decrease in TWA of about 39%.

The results also show that for the samples tested, beyond a certain density value, no appreciable reduction in TWA can be found. The limiting density values correspond to matching cement contents beyond which a similar occurrence was noted in Section 6.4.1. The implication here is that no further increase in BDD would necessarily lead to continued reduction in TWA. The blocks can still be able to absorb water but at almost uniform amounts. Similar correlation was also found between WCS and TWA. Generally, the more a block was found to absorb water, the lower was its strength.

6.5 VOLUME FRACTION POROSITY

In this Section, the term porosity refers to the total amount of voids and pore structure within a block fabric (sand pores, gel pores, capillarity pores, entrapped air, entrained air, etc.) (Young, 1998). The concept of porosity has neither been well researched nor reported in CSB literature. Yet most bulk properties including strength and water absorption are believed to be a function of the total porosity of cement-based materials (Weidemann et al, 1990). The general link between porosity and quality has been widely reported in concrete literature (Neville, 1995). There is no good reason to expect that similar findings would not obtain in CSBs.

In addition to their volume, the shape and size of pores within a block can determine its bulk performance. The capillary porosity which is often the most predominant is believed to be a function of the water-cement ratio and the degree of hydration achieved (Sjostrom et al, 1996). The value of the latter can only increase as long as moisture is available to ensure the completion of hydration. Proper moist curing can therefore be a vital factor in influencing the volume fraction porosity of a block.

The total volume fraction porosity (TVP) in a CSB can be determined directly. This can be done by measuring the weight gain on saturation with water of an initially dry block after evacuation to remove air from the pore network (Jackson & Dhir, 1996). The water absorption is expressed as before in weight percent. The value of the water absorption may be converted to a volume basis porosity by using the following relationship:

$$n = \frac{(WA)\,\rho}{100\;\rho_w}$$

where n = volume fraction porosity
ρ = dry block density (kg/m^3)
ρ_w = density of water kg/m^3)
WA = water absorption (%)

A summary of the calculated total volume fraction porosity of CSB samples are shown in Appendix T. The relationship between these results and other bulk properties are discussed in the following sections.

6.5.1 *CORRELATION BETWEEN STRENGTH AND POROSITY*

The correlation between wet compressive strength and total volume porosity was examined. The mean values plotted are as shown in figure 14.

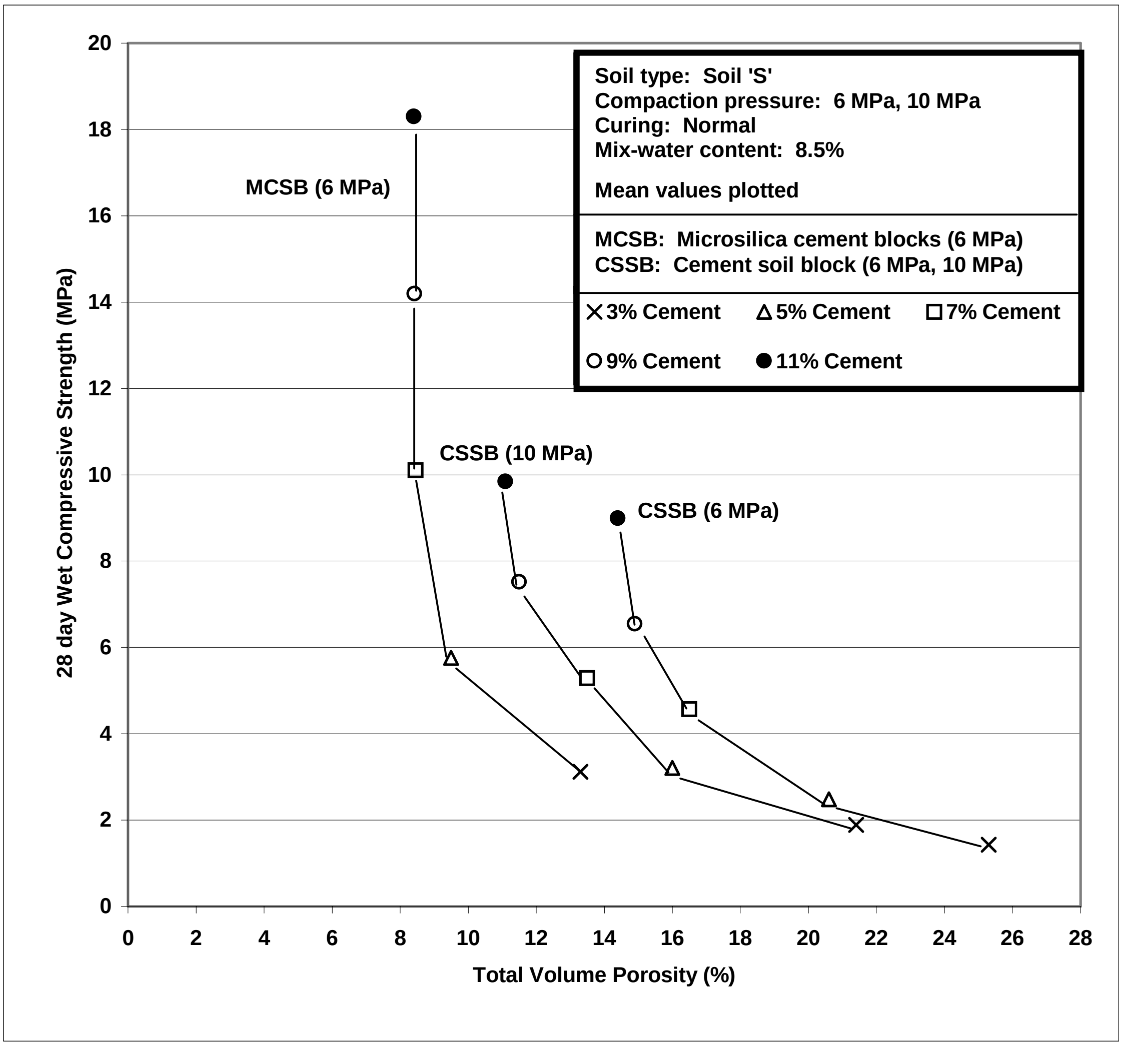


Figure 14: Correlation of WCS and TVP in CSBs (University of Warwick, 2001)

According figure 14, WCS and TVP are negatively correlated. Increase in porosity is accompanied by a decrease in strength. The coefficient of correlation and P values for traditional and improved blocks were: –0.905 (P = 0.035) and –0.771 (P = 0.127) respectively. A strong negative correlation therefore exists between the two bulk properties.

The total volume porosity values are lower in improved blocks than in their traditional counterparts (8.4% to 13.3% as compared to 14.4% to 25.3%). The pore filling effect of microsilica is likely to account for some of the difference between the two types of blocks. The lime plus OPC stabilised blocks exhibited the highest porosity (between 18.9% and 28.4%). The values for both categories of blocks however compare well with those of like materials. Materials with TVP above 30% are considered to be of high porosity (Jackson & Dhir, 1996). All the blocks examined during this research can therefore be considered to be of low porosity.

The decrease in compressive strength with increase in porosity can be partly explained as follows. The compressive strength of a block is limited by brittle fracture. It is therefore sensitive to individual flaws in the block sample under test. Discontinuities between solid phases in a block (due to the presence of voids and pore structure) constitute flaws in it. The higher the amount of voids, the weaker the block is likely to be. Large coarse soil fractions in a block can also create flaws in it. The combination of such large particles and voids in a block can make it more susceptible to brittle fracture failure.

6.5.2 CORRELATION BETWEEN DENSITY AND POROSITY

The above relationship was examined using the results obtained as before (shown plotted in figure 15).

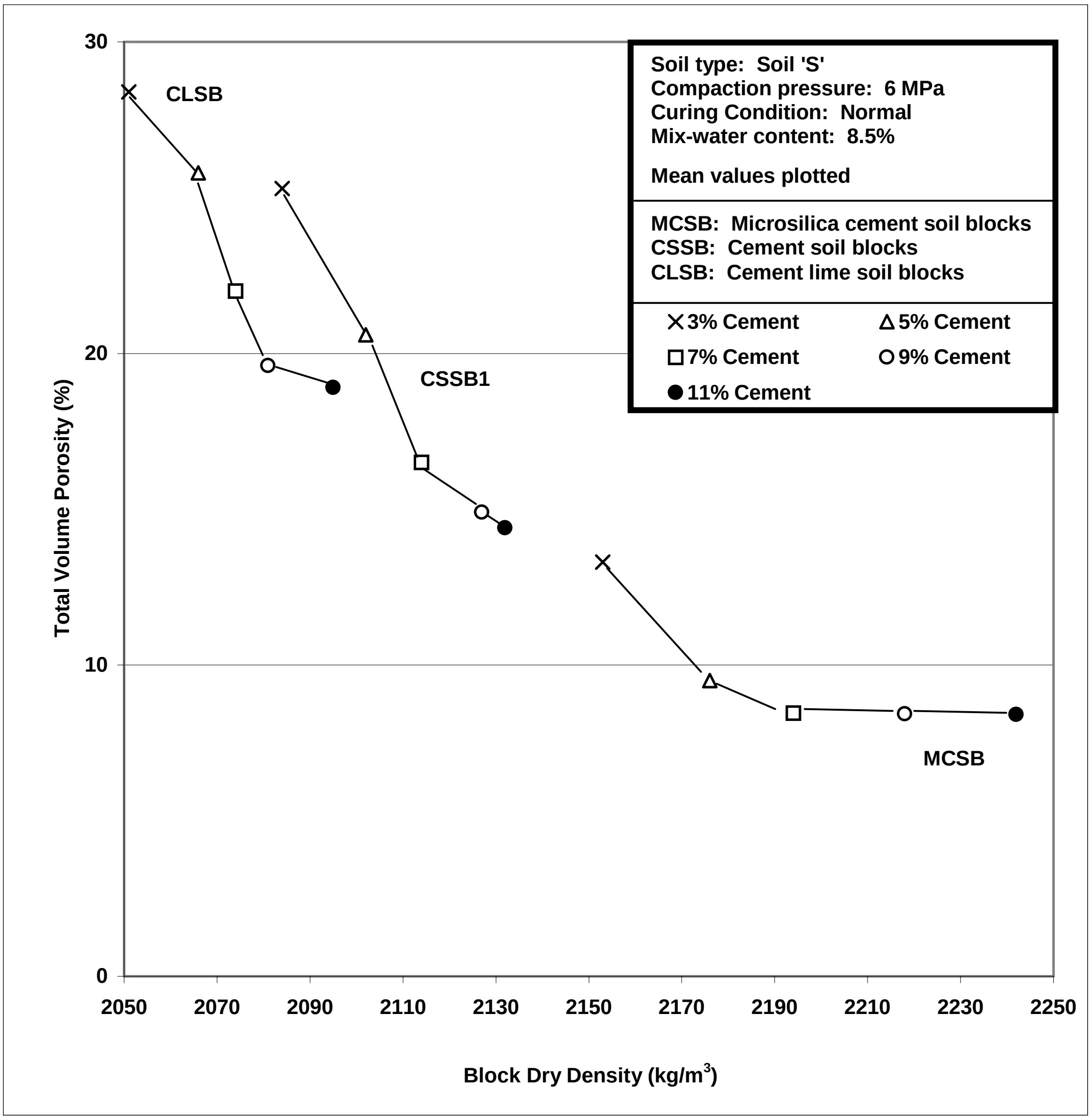


Figure 15: Correlation of BDD and TVP (University of Warwick, 2001)

Figure 15 shows the general correlation existing between the two bulk properties. The coefficient of correlation and P-values are as follows: –0.984 (P = 0.002) for traditional blocks, and –0.935 (P = 0.020) for improved blocks. These statistical values confirm that a very strong negative correlation exists between the two bulk properties. Increase in dry density is associated with a decrease in porosity for all blocks examined.

For example, in traditional cement only stabilised blocks, increase in density from 2084 kg/m^3 to 2132 kg/m^3 (3% and 11% cement contents respectively), resulted in an overall reduction in porosity of about 43%. Similar trends were shown in the improved blocks examined. Reduction in porosity by 37% was found to result from an overall increase in density of 4.1%. These blocks were generally denser than their traditional counterparts. Increased density is accompanied by closer packing of the solids in a block. The closer the packing, the less the amount of voids in a block. It was however also found that further increase in density beyond a certain value did not result in any appreciable reduction in porosity.

6.6 CONCLUSION

From the results discussed in Chapter 6, a number of conclusions can be reached.

The *wet compressive strength* of a block is one of its most valuable properties. It is influenced by the following factors: cementitious matrix (water cement ratio and degree of hydration), degree of compaction, state of moisture, temperature, age and type of coarse soil fraction present. The strength of the cement hydrates, and the bond between them and the coarse soil fraction accounts for most of the strength in CSBs.

It was found in Chapter 6 that the WCS of both traditional and improved blocks increased with increase in cement content and compaction pressure. The inclusion of microsilica in improved blocks was found to significantly improve their strength. The use of microsilica was also found to reduce the gap between the mean WCS and the DCS in blocks. The reduced gap of between 12% and 26% in IPD blocks is comparable to those obtaining in concrete products (9% to 25%). Hitherto, the same gap in CSBs was between 40% and 120%. The considerable reduction in the gap can be associated with an increase in bonding strength between the phases and particles in the block. Use of microsilica is therefore beneficial for improvement in CSB strength and by implication its durability.

It was also found that delays in compaction after wet mixing of soil and cement resulted in an appreciable reduction in the strength of a block. Delays of up to two hours resulted in loss of strength of about 41% in traditional blocks. Blocks compacted within 20 minutes of wet mixing were about 27% stronger than blocks compacted after 45 minutes of delay. Similar trends are expected to occur in improved blocks. These findings confirm earlier work by other researchers. It is therefore recommended that smaller batches of wet mixes that can be compacted within 30 minutes (instead of one hour) be planned for. Compaction of wet mixes more than 60 minutes old are not recommended.

It was also found in Chapter 6 that the WCS of a block can be affected by the method of curing used. Blocks cured under normal conditions were about twice stronger than those cured under open exposure in the laboratory. Those cured under continued moist cover were about three times stronger than exposed blocks. Moreover, blocks cured by full immersion in water (100% relative humidity) were about six fold stronger than those cured exposed. Improved curing conditions were found to be linked to higher strengths in CSBs. This can be partly due to the higher degree of hydration achieved by the OPC in the block (continued presence of moisture). Proper curing conditions are therefore critical if CSBs are to achieve high strength. It is recommended that proper curing guidelines be included in CSB production codes.

The *density* of a block is another valuable indicator of its bulk quality. Its value depends on the degree of compaction used, the form of the block, and the size, grading and density of its individual constituent materials. The higher the density of a block, the better is its overall performance expected to be. It was generally found that traditional blocks were less dense that their improved counterparts. Increase in cement content resulted into increase in density for both categories of blocks by about 3%. Increase in density due to increase in compaction pressure of about 70% only resulted in an increase in density of about 1.2%. The use of CRMs, and increase in cement content appear to be more economic ways of achieving higher densities in CSBs. The experimental density values obtained were also found to be above the recommended minimum of 2000 kg/m^3 (by about 9%). The pore filling effect, increased homogeneity, improved bonding and reduced voids due to the use of CRM was thought to be responsible for the marked increase in density of improved blocks.

The BDD was also found to be strongly correlated to other properties such as WCS, TWA and TVP. Generally, more denser blocks were found to perform better in all the complimentary tests done. Blocks which are too dense might however prove difficult to lay, and costly to transport. It is recommended that the maximum weight of a block should not exceed about 8,500 kg.

The *total water absorption* in CSBs is also an important bulk property that can be used for routine quality checks as well as for their classification. It was found that the TWA of traditional blocks ranged between 6.76% and 12.13%. Comparable figures for improved blocks were considerably lower than these values. The use of CRMs therefore results into a marked reduction in TWA. It was also found that the TWA decreases with increase in cement content and compaction pressure. However, the decrease is gradual and more pronounced at the lower cement contents than at the higher ones. Beyond a certain cement content however (7% in improved blocks and 9% in traditional blocks), increase in cement content did not result into any further appreciable decrease in TWA. All blocks made for experimental tests in the course of this thesis were found to have TWA values below the recommended maximum value of 15%. It was also found that TWA was strongly correlated with BDD, WCS and TVP.

The *total volume porosity* of a block also represents an important bulk property. It was found that porosity of block samples decreases with increase in cement content. Porosity values for traditional blocks ranged between 14.4% and 28.8%, while those for improved blocks between 8.4% and 13.3%. It was found that traditional blocks were generally more porous than their improved counterparts. Porosity was also found to be negatively correlated to strength and density, but positively correlated to water absorption. High porosity in a block is thought to reduce strength due to the presence of flaws and discontinuities in its fabric. All blocks made were found to be of low porosity, i.e. less than 30%.

From the preceding conclusions, the objectives of Chapter 6 were fully met.

CHAPTER 7

SURFACE FEATURES AND PERFORMANCE

7.1 INTRODUCTION

The surface of any building material is one of its most important features. For materials such as CSBs, the quality of their surfaces can affect their durability (Hughes, 1983). The block surface forms its first line of defence against deterioration agents likely to come into contact with the material during its service lifetime. As mentioned earlier in the thesis, the bulk of a block is its least compacted zone and is therefore in need of protection provided by a denser surface.

The deterioration mechanisms that can erode the surface of a block and expose its bulk are likely to lead to accelerated damage (Chapter 3 and 4). A good surface is therefore required if a block is to remain durable for the duration of its service lifetime. How a block surface can influence its performance depends on its surface properties. Properties thought to be affected by the quality of the outer part of a block include: surface wetting, adsorption, adhesion, abrasion, hardness and capillary effects (Young et al, 1998).

The objectives of this Chapter are to:

- identify microstructural features of block surfaces
- monitor the overall performance of the surface in conditions which simulate the main cause of surface deterioration. It was mentioned in Chapter 2 and

found in Chapter 4, that surface erosion was the most serious form of surface deterioration. The softening and abrasive action of water and the heating effects of high temperatures, are thought to combine to contribute to much of the mass loss from the surface of a block. The test method used in this Chapter is the Slake Durability Test. Its pioneering use for CSBs was found to be appropriate for laboratory testing owing to the rapid acceleration of surface erosion (ISRM, 1971).

The rest of this Chapter is presented in three sections, namely: thin-section microstructural features of block surfaces, monitoring the performance of block surfaces using the slake durability test, and conclusion.

7.2 THIN SECTION MICROSTRUCTURAL FEATURES OF CSB SURFACES

The performance of a block is closely linked to its microstructure (Houben & Guillaud, 1994). Awareness of such links has led to several recent advances being made in concrete research (Baker et al, 1991; Taylor, 1998). It was with this in mind that a similar approach was adopted for this research. After all, the two materials both develop their microstructure by solidification from solution formed as the cement particles in either material dissolve in water (Young et al, 1998). In concrete studies it has been found that the resulting microstructure controls most of its key properties, especially those associated with its durability, and it would be reasonable to expect that a similar happening would occur in CSBs.

Investigation method used

The scope of microstructural investigations were limited to the identification and description of the main surface features of blocks. Although other petrographic methods exist, two microscopic methods were considered, namely: examination of a prepared block surface specimen using reflected light and examination using light transmitted through a 'thin section' (Brandon & Kaplan 1999). The latter method was selected for use in this research. Its advantage is that it is also widely used in concrete research to identify mix components, defects types and even causes of defects (Taylor, 1998). To the knowledge of the author, this represents the first published petrological study of CSB like materials..

The CSB samples for microscopic examination were prepared as described earlier in Chapter 5. Several six month old samples, some made using 5% cement and the other using 9% cement plus 2.25% microsilica, were examined. The blocks were compacted at 6MPa and cured under normal conditions (wet followed by laboratory dry curing). Samples of dimensions 100 x 90 x 40 mm were thin-sectioned. Slices of these samples (and others not described here) were cut using diamond saws preceded by vacuum resin impregnation. The slices were then dried and again impregnated using low viscosity epoxy resins. The samples were then ground using standard petrographic procedures to a 30 μm thickness. Oil lubrication was used to avoid the dissolution of water soluble materials in the block. The thin-surface sections were then examined with a petrographic microscope. The examination was done under both plain polarised light and cross-polar light. Micrographs of the thin sections were then produced for analysis and interpretation. Appendix U shows the three sets of micrographs discussed in this thesis. Additional comments are also shown on the same Appendix.

Interpretation of the Results

The interpretation of micrographs remains a highly specialised field. What is described in this Section are key features discernible even by the casual observer. The main object was to identify the following phases and defects:

- general features
- calcium hydroxide (portlandite)
- unreacted cement residues
- cement hydrate phases
- free sand, silt and clay residues
- gross porosity
- microdefects
- possible causal links to surface properties

In terms of general features, the micrographs in Appendix U reveal the existence of an amorphous particulate composite structure of predominantly short range order. As would be expected, the spatial pattern seen throughout is not rotationally repeated symmetrically over the long range (like in concrete). The precipitates look like a collection of individual particles and phases that are fairly well agglomerated. Given the low amount of cement used (5%), it was not expected to find a continuous interlocking phase of OPC hydrates and embedded sand particles. However, such a continuity has been reported in fired bricks mainly due to the resulting mulite structure, and partly explains the marked difference in performance between such bricks and other comparable materials (Jackson & Dhir, 1996). Continuity is known to confer marked improvement in the properties of bricks. By using microsilica in improved blocks, an attempt was made to improve packing and continuity in the block microstructure. Although this does not come out quite clearly in the micrographs (only 2.25% by weight used), evidence from other tests suggest that considerable improvement in performance was achieved (Chapter 6 and Section 7.3 that follows). Nevertheless, the groundmass was far more detailed than in an OPC mortar.

CSBs contain more varied particles and phases than concrete. Distinguishable features observed in the micrographs were: fine gravel, sandy fraction, clay agglomerations, and cement hydrates phases (Appendix U(1) and U(2)). The amorphous but homogenous areas seen in the micrograph resemble C-S-H gels. However, with so little (5%) OPC present, one would indeed be hard pushed to find any technique that could detect the individual cement hydrate products. The micrographs also reveal evidence of portlandite in the sample (Appendix U(3)). Fewer than expected platelets of portlandite, characteristic of hydrated cement paste were present. Appendix U(3) shows a relatively large portlandite crystal, approximately 30μm across, embedded in the matrix. Modification of CBSs using microsilica is therefore justifiable to encourage pozzolanicity.

Normally, it should have been easy to detect unhydrated cement residues. These were however conspicuous by their absence. Despite this surprising finding, conglomerations of unreacted cement-like collections were evident during processing even though the block materials had undergone careful mechanical damp mixing. Similar collections were also observed on new surfaces of blocks that had been subjected to the slake durability test (Section 7.3).

Microdefects which should normally have been detectable at the cement hydrate and sand interface zone were not discernible in the micrographs. Instead, the micrographs show that the cement hydrates and coarse soil fractions are satisfactorily intertwined. Gross porosity was lower than expected, suggesting good compaction. It is unlikely that CSB surfaces obtained from field production sites would have had similar quality finish (Chapter 4). Instead, inclusions, cracks, point defects and production defects would have been more prominent. Overall the findings are encouraging as they indicate no fundamental defects in the material.

From the above findings it can be expected that the microstructure of a block can mediate some of its properties. Block properties likely to be sensitive to the nature of their microstructure can be referred to as being 'structure sensitive'. They are structure sensitive because of their dependence on gross porosity, grain size and level of bonding of the composite structure. Properties such as strength, dimensional stability, water absorption, permeability and durability are likely to be structure-sensitive (Young et al, 1998). Future research should be able to reveal causal links between a particular microstructural feature and a particular block property. Conversely, block properties such as thermal expansion, elastic moduli, specific gravity, etc., are likely to be structure-insensitive. This is because such properties vary only slowly with structural composition, particle sizes and microstructural variations.

The desirable qualities at the surface of a block are impermeability, non-reactivity and high-intergranular strength. These features are likely to be linked to the microstructure of the block surface, which is in turn determined by the processing methods used. By reducing voids in the fabric (microstructure) for example, pores can be reduced. By improving bonding (microsilica, high degree of hydration), contact can be improved. Such procedures could result in considerable surface resistance being offered by the block. An attempt to achieve such a surface is investigated experimentally in the next Section.

7.3 MONITORING THE PERFORMANCE OF CSB SURFACES USING THE SLAKE DURABILITY TEST

In this Section, the need for a new accelerated surface test for CSBs and the main features of the proposed test are discussed. The proposed test is the slake durability test (SDT) which was originally developed for evaluating the resistance of clay-bearing rocks to slaking, abrasion and heating (Eigenbrod, 1969; Chandra, 1970; Franklin et al, 1971; Gamble, 1971; ISRM, 1971; Franklin et al, 1971; Goodman, 1980). In the subsequent sections that follow (Section 7.3.1 to 7.3.5), the results of the application of the test to block samples made as described in Chapter 5 are presented.

Need for a new accelerated surface test for CSBs

Surface erosion has been identified as a major problem for CSBs (Chapter 2 and 4). Yet it has always been difficult to monitor the performance of CSB surfaces when they are subjected to wetting and the abrasive action of water (Ola & Mbata, 1990). Selection of experimental methods to evaluate the integrity of cured block surfaces have proved difficult in the past (Webb, 1988; Gooding, 1994). Of the current surface monitoring test methods documented (drip test, water spray test, brushing test, abrasion test, wet-and-dry cycling test, etc.), none has been without criticism (Houben & Guillaud, 1994). Further, none has gained universal acceptance and application. Current tests have been found to be simplistic, misleading, of no relevance to the main mode of surface erosion, or over-dependant on the competence of the operator. These tests have failed to be predictive enough, with the unfortunate result that substandard blocks were passed. Moreover, after the full curing period is reached, the tests become largely inappropriate. A method of test is required that can monitor the performance of a block irrespective of its pre-cured and post-cured age. An accelerated durability test that can be conducted from a few weeks of production to several weeks or years after production would be quite helpful (Baker et al, 1991).

CSBs like most other building materials are characterised by a wide variation in their surface and bulk properties. The most important surface property of a block is its ability to resist short and long-term deterioration due to wetting, abrasion and drying (Chapter 2). For example it was found in Chapter 4 that CSB surfaces that were satisfactorily protected survived the deleterious effects of rains, humidity and high temperatures. Where similar surfaces were left unprotected in similar conditions, premature defects in the form of surface roughening, pitting, cracking and erosion were found to occur. The defects were more excessive than those observed on the surfaces of comparable materials used under similar conditions. It was further found that the clearly distinguishable surface defects had negatively influenced the attitudes of many users (Chapter 4). CSBs were therefore regarded as being sub-grade and of lower durability than comparable materials.

Under normal conditions, a durable block would be required for walling for the service lifetime of a building. While some surface deterioration might be expected, the deterioration should not be so excessive that the functional requirements of the wall are adversely affected (normal load bearing, resistance to weathering, etc.). Where such phenomenon are expected, the block surface ought to be protected. Surface protection is unfortunately considered to be expensive since more costs are incurred. The erosion of block surfaces should therefore be more accurately and reliably forecasted early enough. This can only be done by using more appropriate and suitable accelerated tests than was hitherto possible. The key specification is that the required test should simulate more accurately the main mechanism of surface erosion as identified in Chapter 2 and Chapter 4. Such a test would be an invaluable asset for site and laboratory use. The author describes in this thesis one such pioneering test which was successfully used to monitor the surface performance of block samples made as described in Chapter 5. The surface monitoring test described is the slake durability test (SDT). Since both clay-bearing rocks and CSBs contain clay and rocky residues (sand, silt, fine gravel), use of the test for the latter was found to be quite appropriate. As will be discussed in subsequent Sections, use of the test was further extended to evaluate the performance of like materials such as fired bricks, concrete blocks and rock samples.

The Slake Durability Test

In this subsection, the main features of the test, the factors likely to influence the results, merits of the test and classification systems for evaluating the test results are discussed.

The *main features* of the SDT are briefly described here (full details are provided in Appendix V). The main test equipment used consists of a standard cylindrical drum 140 mm in diameter and 100 mm long (ISRM, 1971). The drum frame is enclosed by a standard 2 mm aperture sieve mesh which forms its wall. Four to five oven-dried prism block samples (about 30 x 30 x 30 mm) with a combined total weight of between 450 and 550 grams, are loaded into the drum. The drum is closed and the whole system rotated using an electrically operated motor at 20 revolutions per minute. The rotation is continued for 10 minutes through a bath filled to an assigned mark with ordinary tap water at 20°C. In the apparatus used, four drums all attached to the same motor were rotated simultaneously. Due to internal contact between the samples within the block, mixing and softening in water, attrition and abrasion from the mesh sieve walls, the surfaces of the block samples are continuously eroded. After 10 minutes of the generally slow rotation, the eroded block sample materials can be seen partly suspended in water, and partly settled at the bottom of each bath. The state of the slaking water, owing to the presence of suspended material, is clearly distinguishable by the amount and degree of discoloration observed. The partially eroded block samples are then removed from the drum, then re-weighed. The drying, wetting, abrasion and redrying cycles attempts to simulate the most severe environmental conditions that a block sample can be expected to endure in real service life.

The slake durability index is then defined as the percentage ratio of final to initial dry mass of the block samples (ISRM, 1971). The SDI for each sample to the nearest 0.1% was calculated using the formula:

$$SDI = \frac{M_f \times 100}{M_i}$$

where: SDI or (I_d) = slake durability index (%)
M_f = final mass (g)
M_i = initial mass (g)

The SDI value can be used to assess the degree of resistance offered by each block surface. Samples of traditional and improved blocks, concrete blocks, fired bricks and various rock samples were all tested in the same manner. Comprehensive results for all samples tested are shown in Appendix W(1) to W(7). The results are discussed in Sections 7.3.1 to 7.3.5.

The *factors considered likely to influence the results* were noted as: the equipment; sample dimensions; sample pre-treatment; duration of slaking; and chemistry of the slaking fluid. These are briefly discussed below.

- The *equipment or apparatus* used was the standard one. Its sieve mesh size (2 mm), drum size (140 x 100 mm) and speed of rotation (20 revolutions per minute) remained the same for all categories of blocks and other materials tested. If any of these had been varied, then comparison of the results would have been misleading.
- The *sample dimensions* selected were such that they would be approximately the same for all samples tested. The sample dimensions used were about 30 x 30 x 30 mm with a combined weight of between 450g and 550g. About four to five pieces of the same material were placed in the drum each time.
- *Sample pre-treatment* was kept uniform for all samples. They were all pre-oven dried, cooled under cover, and stored under cover. A similar procedure was followed after each test. In this way, a controlled and reproducible condition of moisture was ensured for all categories of samples tested. In a way, this could be equated to the intense drying of a block by the sun in the humid tropics. Drying has been though to accelerate the suction rate of blocks (Jackson & Dhir, 1996). In this test therefore, the very worst scenario has been applied to block samples since drying accentuates the deterioration process. Since SDI is based on the comparison of weights, before and after the test, oven drying was found to be essential for accuracy and repeatability. No similar durability test has been able to achieve this level of reproducibility (variance 0.118). Clearly, comparison of dry weights is more meaningful than comparison of wet weights. The latter would give varying inaccuracies since there is no known way of controlling initial and final water contents in the samples. Moreover, by drying, the moisture history of a block sample can be rendered useless so that previous storage conditions do not become an issue.

All samples can then be reduced to nearly the same level of zero moisture content at the start and finish of the test regime. Weighing of all cooled oven-dried samples were done using an electronic weighing scale with a display.

- *Duration of slaking* was maintained for all samples at 10 minutes (± 1%) without exception. A stop clock was used in addition to an electronic wrist alarm watch. The duration used is also the standard recommended period. If shorter durations had been opted for, the potential for errors was likely to be high. It would have for example been difficult to discriminate between any two highly durable blocks within a much shorter time. Even in actual service conditions, deterioration requires a period of initiation, followed by progression. Errors associated with timing of the test would have also contributed to poor results. Longer durations on the other hand, would have caused weaker or less durable blocks to show a 100% mass loss (or zero durability). This would have defeated the primary purpose for the test which was simply comparative and predictive.
- *Chemistry and nature* of the slaking fluid were also considered. It was found that use of distilled water and Coventry laboratory tap water at 20°C did not produce significantly dissimilar results. The use of cold tap water was therefore adopted for all test samples. Use of fluids other than water would most likely affect the results. Since it is the effects of rainwater that were being simulated, it was found not to be necessary to pursue this factor any further.

All the above factors were kept the same for all samples tested. These specifications are recommended for future similar tests.

The *merits* of using the SDT are associated with its extreme severity on the one hand,

and its simplicity on the other. The slake durability test aims at accelerating weathering to a maximum by combining the processes of slaking, abrasion and drying. During the test, as block surfaces are eroded, the new surfaces which emerge are exposed to further similar treatment. The test can therefore be said to be a very severe accelerated surface test. The more severe the test, the better even if such a test might appear to exceed the worst possible weathering conditions which a block is likely to get exposed to in actual practice. The SDT is likely to give a reasonable indication of future service behaviour of a block over time. The test measures within a much shorter time the durability behaviour of a block sample by attempting to reproduce outdoor conditions. This enables the durability of a block to be assessed within a much shorter time than would have been possible under actual conditions of use. Some short term significance can be derived from the ensuing results.

The SDT was also adopted as the main surface test due to its other several attractions over existing methods (rain erosion test, abrasion test, wet and dry cycling test, brushing test, etc.). In any case all these current tests, as mentioned earlier, still remain non-standardised and fragmented. The strong points in favour of the SDT over other durability evaluation methods can be summarised as follows:

- Simplicity
- Controllability
- Reproducibility
- Accuracy
- Reliability
- High speed and practicality
- Timeless capacity (blocks of any age)

Moreover, the SDT was found to be capable of causing significant mass loss from block surfaces of any age. All other current test methods are valid only for blocks of a certain pre-cured age. Through the SDT it was also possible to deduce the degree of alterability of a block surface. This was found to be linked to a quantitative index. As an index test, the method was found to be helpful in comparing not only one block with another, but also CSBs with other like materials. A test similar to this was not available in the past for use with CSBs. While the SDT test might not yet be able predict the rate of surface deterioration, it is nevertheless more indicative than other existing methods. In any case, the prediction of the rate of surface deterioration has to take into account all factors other than slaking, abrasion and drying that the test attempts to simulate. Further, as an index test, the SDT represents a compromise between simplicity and precision (Gamble, 1971). Most current durability tests are so complicated and delinked from the main surface deterioration mechanism that the interpretation of results is difficult. For each mechanism or group of similar mechanisms, other durability tests should be devised. Use of the test to compare the performance of traditional and improved blocks are discussed in subsequent Sections of this Chapter. It is recommended at the end of the this Chapter that a modified hand operated version of the SDT apparatus be devised for field use.

The *classification and grading* of SDT results is based on existing standards for rocks. In the present classification system, six classes of durability are provided for. These, together with the proposed recommended classes and grades for CSBs, are shown in table 11. Unequal subdivisions have been used. This might be more useful particularly for the more durable blocks. Most well made blocks might have 'extremely high' SDI (i.e. they slake to a negligible extent). In such cases, smaller subdivisions are more helpful in reflecting the slight differences in resistance to breakdown.

S/N	Author (Year)				Proposed Classification for CSBs		
	Gamble (1971)		Franklin & Chandra (1972)				
	Classification	SDI(%)	Classification	SDI(%)	Classification	SDI(%)	Grade
1	Very high durability	> 99	Extremely high	95-100	Extremely high	95-100	A
2	High durability	98-99	Very high	90-95	Very high	90-94	B
3	Medium high durability	95-98	High	75-90	High	75-89	C
4	Medium durability	85-95	Medium	50-75	Medium	50-74	D
5	Low durability	60-85	Low	25-50	Low	25-49	E
6	Very low durability	60 <	Very low	0-25	Very low	0-24	F

Table 11: Current classification systems for slake durability index in clay-bearing rocks and proposed standards for CSBs
(Adapted from Gamble, 1971; Franklin & Chandra, 1972)

The test results for all samples tested are shown in Appendix W(1) to W(7). The results when compared to the range of classes shown in table 11 represent a valuable indication of their resistance to surface erosion. Blocks giving low durability index should be investigated further to determine whether adequate processing safeguards were followed. The test results can be used in several ways:

- as an aid to block classification
- for selection of blocks for particular applications
- for quality control during production
- for prediction of the surface performance of a block

- for selecting suitable production equipment

The SDT therefore offers a new possible quantitative method of discriminating between various types of blocks. The proposed boundaries shown in table 11 are tentative and can be reviewed on the basis of results from future research or from those based on service record and experience. The use of the SDT is likely to ensure that block durability is no longer neglected in favour of other properties such as strength, whose values can be determined quantitatively (Lunt, 1980; ILO, 1987; Rigassi, 1995).

7.3.1 Effect of Varying the Stabiliser Type on the Slake Durability Index of CSBs

The effect of varying the cement content on the slake durability index of CSBs were investigated experimentally. All CSB samples were prepared in accordance with the descriptions given in Chapter 5. The mean values of the test results for blocks compacted at 6 MPa are shown in figure 16.

The mean values used for plotting the graph shown in figure 16 are tabulated for comparison purposes and for durability classification (table 12).

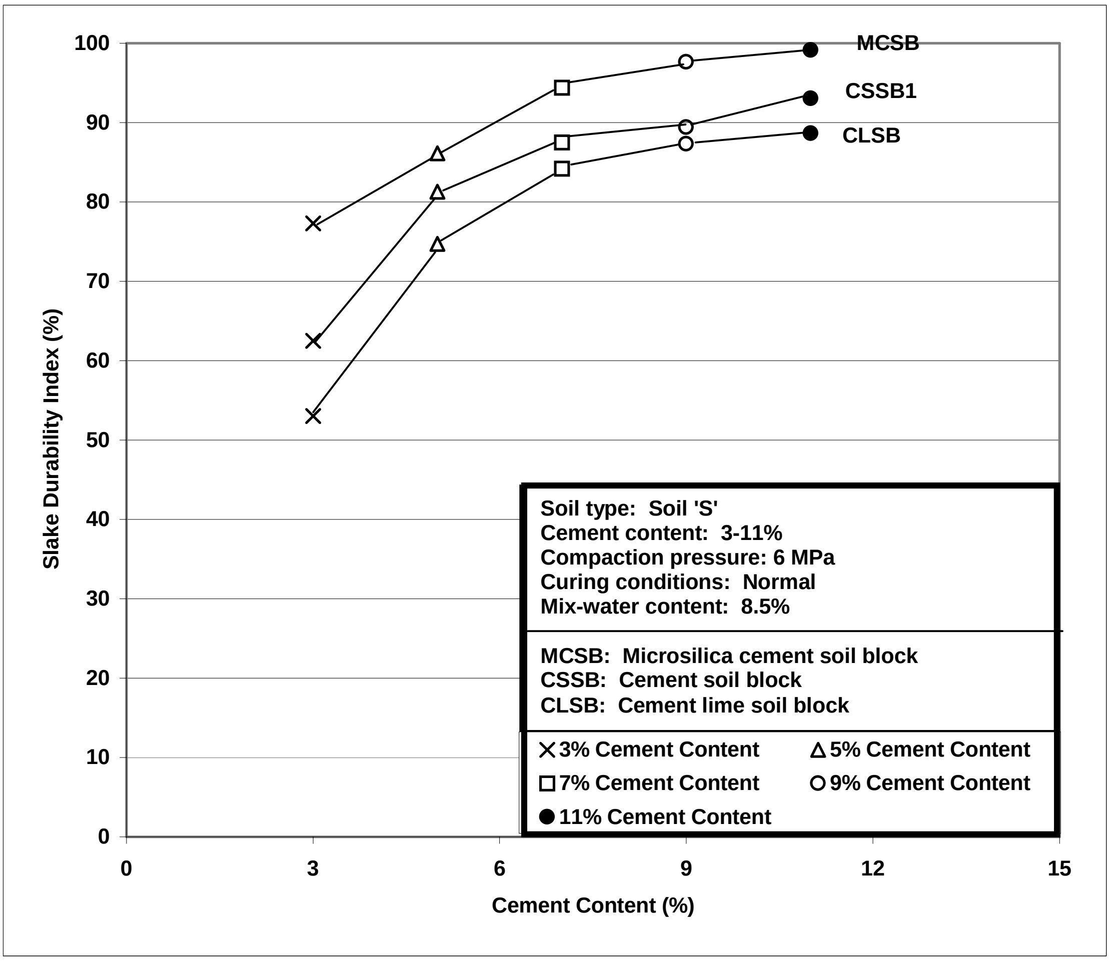


Figure 16: Effect of varying the cement content on the SDI values of improved and traditional blocks (University of Warwick, 2001)

SN	Material	cc (%)	Mass Loss (%)	SDI (%)	Durability Classification		
					Existing (Franklin & Chandra, 1972)	Proposed	Grading
1	MCSB	11	0.9	99.1	Extremely high	Extremely high	A
2	MCSB	9	2.4	97.6	Extremely high	Extremely high	A
3	MSCS	7	5.6	94.4	Very high	Very high	B
4	MCSB	5	13.9	86.1	High	High	C
5	MCSB	3	22.7	77.3	High	High	C
6	CSSB	11	7.0	93.0	Very high	Very high	B
7	CSSB	9	10.6	89.4	High	High	C
8	CSSB	7	12.5	87.5	High	High	C
9	CSSB	5	18.7	81.3	High	High	C
10	CSSB	3	37.5	62.5	High	High	D
11	CLSB	11	11.4	88.6	High	High	C
12	CLSB	9	12.7	87.3	High	High	C
13	CLSB	7	15.8	84.2	High	High	C
14	CLSB	5	25.3	74.7	Medium	Medium	D
15	CLSB	3	47.0	53.0	Medium	Medium	D
16	FBS	-	0.2	99.8	Extremely high	Extremely high	A
17	RBS	-	1.7	98.3	Extremely high	Extremely high	A
18	CBS	12-18	3.4	96.6	Extremely high	Extremely high	A

Table 12: SDI results for various samples tested and their durability classifications (University of Warwick, 2001). Abbreviations as before.

The results confirm that mass losses occur in CSBs when they are subjected to continued wetting, abrasion and drying. It was found that loss in mass in traditional blocks were higher than those in improved blocks. Table 12 shows the SDI values of the various materials tested and the range of values obtained. As can be seen, improved blocks with 7% and more cement content performed almost as well as FBS, RBS and CBS (all grade A). At the 9% cement level and above, improved blocks performed better than concrete blocks. During the test, it was found that one of the three test results from both fired brick samples and improved block with 11% cement content achieved 100% SDI values (Appendix W(1) – (7)).

According to the tentative classification system shown in Table 12, traditional blocks with cement content 5% and above can be regarded as having high durability (grade C and better). Improved blocks of similar cement content can be regarded as having high durability, although the SDI value was tending towards the very high durability classification levels. Even the 3% cement content improved blocks were found to be of high durability (grade C). This SDI surface test has been successful in grading blocks according to their degree of resistance to weakening. The findings clearly confirm that the test can be used for classification of blocks, irrespective of their storage and production history.

The graph in Figure 16 also shows that a strong correlation exists between increase in cement content and the slake durability index of blocks. For all block samples tested, it was found that increase in SDI due to increase in cement content was more pronounced at the lower stabiliser content levels than at higher ones. For example, in the traditional (OPC only) blocks, increase in cement content from 3% to 7% was accompanied by a matching increase in SDI of about 40%. Further increase in cement content from 7% to 11% resulted into a lower increase in SDI of about 6%. A similar trend was found with improved blocks where the corresponding increases over the same ranges of cement contents were 22% and 4.9% respectively. In both cases a further increase in cement content beyond the 7% level did not result in any appreciable increase in SDI.

The test results also show that most improved blocks were comparable to rocks. Rocks are known to be almost impermeable and are of high inter-granular strength. The fact that improved blocks were found to be comparable to rocks confirms that the use of microsilica can considerably improve bonding in blocks. The loss in mass as evidenced in all blocks implies that under certain environmental conditions, surface protection measures should be considered. As mentioned earlier, this could take the form of external render, low-roof overhangs, skirting plaster, thin-surface coating and thin-surface enriched layering of blocks. Where enriched surface layering is considered, costs could be saved by either using interlocking blocks, frog-bedded blocks, or hollow blocks.

7.3.2 EVOLUTION OF DURABILITY WITH CURING AGE

The SDT was also used to monitor the development of surface resistance with curing age in CSBs. The samples tested were all stabilised with 5% cement and compacted at 6 MPa. They were tested at 7, 14, 21, 28 and 56 days. The mean values of the results obtained are shown in Figure 17.

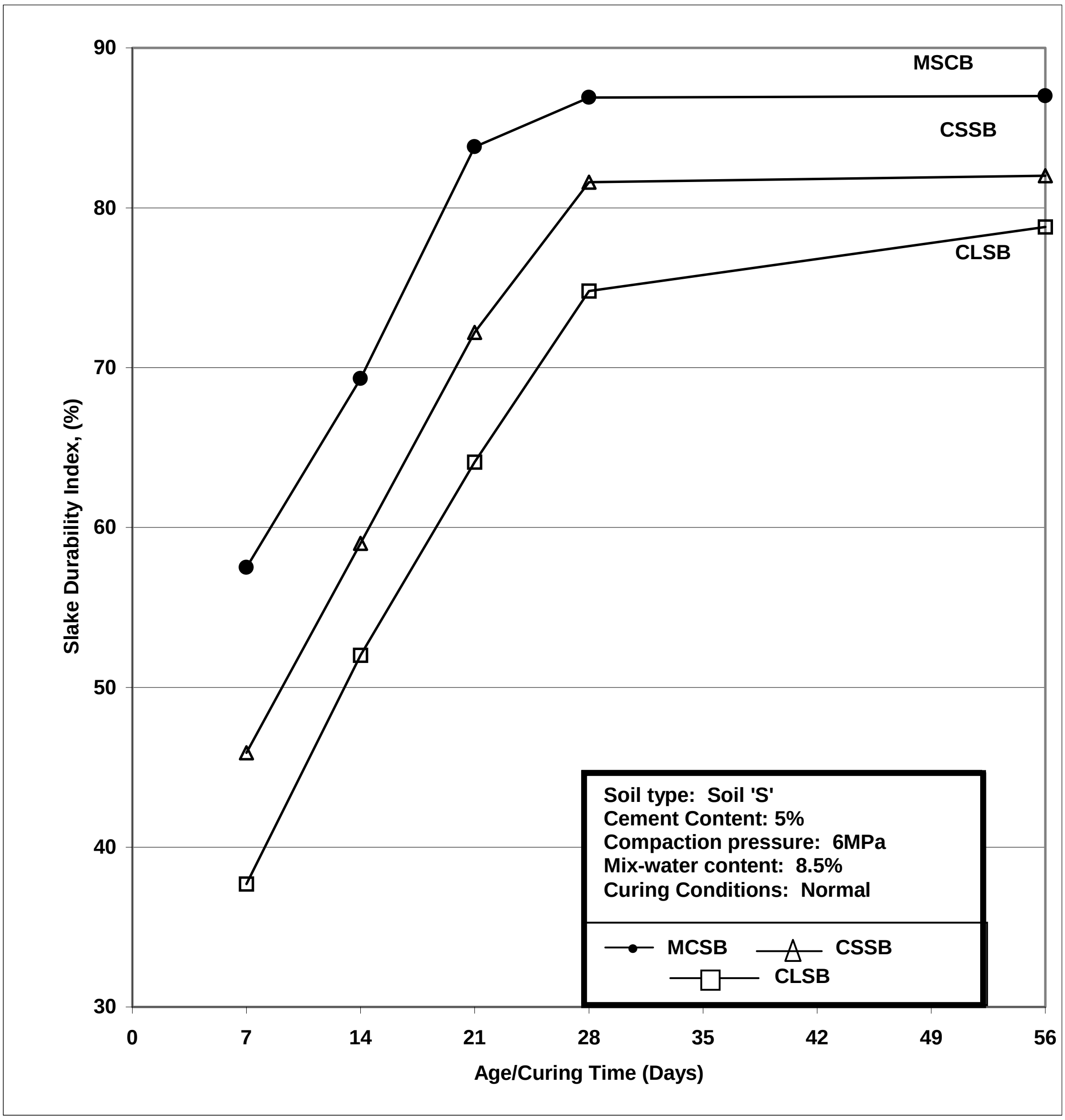


Figure 17: Evolution of SDI with curing age in traditional and improved blocks. (University of Warwick, 2001)

As expected, the results in figure 17 show that for both categories of blocks, increase in curing age is accompanied by an increase in the SDI value of a block. The increase in all cases is more pronounced before the 28th day after production than later. Except for blocks stabilised with both cement and lime, the increase in SDI value after 28 days was not appreciable. Table 13 shows a summary of the values obtained.

Age/Time	Slake Durability Index					
days	%					
	MCSB	R	CSSB	R	CLSB	R
7	57.5	1.0	45.9	1.0	39.7	1.0
14	69.3	1.2	59.0	1.3	52.0	1.3
21	83.8	1.4	72.2	1.6	64.1	1.6
28	86.9	1.5	81.6	1.8	74.8	1.9
56	87.0	1.5	82.0	1.8	78.8	2.0

Key: R = ratio (Age (SDI/7 days value)

Table 13: SDI values for various CSBs at different curing periods (all 5% cc, compacted at 6MPa).

The SDI values at 28 days for traditional blocks were about 1.8 times those at 7 days. The comparable ratio for improved blocks was only 1.5. Improved blocks were found to have gained strength more rapidly than traditional counterparts. Other ratios are also shown in table 13. These results show that SDI can also be used as a quick predictive test for gain in strength over time during curing. This can be a very useful surrogate test to identify quality problems in blocks at a very early age after production. To the knowledge of the author, this is the first published finding of such results.

7.3.3 *CORRELATION OF SLAKE DURABILITY INDEX AND WET COMPRESSIVE STRENGTH IN CSBs*

The above relationship was investigated from the results discussed earlier (Chapter 6). The mean values for SDI and WCS are shown in Figure 18.

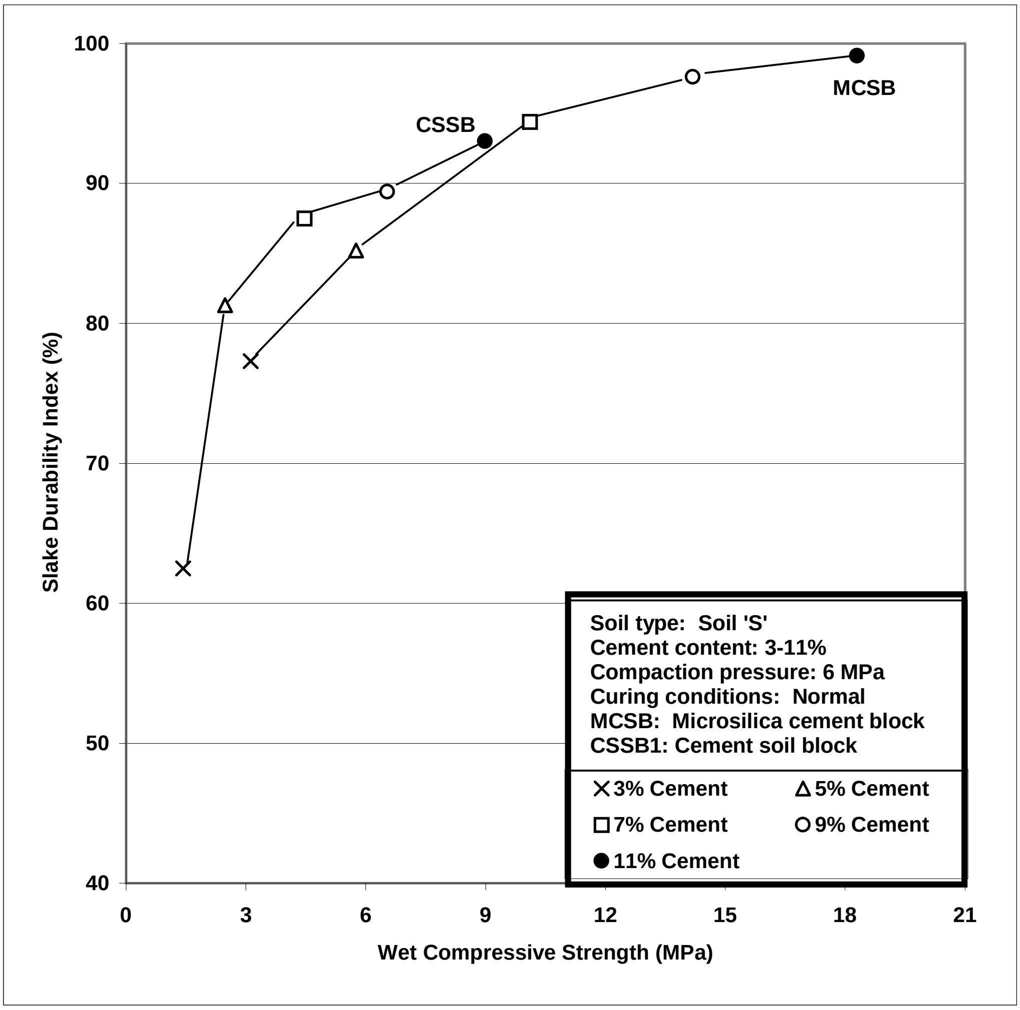


Figure 18: Correlation of slake durability index and wet compressive strength in CSBs (University of Warwick)

The graph in figure 18 shows a general positive correlation existing between SDI and WCS in both categories of blocks. The correlation coefficient for traditional blocks is

0.846 (with a two tailed significance of 0.71). The equivalent correlation coefficient for improved blocks is 0.938 (significant at the 0.05 level (2-tailed)). These non-parametric correlation coefficient values are all above 0.5 and approaching 1. They confirm that a very strong correlation exists between SDI and WCS (28-day) in stabilised blocks. The correlation is stronger in improved blocks than in traditional ones. If all points were to lie on the same curve, then WCS and SDI are surrogates for each other. The correlation is likely to remain valid only for homogenous blocks and not (for example) surface enhanced blocks. The SDT is a better test since it is more related to the surface resistance of a block, and is much easier to perform than the WCS test.

7.3.4 *Correlation of Slake Durability Index and Total Water Absorption*

The association between SDI and TWA in both categories of blocks were examined. The mean values are shown plotted in figure 19.

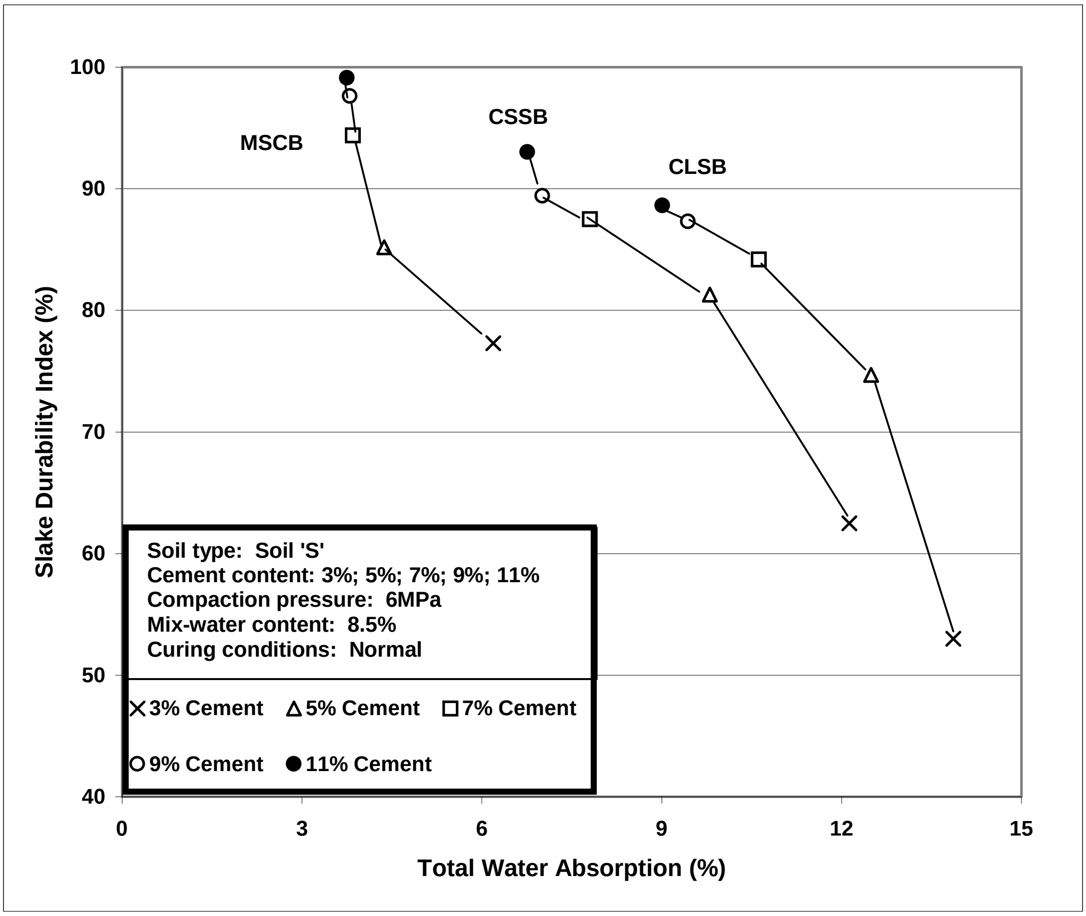


Figure 19: Correlation of Total Water Absorption and Slake Durability Index in CSBs (University of Warwick, 2001)

The results in figure 19 show that a general negative correlation exists between SDI and TWA in the blocks tested. The correlation coefficient for traditional blocks is -0.975 (with a two-tailed significance of 0.005). The correlation between SDI and TWA is significant at the 0.01 level (2-tailed). Similarly, the matching correlation coefficients for improved blocks is –0.939 (with a two tailed significance of 0.018).

Again the correlation is significant at the 0.01 level (2-tailed). These confirm a strong negative correlation between SDI and TWA. The finding implies that the higher the surface resistance, the lower the water absorption. This is a very desirable relationship in CSBs. The results show that both SDI and TWA can valuable indicators of the durability of a block.

7.3.5 CORRELATION OF SLAKE DURABILITY INDEX AND DENSITY

A plot of the mean values of SDI and BDD for the two categories of blocks are shown in figure 20.

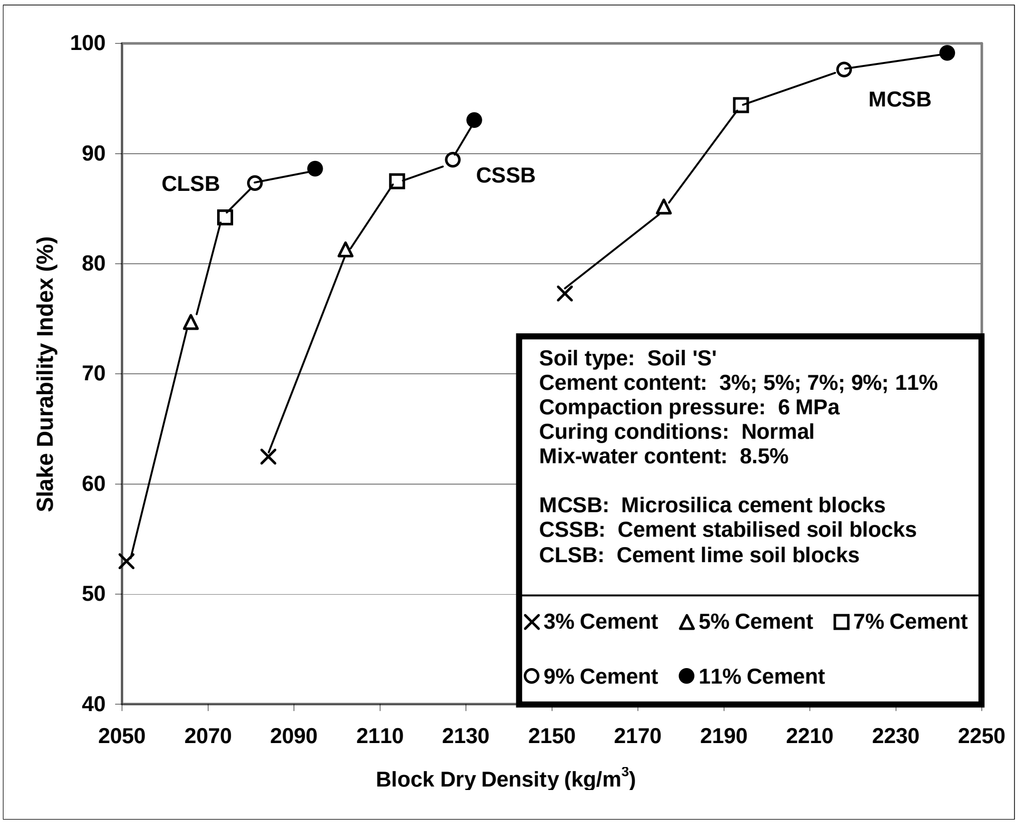


Figure 20: Correlation of Slake Durability Index and Block Dry Density (University of Warwick, 2001)

The graph in figure 20 shows a general positive correlation existing between SDI and

BDD. The correlation coefficients for traditional blocks is 0.953 with a (2-tailed significance value of 0.005). The correlation between the two properties is significant at the 0.05 level (2-tailed). The equivalent correlation coefficient for improved blocks is 0.944 (with a two-tailed significance of 0.016). The correlation is significant at the 0.05 level (2 tailed). Both values confirm a strong correlation between SDI and BDD. An increase in density can be expected to be accompanied by an increase in the durability of a block. The denser the packing of particles and phases in a block, the stronger and therefore more durable it is likely to be. Density is therefore a valuable indicator not only of strength but also of durability in blocks.

Increase in SDI with increase in density appears to be greater in traditional blocks than in the improved ones. Increase in density of about 2.3% is accompanied by an increase in SDI of about 49%. While increase in density of 4% over the same range of increase in cement content in improved blocks results into an increase in SDI of only 28%. So the denser the block, the less is the increase in SDI, but the higher is its resistance to surface abrasion.

7.4 CONCLUSION

From the discussions in the preceding Sections of Chapter 7, a number of general conclusions regarding the following key areas can be made .

The *surface microstructural features* of block samples as observed confirm the existence of an amorphous particulate composite, of predominantly short range order. The matrix shows sand and silt in a highly textured groundmass. The porosity was lower than expected indicating good packing possibly due to the compaction used. The groundmass was homogeneous, with some clayey inclusions seen in the 100 μm range. There was hardly any difference between the microstructure of the surface and bulk. Fewer than expected platelets of calcium hydroxide were present, with no CH precipitation in voids. Their presence justifies use of microsilica to promote pozzolanicity, and thus development of a secondary binding product. Generally no fundamental defects were observed in the material. It can be concluded that the method used is promising, and should be extended to examine samples from CSB production sites in future.

The *slake durability test* was found to have great potential in evaluating surface performance of various block samples. The test procedure was found to be more simple, controllable, reproducible, accurate, reliable and speedy. Moreover, it can be applicable to blocks of any age or stage of curing. The SDI values obtained could be satisfactorily compared with values from other like materials (rocks, concrete, fired bricks, etc.).

From the discussions in Chapter 7, a tentative classification and grading system for potential use in discriminating CSB samples is recommended. The classification is based on six levels of slake durability index SDI, namely: A = extremely high (95-100%); B = very high (90-94%); C = high (75-89%); D = medium (50-74%); E = low (25-49%); and very low (0-24%). While grade A represents blocks of extremely high durability, grade F represents blocks of very low durability (equivalent to unstabilised blocks). Blocks of low and medium durability can be investigated further to identify any production inadequacies.

While all previous tests relied on the veracity of an operative, and were clearly delinked from simulating the main mechanism of surface deterioration in CSBs, the SDI test is independent, and accurately approximates surface deterioration by wetting, abrasion and drying. The test is therefore strongly recommended for adoption and use in testing CSB samples of all backgrounds and ages. Its modification for manual use on block production sites is highly recommended.

It was found in Chapter 7 that increase in cement content resulted into a similar increase in the SDI value of all block categories. It was also found that all stabilised blocks and most comparable materials, were vulnerable to mass loss when subjected to continued wetting, abrasion and drying. Traditional blocks were found to be less resistant than matching improved blocks. The former lost between 12% and 45% more mass than the latter when both were tested under similar conditions.

At the range of interest (5% cement content), traditional blocks were found to have SDI values above 75%, and can therefore be classified as having high durability (grade C). Improved blocks of the same category were found to have very high durability (SDI above 90%). Unstabilised blocks were found to be of very low durability classification (0-24%). These are not recommended for use in building. The results also showed that the majority of improved blocks were comparable to rocks, concrete blocks and fired bricks (SDI > 90%). It can be concluded that the inclusion of microsilica in these blocks effectively increased the bond strength between the particles in the block. The approach therefore offers great potential for strengthening block surfaces and increasing their resistance against rain erosion.

It was also found that SDI values were positively correlated to compressive strength and density, but negatively correlated to water absorption in blocks. The SDI value is therefore a valuable indicator and surrogate measure of strength, density and water absorption in blocks. Use of the index can be favourably extended to compare the performance of other like materials.

Lastly, it was also found in Chapter 7 that with increase in curing age, a corresponding increase in SDI value of the block was recorded. The increase was uniform but more pronounced before the 28th day than after. The SDI value at 28 days was higher than that at 7 days by 51% in improved blocks, and by 88% in traditional blocks. Similar levels of change in strength with curing age have been reported in the literature. The SDI can therefore be used as a quick predictive test for gain in strength over time during and after curing.

With the preceding conclusions, the objectives of Chapter 7 were fully met.

CHAPTER 8

CONCLUSION

The principal objective of this thesis was to investigate the durability of CSBs, especially when used in conditions similar to those found in the humid tropics. Interest in the durability of CSBs is likely to remain a major concern for the foreseeable future given the potential the material has for reducing the enormous shelter backlog in developing countries (1.1). The figure in brackets relates to the section where the issue was discussed in this thesis. The adequate performance of a CSB throughout its service lifetime depends primarily on the interplay between three factors: choosing the right constituent materials, using the correct processing methods, and properly counteracting the effects of the exposure environment (1.2).

At the time of commencing this research, there was hardly any documented record of previous research in the same field. For this reason, a multi-pronged methodology was adopted involving: literature review (Part A of the thesis), laboratory experimentation, and an exposure condition survey (Part B of the thesis) (1.3). In this final Chapter, summary recommendations and conclusions are presented in three separate sections covering Part A, Part B and the highlights of the implications of the findings on further areas of research.

8.1 RECOMMENDATIONS AND CONCLUSIONS: PART A

The aim of the literature review conducted as part of the research was to provide the intellectual context for the work and to determine how far other researchers had reached. It was also meant to determine whether the literature on durability and stabilisation were accessible.

Chapter 2 explored the concept of durability and deterioration in CSBs (2.2). It was found that CSB literature on the subject was scarce and inaccessible. Since both concrete and CSBs develop their microstructure from the precipitation of solids from solution following the hydration of cement, documented findings on the former were used to try to understand related phenomenon in the latter. It is recommended that this approach be pursued further.

From the literature survey conducted, it was found that no uniformly accepted expression for durability existed. It is therefore recommended that the durability of a CSB be regarded as "a measure of its ability to sustain its distinctive characteristics of strength, dimensional stability and resistance to weathering under conditions of use for the duration of the service lifetime of the wall of which it forms part". This concept of durability is based on three important parameters: intended function of the block (for walling); conditions of exposure (weathering elements); and age of exposure (time in years). Due to the effects of exposure conditions, the properties of a block can be altered over time, and so their durability will not remain constant. Durability is therefore more dependent on exposure conditions than just time.

According to the literature surveyed, the time-related loss of quality of a block is its deterioration (2.2). It implies that the durability of a block can be regarded as its ability to resist deterioration. Due to deterioration however, the durability of a block can fall with time. The more a block deteriorates, the less durable it is, and will become over time. An assumed progressive deterioration model characterised by a gradual loss of performance (typified by a deterioration gradient) would be more applicable to the durability-time relationship. The service life of a block can be regarded as the actual period of time during which no excessive expenditure is required on its maintenance or repair in actual use (2.2.). The design life of a block is the period set by the designer of the building of which the block forms part. A gap exists in CSB literature on the concepts of service and design life. Further research is recommended with a view to reducing the gap between the two.

Chapter 2 also discussed various deterioration agents and their likely mechanisms (2.3). Three categories of deterioration modes were identified: water, temperature, and chemical related actions. Water-related deterioration was categorised as occurring in four different forms: abrasive action, solvent action, swelling action, and catalytic action (2.3.1). The most prominent of these was the direct abrasive action of rain on the surface of blocks leading to surface erosion. The exact mechanism and rate of surface deterioration is not yet well understood. Further research is recommended in this area.

Temperature-related deterioration was reported to cause both reversible and irreversible changes in block properties, occurring in three main ways: expansion and contraction, shrinkage and drying, and catalytic action. The main defect types associated with this mechanism of deterioration were surface and bulk cracking and crazing (2.3.2).

It was found through the literature survey that chemically-related deterioration was the least covered in CSB literature (2.3.3). Yet both soil and cement contain sources of potentially reactive minerals. Three categories of chemically related deterioration were identified: leaching-out effect (of clay and calcium hydroxide), expanded product formation (due to action of sulfates, soluble salts crystallisation and alkali-aggregate reactions leading to internal stress generation), and direct decomposition of the OPC hydrate binder (from acidic conditions). Leaching out effect and expanded product formation were regarded as being the most common. It is recommended that use of lime and pozolans in combination with OPC be considered in vulnerable materials. The two help in stabilising both clay and the freed calcium hydroxide from the reaction of OPC and water. It is further recommended that careful soil selection that avoids use of soils with an excess of clay (> 30%), and proper curing that ensures a maximum degree of hydration, be considered as ways of minimising some of the effects of chemically related deterioration. Limits should also be set on the amounts of sulfates (< 2.5%), active silica and carbonates, soluble salts (< 6%) and organic matter (< 3%) found in soils to be used for CSB production. At the moment, there are no such limits. The limits shown in brackets are from recommendations found in concrete literature. Despite these findings, the objectives of Chapter 2 were fully met.

Chapter 3 reviewed from literature sources current methods used to select the main constituent materials in CSBs, the mechanisms of cement-soil stabilisation, and processing methods for blocks (3.1).

The main constituent materials in CSB production were identified as: cement, soil and water (3.2). Coverage of these three materials varied a great deal in the literature reviewed, with quality of cement and water being the least documented. The function of OPC in a CSB is to bind and hold the soil particles together in a dimensionally stable unit (3.2.1). Coverage of OPC in CSB literature was very scant. No mention was made of the main desirable OPC physical properties such as specific surface area (300-350 $m^2\ kg^{-1}$) and particle size distribution (90% more than 5 μ : 1% < 90 μm). These two properties govern the manner in which OPC effectively stabilises soil. Moreover, the implications of the different rates of reaction and influence of the several OPC constituents on the stabilisation mechanism were not covered in CSB literature. Neither were the effects of the various hydrates formed following the reaction of OPC and water covered. These hydrates have implications on the durability of CSBs. By discussing issues such as these in Chapter 3, an attempt was made to fill the existing CSB gap in literature. Capillary porosity for example is closely associated with strength, and is controlled by the water cement ratio and the degree of hydration. While the former can be reduced by the use of very fine pozzolans (e.g. microsilica), the latter can be attained by ensuring that a high degree of hydration is achieved (by proper wet curing). It was this finding from the literature on cement chemistry that led to the successful manufacture for the first time of improved blocks of superior strength and durability than comparable conventional blocks (Chapters 6 and 7). The approach used is strongly recommended for CSBs meant for use in severe climatic conditions such as the humid tropics.

Chapter 3 also discussed findings from the literature review conducted on the characterisation and selection of soil for CSB production (3.2.2). It was found that soil classification and selection criteria were generally well covered in most CSB literature. Classification by particle size distribution is the most commonly used method. It is recommended that other methods based on plasticity, compactability, cohesion and chemical content also be investigated further for future use. The current soil selection criteria recommends the use of a well graded soil containing adequate proportions of coarse soil fraction (fine gravel and sand) and sufficient fines (silt and clay) for cohesion. The soil should ideally have about 75% coarse fraction and about 25% fines content (of which at least 25% is clay). As soils are highly variable and complex materials even in nature, it is recommended that even where soils on site do not conform to the above specifications, they be not rejected but modified. A dense, well graded soil requires less cement to bind its particles together due to the increase in specific surface area. The effect is even greater when a limit is set for maximum size fraction (< 6 mm). At the time of the research, it was established that various authors recommended different maximum size fraction sizes (5 mm, 6 mm, 15 mm, 20 mm). A limit of 6 mm is recommended.

The quality of water for mixing and curing is poorly covered in CSB literature (3.2.3). Due to the scarcity of water in most developing countries, the sources are varied and so is the quality. It was noted that the use of untreated water of no known service record cannot be ruled out.

Chapter 3 also reviewed current cement-soil stabilisation principles (3.3). The conclusion that emerges from the review is that, despite the recent scientific advances made, cement-soil stabilisation still remains an inexact science. Soil properties can be modified by mechanisms that vary the soil-water-air interphase through minimising the volume of interstitial voids and improvement of cohesion and bonding between its particles (3.3.1). The literature documents three theoretical and practical methods of achieving this: mechanical (compaction), physical (improvement of soil grading), and chemical (using a stabiliser such as OPC). The effect of chemical stabilisation mechanisms are widely documented as being more permanent. It is therefore recommended that chemical stabilisation of soil be done even when the other two methods have been used (3.3.3).

Further research is required to determine the proportions of the final CSB matrix known to comprise the following: cement hydrates, conventional cement-sand mortar, calcium hydroxide, unstabilised clay and sand, and unhydrated cement residues. According to literature sources, the predominance of any one of these products in a CSB fabric can influence its durability.

In Chapter 3, the block production process was described as being a major input variable that can affect the properties and behaviour of a block (3.4.1). The main processing stages identified from the literature were: soil preparation, mixing, moulding, and curing. The sequencing is so dependent that one stage must be completed before the next one can begin. The importance of each of the sub-stages in the block production process has often been underrated. Underestimation of the above steps can lead to the production of blocks of low strength and durability (3.4.1, 3.4.2, 3.4.3, 3.4.5). Generally, as the findings described in this section show, the objectives of Part A of the thesis were fully met.

8.2 RECOMMENDATIONS AND CONCLUSIONS: PART B

Part B of the thesis was devoted to direct investigations incorporating an exposure condition survey in a humid tropical environment and laboratory experimental work. The findings were reported in Chapters 4, 5, 6 and 7.

Chapter 4 described methods and findings from the exposure condition survey conducted in Uganda where CSBs have been in use since the late 1980s (4.1). Uganda is a humid tropical country, where deterioration agents occur naturally. The exposure conditions were considered to be sufficiently representative of similar conditions in most of the humid tropics. Four methods were used during the fieldwork: (i) collection of data on the inventory of CSB structures and the exposure condition, (ii) condition survey of existing buildings (random inspection, in-service testing, maintenance records), (iii) observation of methods of work at CSB production sites, including field indicator testing for soils and quality test checks of OPC and water, and (iv) interviews and questionnaires (4.1).

From the provisional inventory of CSB buildings in the country, it was found that a large stock of over 400 buildings had been built since 1987 (4.2.1). This however represents a very small fraction of the total number of buildings constructed over the

13 year period. The buildings were constructed in an attempt to reduce the enormous housing backlog (estimated at 3 million by the year 2006). Up to 90% of the CSB buildings were found in high density, low income urban areas (Namuwongo in Kampala, and Malukku in Mbale). The general conclusion made was that the rate of construction was not yet able to meet the enormous demand for low cost housing. The demand for CSB buildings is therefore likely to remain high for the foreseeable future.

Chapter 4 also described the characteristics of the natural exposure environment in Uganda (4.2.2). This was done to identify the main naturally occurring agents whose effects were likely to prove deleterious to CSB structures during their service lifetime. The main agents identified were rain, temperature and relative humidity. It was found from records that the average rainfall intensity was above 7.5 mm/hr (i.e. heavy rainfall), with drop sizes varying from 0.5 mm to 6 mm. The duration of rains varied between one and six hours. With a frequency of two rainy seasons lasting about 6 months, it can be concluded that water-related deterioration of CSBs is likely to occur during the service lifetime of such buildings. It is recommended that more research be done on erositivity of rain including the contribution of the interactions of rain drop size, drop size distribution, fall velocity and impact kinetic energy to the deterioration process.

It was also found from records that ambient temperatures averaged about 25°C, with surface temperatures in the shade reaching about 100°C. It can be concluded that under such conditions, temperature related deterioration will occur within the service lifetime of a block. Moreover, with the presence of large water bodies (lakes, rivers, swamps) throughout the country, high temperatures ensure that there is a high relative humidity (30-90%). These conditions can serve as catalysts to chemical and biologically related deterioration mechanisms. The conclusion made was that as characterised, the exposure conditions in the country provide an ideal setting for most deterioration mechanisms discussed in Chapter 2.

Chapter 4 described several reasons why visual inspection as a way of evaluating defect types and their severity on exposed block surfaces had been selected (4.3.1). All 58 buildings inspected (representing about 15% of the total CSB building stock) were all chosen at random. Their ages ranged from one month to 12 years. It was found that defect types were wide ranging: surface erosion, spaling, pitting and roughening (due to rain); surface and bulk cracking and crazing (due to temperature variations); surface and plant growth (due to biological action); disintegrated loose material residues (due to chemical action); and interblock and mortar cracks (due to settlement). The predominant defect types were surface erosion (75%) and cracking (25%). These findings confirmed that premature deterioration of CSBs can occur in the humid tropics. It was also found that like materials used under similar conditions for the same period of exposure did not show similar defects.

Chapter 4 also described findings from in-service measurements done to determine the amount of volume reduction that had occurred due to mass loss, and the dimensions of cracks (4.3.2). It was established that surface erosion can lead to irrecoverable loss of volume in a block. It was found that the reduction in volume varied with the elevation of a block within a wall, the orientation of the wall façade, and the age of exposure. For the 12-year old building, volume reduction at the higher and lower levels of its walls averaged about 28% and 35% respectively. The mean volume reduction for the east-west façade was about 34%, while that for the north-south one was about 28%. The mean volume reduction for all facades in the 8-year old structure was about 22%, while that for the 12-year old building was 31%. The average estimated rate of annual mass loss for both structures was below 3%. The rate of mass loss can be influenced by the degree of resistance offered by a block surface. It is recommended that CSB surfaces used under similar exposure conditions be made more denser, smoother and of higher intergranular strength. Other surface protection measures should also be considered, such as: rendering, surface coating and layering with higher intergranular strength mixes at the surface. Adequate surface protection is likely to remain the most economic way of increasing the durability and thus extending the service life of a block.

The severity of cracking on CSB surfaces was found to follow the same trend as surface erosion (4.3.2). It was established that while cracks occurred on all wall facades, their widths on the east-west facades (2.5 mm to 2.9 mm) were markedly greater than on the north-south facades (0.65 mm to 0.80 mm). The measured values were found to exceed the maximum permissible crack widths specified for concrete (0.25 mm for severe exposure, and 0.15 mm for normal exposure conditions). Such comparisons do not take into account the fact that CSBs contain clay, while concrete does not. It is recommended that similar permissible maximum crack widths, higher than those given for concrete, be set for CSBs. It was also found that exceptionally thick mortar was widely used for bedding blocks (15-20 mm thickness). Such mortar thickness can prevent flexible movement on expansion of blocks encouraging cracking and is therefore not recommended. It can be concluded that while a particular cause within or outside a block might initiate cracking, its subsequent development can be due to other causes. The different types of cracks observed (star shaped, linear, interconnected and penetrating) indicated that there were more than just one cause of cracking in CSBs. The linking of particular crack patterns to likely causes is recommended for further research.

Findings from preliminary field indicator tests showed that the test methods used can be valuable indicators of soil properties and behaviour (4.3.3). The conclusion made was that although the tests were largely empirical, they could still enable the general suitability and acceptability of a soil to be determined rapidly and at lower costs. It is recommended that of the 15 different indicator soil tests described, the linear shrinkage test and the sedimentation test be made compulsory. This is because they are less vulnerable to operator errors than all the other tests. The tests should also be done in the order in which they were presented in this thesis. It is further recommended that the outright rejection of soils as being unsuitable as advocated for by previous authors be avoided. It should only be done when laboratory tests show that it will prove too costly to modify the soil by improving its grading (removal of the excess fraction or inclusion of the missing fraction through controlled mixing).

It was found from visits to block production sites that no proper processing procedures were being followed (4.4.1). Yet the production process represents a major input variable that can influence the properties and performance of a block. The observations of shortcomings noted during soil extraction and preparation, mixing, moulding and curing confirmed fears that poor site practice, bad workmanship, lack of supervision and codes of practice can affect the final quality of a block. It is recommended that appropriate codes of practice, preferably based on a checklist system of good practice, be made available on block production sites. It can be concluded that without proper standards and codes, even skilled supervisors and foremen might not be able to appreciate the consequences of bad methods of work.

The results of quality checks on OPC and water used on sites showed that variations from standard specifications can significantly affect the properties and performance of a block (4.4.2). Quality checks on prisms made using the cement on site showed that the wet compressive strength (28-day) was about 15% lower than the specified minimum of 32.5 MPa for the class of OPC used (class 32 N OPC; BS 12, 1990). The prisms tested for tensile load were between about 25% and 30% lower in capacity than the recommended load values at one day and at 28 days respectively. The conclusion here is that the OPC found being used on site was not of the same quality as the specified one. It must have been contaminated at some stage (purchase, storage, mixing). Press reports seen more than one year later confirmed that malpractices involving the adulteration of OPC with clay was rampant. It can be concluded that use of low grade OPC will affect the properties and thus performance of a block. It is recommended that regular quality checks be conducted on OPC found on CSB production sites.

It was also established that use of water of unknown service record can result in blocks of lower wet compressive strength (4.4.2). The difference in strength from the specified minimum was about 23%. Tensile load tests using the same mix water source showed that the prisms were about 43% lower in wet compressive strength than the specified minimum. The conclusion here is that use of water of unknown service record can affect the strength and by implication, other properties of a block. However, since water is scarce in developing countries, the continued use of such water sources cannot be ruled out. It is therefore recommended that simple water purification and quality improvement methods be adopted (3.2.3). It was also noted that use of pre-treated tap water was taken for granted by most CSB authors.

Results of interviews and questionnaires conducted revealed a number of wide ranging issues (4.5). The number of respondents contacted was 35 (stakeholders including users, professionals, government officials, project managers, etc., all chosen at random), and the response rate was 100%. A number of conclusions can be drawn from the results.

It was found that the walling materials of choice based on previous experience and tradition were fired bricks (40%), followed by concrete blocks (33%) and CSBs (22%). Adobes were the least preferred, being considered materials of the last resort. The main reason given for preferences was the durability of the material as evidenced by its service life record (77%), followed by costs (15%) and tradition (2%). Preferred block types were found to be dry stacked interlocking blocks (55%), followed by solid blocks (32%), and bed frogged blocks (10%). Hollow blocks, despite their cost saving potential, were the least preferred (3%). The most common defect types observed by respondents over the years were surface erosion (including pitting and roughening) (75%), followed by surface and bulk cracking and crazing (20%). These findings are in agreement with earlier findings reported after the visual inspection was done. Preferred surface protection methods were external plaster and render (54%), followed by surface coatings (23%), and architectural design that incorporated a low roof overhang (18%).

According to these respondents, suggested ways of improving the service life of blocks and thus promoting their image amongst potential users include improvement of bulk strength and bonding (40%), dissemination of standards and codes (28%) and improved architectural design (20%). It can be assumed that the views of the stakeholders interviewed as summarised here represents the broad opinion and experiences of other users in developing countries. The above findings show that the objectives of Chapter 4 were fully met.

In the experimental design described in *Chapter 5*, all the input variables that can influence the quality and performance of a block were identified (5.2). They include constituent materials and processing methods (5.2 to 5.4). The soil type was fixed for all blocks, while the stabiliser type and amount, moulding pressure and curing conditions were varied. The main objective was to compare the properties and performance of improved blocks (with 10% of the cement content comprising microsilica) and traditional blocks (OPC only stabilised and OPC plus lime stabilised). Bulk and surface properties of both categories of blocks were extensively tested. The number of specimens produced for each test was three. The decision to use three specimens was based on earlier preliminary tests which concluded that the variability for the major tests were quite low (5.4.2). With the above findings, the objectives of Chapter 5 were fully met.

Chapter 6 described findings from bulk property tests which included wet compressive strength, block dry density, total water absorption and volume fraction porosity The mean WCS of improved blocks were found to be more than double those for matching traditional blocks (6.2.1). Although some improvement in strength had been expected, the order of magnitude achieved was surprisingly greater than predicted. The conclusion here is that the use of a partial cement replacement material (such as microsilica) can be an effective way of increasing strength, and by implication the durability of a block.

It was also found in the case of improved blocks, that the WCS value at the 5% cement content level (range of interest) was about five times greater than the recommended minimum value of 1.2 MPa (6.2.1). Even at the lower cement content of 3% (generally not used), the WCS value attained was surprisingly about three times higher than the minimum recommended value. The trend of the graph showed that where microsilica is used, only 1% of OPC would be required to achieve the minimum wet compressive strength of 1.2 MPa. There is no previous record prior to this thesis to show that similar spectacular gains in strength have ever been achieved in CSBs. The use of microsilica in enhancing strength in blocks is therefore strongly recommended.

The effect of increase in cement content with strength in blocks was found to closely correspond in all cases (6.2.1). Overall increase in strength in both traditional and improved blocks with increase in cement content from 3% to 11% was about six-fold. It was generally found that the increase in WCS was higher at the lower cement content levels than at the higher ones (220% compared to 97% respectively). It can therefore be concluded that use of cement contents beyond 7% is not an economic way of achieving further strength in CSBs.

It was also established that increase in compaction pressure resulted in an increase in WCS. A 70% increase in compaction pressure resulted in a 32% increase in WCS (6.2.1). The increase in WCS is however considerably lower than that achieved through a similar increase in cement content. It can be concluded that increase in cement content is a more effective way of increasing the WCS in blocks. Even where blocks of high cement content were compacted at lower moulding pressures, they were found to perform satisfactorily. The opposite was not found to be the case. This confirms earlier conclusions that the ultimate cured wet strength of a block is more sensitive to changes in cement content than compaction pressure. Moreover, it was also found that the degree of increase in WCS with increase in compaction pressure diminishes as the pressure increased. It can therefore be concluded that block presses operating within the range 2 MPa to 8MPa can be adequate to produce blocks of sufficient WCS.

It was also found in Chapter 6 that the ratio between mean dry and wet compressive strength was much lower in improved blocks than in the traditional ones (6.2.2). The ratio ranged between 1.4 and 1.9 in traditional blocks, but only between 1.1 and 1.3 in improved blocks. The ratios for the improved blocks were found to compare well with those for concrete blocks (between 1.09 and 1.21). The findings show that the higher and broader the ratio between mean DCS and WCS, the lower can the degree of intergranular bonding be expected to be. It can be concluded that the reduction in ratio for improved blocks is directly attributed to the inclusion of microsilica. This must have transformed the weaker and more porous CSB fabric into a far denser, more homogeneous and more impermeable matrix, than was hitherto possible. The use of CRMs in improving CSB properties such as strength is therefore strongly recommended. It is further recommended that use of the value of the ratio between the mean DCS and WCS be adopted as a tool for assessing the quality of bonding achieved in CSBs. Where CRMs are used, it is recommended that various cost reduction measures be considered: use of thin surface layered blocks, hollow blocks, frog-bedded blocks, and interlocking blocks.

It was shown in Chapter 6 that the effects of processing variables such as hold-back time on the WCS of blocks can be adverse (6.2.3). A progressive loss of quality was found to occur on delay in compaction of a damp soil cement mix. It was established that blocks compacted within 20 minutes of delay after damp mixing were about 27% stronger than those compacted after 45 minutes. Blocks compacted within two hours of delay were about 41% weaker. These findings compare well with those of earlier researchers. Similar effects can be expected to occur in improved blocks. It is therefore recommended that only batches that can be compacted within 30 minutes, instead of the currently used one hour, be mixed and used up in that time. The findings also confirm earlier ones which noted that poor site practice can result in the production of low quality grade blocks. It is strongly recommended that all CSB production processes be treated with the same high level of skill, competence and supervision. This should be reinforced through standards, codes, checklist systems and certification requirements.

The effect of varying curing conditions on the performance of was blocks was investigated in Chapter 6 (6.2.4). Blocks cured under exposed conditions were found to be about two-fold weaker than blocks cured under standard conditions. Had they been left exposed directly under the sun (as is commonly the practice on block production sites), the loss in WCS would have been even higher (4.4). Blocks cured by prolonged covering throughout were found to be about 29% stronger than their standard cured counterparts. Blocks cured fully immersed in water were about three times stronger than standard cured blocks, and about six-fold stronger than those cured in open exposure in the laboratory. Variation in curing conditions affects the state of moisture in a green block. It can be concluded that the fully immersed blocks emerged strongest because hydration was allowed to continue until a maximum degree of hydration was achieved. It is therefore recommended that the curing of blocks be done in such a manner as to allow the continued presence of moisture to complete the hydration reaction of OPC. Wet curing should be extended to longer periods than currently allowed for. These results also confirm the urgent need for proper codes of practice to be observed during the manufacture of blocks.

From investigation into the effects of varying the stabiliser type and content on the block dry density, it was found that the latter varied markedly with changes in the former (6.3). For matching OPC content, it was found that the density in improved blocks was between 3.3% and 5.2% higher than in corresponding traditional blocks (6.3.1). The conclusion here is that inclusion of microsilica in improved blocks had a pore filling effect, and resulted in increased homogeneity, improved bonding and reduced voids content in the block. Dry density can be a valuable indicator of quality in a block. Density however also depends on the degree of compaction used, the density of the constituent materials, the size and grading of soil particles and on the form of a block (solid, hollow, etc.). It was also established that no uniform standard exists for the determination of dry density in CSBs. It is recommended that the method requiring oven pre-drying to constant mass be adopted.

It was found in Chapter 6 that a strong positive correlation exists between density and the 28 day WCS in both categories of blocks (coefficient of correlation was 0.971 for traditional blocks and 0.996 for improved blocks) (6.3.2). It can be concluded that increase in density can result into an increase in WCS. The increase was however found not to be uniform throughout, being more pronounced at the lower cement contents than at higher ones. However, very high densities could prove disadvantageous during block laying and transportation. It is recommended that production of blocks heavier than 8.5 kg be avoided.

It was also found in Chapter 6 that due to the existence of pores within their fabric, all categories of blocks absorbed water (6.4). Increase in stabiliser content resulted into a decrease in TWA (6.4.1). Traditional blocks absorbed more water than their improved counterparts (more by 120%). The overall decrease in TWA with increase in cement content from 3% to 11% was around 40%. Generally, the less water a block absorbs, the better is its performance expected to be. It can be concluded that TWA is a valuable indicator of quality of a block as it can be used to estimate the total volume of pore space (voids).

The results showed that beyond a certain stabiliser content, water absorption by a block ceases to decrease any further, becoming almost uniform instead. The limiting value was found to be 9% in traditional blocks, but only 7% in improved blocks. It can be seen that lowering of the limit to 7% in improved blocks must have been due to the pore filling effect of microsilica.

It was also established from the results that the TWA values obtained were much lower than the recommended maximum value of 15%. Values for improved blocks were the lowest (3% to 6%). The conclusion here is that use of microsilica in improved blocks was an effective way of lowering TWA (than increase in compaction pressure). It is recommended that TWA values in blocks be used for routine quality checks, for comparison with set standards, for approximation of the voids content, and for classification of blocks according to required durability, and structural use. It is also further recommended that existing TWA test methods be standardised. Current tests do not take into account the need to oven pre-dry blocks to constant mass in order to expel air and water from the pores before immersion in water.

In Chapter 6 a strong correlation was found to exist between TWA and density (correlation coefficients were –0.985 and –0.820 for traditional and improved blocks respectively). It can therefore be inferred that increase in BDD will result in a decrease in TWA (6.4.2). For example, increase in density of 2.3% resulted into a decrease in TWA of 44% in traditional blocks (39% in improved blocks). The results also showed that beyond a certain density value (corresponding to the limiting OPC contents described earlier), no further appreciable reduction in TWA could be expected.

A general link between TVP and the performance of blocks was established in Chapter 6 (6.5). It was shown that a very strong negative correlation exists between TVP and WCS (coefficients –0.905 for traditional blocks, and –0.771 for improved blocks) (6.5.1). The conclusion here is that the greater the pores, the higher the number of flaws and localised faults within a block fabric, and so the weaker it is. The TVP was lower in improved blocks (8.4% to 13.3%) than in corresponding traditional blocks (14.4% to 25.3%). This can be attributed to the pore filling effect of microsilica. It is recommended that use of microsilica be considered in future as an economic way of reducing the TVP in CSBs.

It was also found in Chapter 6 that the correlation between BDD and TVP was strong and negative (correlation coefficients –0.984 in traditional blocks, and –0.935 in improved blocks) (6.5.2). Increase in density of about 4.1% was found to result in the lowering of the TVP by about 37% in improved blocks. It can be concluded that increased densification can be an effective way of reducing the TVP in blocks. The TVP is however also a function of water-cement ratio and of the degree of hydration achieved. The value of the latter can be increased only when moisture is available to complete the hydration process. It is therefore recommended that proper moist curing be used as a way of reducing the TVP in CSBs. The general link established between TVP and other block bulk properties are similar to those reported in concrete literature. The TVP approach has not been used before in quality evaluation of CSBs. It is recommended that TVP be included as a quality check parameter for CSBs. With the preceding findings in Chapter 6, it can be concluded that improved blocks performed significantly better than their traditional counterparts in terms of all properties for which they were tested (WCS, DCS, BDD, TWA, TVP). The objectives of Chapter 6 were fully met.

Chapter 7 described findings from petrographic analysis and surface performance monitoring tests done on improved and traditional block samples. It was noted that as the outermost boundary of a block, the surface represents its first line of defence against deterioration agents and is therefore an important feature for a block (7.1). It was also noted that any erosion of the block surface that exposes the bulk would most likely lead to an accelerated rate of deterioration.

From the thin-section micrographs of block surfaces examined it was found that the general features revealed the existence of an amorphous particulate composite structure of predominantly short range order (7.2). This was expected since particulate regularity in such a composite material is difficult to attain. Moreover since CSBs like concrete are formed from the rapid precipitation of solids from solution, random packing such as was observed should be expected. This contrasts with the distinctly continuous interlocking phases reported in fired bricks (due to mulite formed from firing). No previous publication of similar petrographic analysis exists for CSBs.

The most distinguishable features noted were coarse soil grains (fine gravel, sand), gross porosity, calcium hydroxide, clay inclusions and aggregations of OPC hydrates in the groundmass. At the resolution used, the micrographs could not resolve sub-micron phases such as individual clay or microsilica grains. The presence of calcium hydroxide justifies the use of microsilica to promote pozzolanicity in CSBs. It was however difficult to detect any micro-defects in these particular samples. It is very unlikely that similar micrographs of samples made in the field would have yielded the same results. The conclusion here is that the samples were well mixed. The micrographs confirm the release of calcium hydroxide which when left in a block fabric can be detrimental to its durability (Chapter 2). The surprisingly low gross porosity detected in improved blocks also vindicates the use of microsilica in CSBs.

The conclusion here is that by reducing voids through densification or inclusion of CRMs, pores can be reduced, hence lowering water absorption and permeability properties of a block. Further, by improving bonding through the use of CRMs and proper wet curing, closer and more rigid contacts can be attained, hence improving the surface resistance of a block. Use of microsilica in CSBs is therefore strongly recommended. Use of petrographic examination of CSBs should be extended to examine samples from various production sites.

It was discussed in Chapter 7 that no proper accelerated surface test is currently available for monitoring the performance of CSBs (7.3). Existing methods (the water drip test, water spray test, brushing test, hardness test, absorption test and the wet-dry-cycling test) all lack reliability, repeatability and accuracy. These tests were found to be operator dependent and difficult to conduct. This explains why blocks were in the past passed as durable only to prematurely succumb to normal or severe exposure conditions. The slake durability test (SDT) was therefore proposed and used as a quick predictive accelerated test for monitoring the performance of CSB surfaces of various categories (7.3). It is recommended that the standard procedures used for the test be maintained for all future tests on CSBs. It is also recommended that further research be undertaken to modify the test apparatus to make it convenient to use on a block production site (e.g. manual operation instead of mechanical).

Using the SDT, the effect of varying the stabiliser type and content on the quality of block surfaces were monitored (7.3.1). It can be concluded that more rapid mass loss will occur from the surfaces of traditional CSB samples, than from those of like materials such as fired bricks, concrete blocks and rocks. It was found that mass loss was markedly higher in traditional blocks than in improved blocks of matching cement contents.

Improved blocks of cement content above 9% were found to have mean SDI value of about 99.1%, performing as well as fired bricks and concrete block samples (mean SDI values of 99.8% and 96.6% respectively). According to current and proposed SDI classification system, improved blocks of 5% cement content and above could be categorised as being of "very high durability" or grade B and better blocks. Comparable traditional blocks of the same cement content if carefully made would be classified as being of "high durability". It can be concluded that the use of microsilica reduces the loss in mass in blocks, by considerably increasing its surface resistance to cyclic wetting, abrasion and drying. Use of microsilica (or other similar CRM) is therefore highly recommended as a way of improving the surface resistance of a block.

A strong correlation was found to exist between increase in cement content and the SDI value of all categories of blocks tested (7.3.1). It was established that increase in SDI with increase in cement content was higher at the lower cement content levels (less than 7%) than at higher ones (40% compared to 6% in traditional blocks and 22% compared to 4.9% in improved blocks). In both cases, increase in OPC content beyond 7% did not result into any further appreciable increase in SDI values. This phenomenon of diminished increase in performance with increase in OPC content beyond about 7% has featured in almost all the properties evaluated. It can be concluded that at 7% cement content, CSBs will perform better in most respects than at the current lower recommended level of 5%. The elevation of the minimum amount of OPC used (5% to 7%) is strongly recommended for CSBs meant to be used in the humid tropics. Ways of reducing costs such as bed-frogging of blocks or use of interlocking blocks that do not require use of mortar or render could be investigated. Since rapid loss in mass was detected in most block samples, it is recommended that surface protection measures be used for CSBs as described earlier (especially blocks with 5% cement content and below).

It was also found in Chapter 7 that the SDI value was strongly correlated with the evolution of strength in blocks during curing (7.3.2). Increase in curing age was found to correspond to increase in SDI values. The increase was more phenomenal before the 28th day (for OPC stabilised blocks) and on the 56th day (for OPC plus lime stabilised blocks). No appreciable increase in SDI with curing age was recorded after these periods. The SDI value for improved blocks at 28 days was about 85% higher than at seven days. The comparable figures for traditional blocks was about 51%. It can be concluded that improved blocks gained strength, and thus surface resistance, more rapidly than their traditional counterparts. The results further indicate that SDI values can be used as a surrogate test for quality in CSBs irrespective of the pre- and post-curing periods. The pattern was similar to that found with increase in strength over time during curing. Moreover, the SDT test was found to be applicable even six months and after the prescribed curing periods. The conclusion is that a new test that can reliably test the evolution of strength similar to wet compressive strength has been found for CSBs. A further conclusion is that the SDT can be used to evaluate and classify blocks irrespective of their curing age and storage history. This was not possible prior to these findings.

The correlation between SDI and WCS was found to be very strong and positive, thus confirming the preceding conclusions (7.3.3). The conclusion here is that the higher the value of the 28 day WCS, the greater is the resistance offered to surface erosion due to wetting, abrasion and drying. It was however also established that there was a diminished increase in SDI with increase in WCS. The SDI is therefore a valuable indicator of both strength and surface resistance in CSBs.

A strong correlation was also found to exist between SDI and TWA (coefficients were –0.975 and –0.939 for traditional and improved blocks respectively) (7.3.4). The higher the SDI value, the lower the TWA. The inference here is that higher surface resistance corresponds to lower water absorption. Both properties are therefore valuable indicators of surface and bulk quality respectively.

It was also established in Chapter 7 that a strong positive correlation exists between

SDI and the BDD (7.4.4) (correlation coefficients were 0.944 and 0.953 for improved blocks and traditional blocks respectively). Both were significant at the 95% confidence level using the 2-tailed test. The conclusion made is that increase in density can be associated with increase in the SDI value of a block. The denser the packing of particles and phases within a block (i.e. lesser voids), the stronger and therefore more durable it can be expected to be. Increase in SDI with increase in density was higher in traditional blocks than in the improved ones (2.3% increase in density resulting into a 49% increase in SDI, as compared to 4% increase in density resulting into a 28% increase in SDI in improved blocks). The conclusion here is that the denser the block, the less is the increase in SDI value, but the higher is the resistance to surface erosion. Increase in density is therefore an economic way of increasing the SDI value in blocks.

As the preceding findings have shown, use of the SDT as a new surface quality test is strongly recommended. Use of the procedure was found to be simple, controllable, fast, practical, accurate and of timeless value. The test method was also found to be an excellent accelerated test procedure since loss in mass occurred with significant short term value for research. The test also simulated the main deterioration mechanisms on block surfaces (erosion due to repeated wetting, abrasion and drying). Further research is recommended into the test method with a view to having the results calibrated with those obtained from natural exposure condition surveys. It is possible that the test results could one day be used to estimate the rate of surface erosion due to this particular mode of deterioration.

It is further recommended that the proposed SDI classification system be adopted for use with CSBs. The SDI test results can be used in several ways: as an aid to block classification, for selection of blocks for particular applications, for quality control during production (and delivery to site), for prediction of the rate of surface material loss, and for selection of suitable presses. The use of the SDT is likely to ensure that the durability of CSBs can for once be quantitatively determined in a more uniform and independent manner than before. Minimum required values can be specified and included amongst initial performance characteristics of CSBs. This is likely to bring an end to widespread attempts to characterise CSBs qualitatively as being of low or high durability without any standardised method of quantitative determination. From the preceding findings and conclusions, the objectives of Chapter 7 and Part B of this thesis were fully met.

8.3 RECOMMENDATIONS FOR FURTHER RESEARCH

The main objectives of this thesis have been fully met (8.1, 8.2). The findings have however flagged up a number of new questions for further future research. It was not possible to undertake the identified new research areas within the current study. The areas for further research include the following:

- Durability concepts should be developed further so that a proper expression for the term (that extends what was described in this thesis) can be documented in CSB literature. This should be based on the intended function of a block, its conditions of use, and time in years.
- Deterioration agents should be ranked according to their severity as attempted in this thesis, and the mechanisms of their action investigated further with a view to understanding them better (surface versus bulk phenomenon).
- Surface protection methods should be researched into with a view to reducing costs. The cost and applicability of high durability blocks which are not rendered could be compared with those of low durability blocks which are

rendered. The use of surface enriched thin layers, hollow blocks, interlocking blocks and bed frogged blocks should be investigated as ways of reducing costs while maintaining adequate surface properties.

- The role of the various OPC hydrates in determining the durability of blocks requires further research. Ways of lowering the water-cement ratio and increasing the degree of hydration also require further work.
- In-service performance data of CSBs should continue to be collected and documented. Data banks could be established where such information can be centrally collected and sourced. Of particular interest to further research should be information on volume reduction due to mass loss, and crack formation in CSBs.
- Accelerated test methods for block surface evaluation and monitoring require further research. The SDT or similar tests that are not operator dependent, easy to conduct, and to interpret results, should be researched into. The test method should simulate the main modes of deterioration for the particular type of surface resistance required and should be convenient to use on site.

Finally, the use of CSBs as a cheaper alternative walling material is likely to increase in the foreseeable future. It is the improved durability of a block, rather than of any other property, that is likely to ensure its widespread acceptance in developing countries.

BIBLIOGRAPHY

1. Adam, E.F.A. (1982). *Stabilised soil blocks for low cost housing in Sudan.* Hatfield Polytechnic and Building Research Establishment. Garston, England.

2. Agarwal, A. (1981). *Mud, mud. The potential of earth-based materials for Third World Housing.* International Institute for Environment and Development (I1ED). Earthscan Publications. London, England.

3. Aksa, Z. (1984). *A lime stabilised soil building material.* Institute of Human Settlements. Banding, Indonesia.

4. Akroyd, T.N.W. (1962). *Laboratory testing in soil engineering.* Soil Mechanics Ltd. London, England.

5. American Society for Testing Materials. (1975). *Wetting and Drying Test of Compacted Soil-cement Mixtures.* ASTM D 559-44. USA.

6. Andersson et al. (1983). *Blockmaking machines for soil blocks.* Swedish Association for Development of Low Cost Housing (SADEL). Lund, Sweden.

7. Anderson et al. (1982). *Cement-Stabilised Torba.* Swedish Council for Building Research and Swedish Agency for Research Cooperation with Developing Countries. Lund University. Lund, Sweden.

8. Ashworth, A. (1994). *Cost Studies of Buildings.* 2nd Edn. Longman Scientific and Technical. Essex England.

9. Atkinson, B. (1970). *Weathering and Performance of Building Materials.* MTP. London, England.

10. Apers, J. (1983). *Binders: Alternatives to Portland Cement.* Technisch Dossier. ATOL. Leuven, Belgium.

11. Arrigone, J.L. (1986). *Appropriate Technology Adobe Construction.* A Research and Demonstration Project on the Use of Adobe for Low-cost Housing Construction. National Building Research Institute / Council for Scientific and Industrial Research. Pretoria, South Africa.

12. Baker et al., (Ed). (1991). *Durability of Building Materials and Components.* Proceedings of the Fifth International Conference held in Brighton, 7-9 November 1990. Chapman & Hall. London, England.

13. Bessey, G.E. (1975). *Production and use of lime in developing countries.* Overseas Division. Building Research Establishment. OBN 161. Garston, England.

14. Bilham, E.G. (1962). *The Classification of Heavy Falls of Rain in Short Periods.* HMSO. London, England.

15. Bishop, J.D. (1973). *Library Bibliography 250: Soil Stabilisation.* June 1973. Building Research Establishment. Garston, England.

16. Blanchard, D.C. (1948). *Observations on the Behaviour of Water Drops.* Gen.El.Res.Lab. Project Cirrus, 7. USA.

17. Building Research Establishment. (1979). *Estimation of thermal and moisture movements with internal stresses.* Digest No. 229, Part 3. BRE. Garston, England.

18. Building Research Establishment. (1980). *Building in Hot Climates; A selection of Overseas Building Notes.* Her Majesty's Stationary Office. BRE. Garston, England.

19. Building Research Establishment. (1982). *Repairing Brickwork.* BRE Digest 261, 1982. BRE. Garston, England.

20. Bungey, J.H. & Millard, S.G. (1996). *Testing of Concrete in Structures.* 3rd Edn. Blackie Academic and Professional. Cambridge, England.

21. Bureau of Reclamation. (1974). *Earth Manual.* United States Department of the Interior. Washington DC, USA.

22. Brandon, D. & Kaplan, W.D. (1999). *Microstructural Characterisation of Materials.* John Wiley and Sons. Chichester, England.

23. Briggs, P. (1994). *Guide to Uganda.* Bradt Publications. Chalfont St Peter. Bucks., England.

24. British Standards Institution. (1950). *Durability.* CP3: 1950. BSI. London, England

25. British Standards Institution. (1970). *Methods of test for resistance to air and water. Part 2: Permeable walling construction (water penetration).* BS 4315: Part 2: 1970. BSI. London, England.

26. British Standards Institution. (1971). *Specification for Portland cement (ordinary and rapid hardening).* BS 12: 1971. BSI. London, England.

27. British Standards Institution. (1972). *Specification for building limes.* BS 890: 1972. BSI. London, England.

28. British Standards Institution. (1981). *Precast concrete masonry units.* Part 1. *Specifications for precast concrete masonry units.* BS 6073: Part 1: 1981. BSI. London, England.

29. British Standards Institution. (1981). *Precast concrete masonry units. Part 2. Method for specifying precast concrete masonry units.* BS 6073: Part 2: 1981. BSI. London, England.

30. British Standards Institution. (1985). *Clay bricks.* BS 3921: 1985. BSI. London, England.

31. British Standards Institution. (1985). *Code of practice for use of masonry. Part 3. Materials and components, design and workmanship* (formerly CP 121: Part 1). BS 5628: Part 3: 1985. BSI. London, England.

32. British Standards Institution. (1986). *British Standard Guide to Building Maintenance Management.* BS 8210: 1986. BSI, London, England.

33. British Standards Institution. (1990). *Stabilised materials for civil engineering purposes. Part 1. General requirements, sampling, sample preparation and tests on materials before stabilisation.* BS 1924: Part 1: 1990. BSI. London, England.

34. British Standards Institution. (1990). *Stabilised materials for civil engineering purposes. Part 2: Methods of test for cement-stabilised and lime-stabilised materials.* BS 1924: Part 2: 1990. BSI. London, England.

35. British Standards Institution. (1990). *Method of test for soils for civil engineering purposes. Part 1. General requirements and sample preparation.* BSI 1377: Part 1: 1990. BSI. London, England.

36. British Standards Institution. (1990). *Method of test for soils for civil engineering purposes. Part 2. Classification tests.* BS 1377: Part 2: 1990. BSI. London, England.

37. British Standards Institution. (1990). *Method of test for soils for civil engineering purposes. Part 4. Compaction-related tests.* BS 1377: Part 4: 1990. BSI. London, England.

38. British Standards Institution. (1992). *Guide to Durability of buildings and building elements, products and components.* BS 7543: 1992. BSI. London, England.

39. British Standards Institution. (1995). *Building maintenance.* BSI. London, England.

40. Brunskill, R.W. (1997). *Brick Buildings in Britain.* Victor Gollancz. London, England.

41. Carter, R.F. & Bently, S.P. (1971). *Correlations of Soil Properties.* Pentech Press. London, England.

42. Caroll, R.F. (1990). *Disease reduction by improved house construction.* Overseas Building Note, OBN 194, August 1990. Building Research Establishment. Garston, England.

43. Caroll, R.F. (1992). *Bricks and blocks for low-cost housing.* Building Research Establishment. OBN 197. March 1992. Garston, England.

44. Casagrande, A. (1947). *Classification and identification of soils.* Proceedings of the American Society of Civil Engineers (ASCE), June 1947, pp 783-810. USA.

45. Case, J., Chilver, L., Ross, C. (1998). *Strength of Materials and Structures.* 3rd Edn. Arnold/Hodder Headline Group. London, England.

46. Chandra, R. (1970). *Slake-durablity Test for Rocks.* MSc Thesis. University of London. London, England.

47. Coad, Dr. J.R. (1979). *Lime stabilised soil building blocks.* Building Research and Practice, Vol. 7. No. 2, International Council for Building Research, Studies and Documentation (CIB). Paris, France, pp 80-89.

48. Cinva-Ram. (1957). *Portable Block Making Press. General Information Series No. 4.* Scientific and Documentation Exchange Service, Inter-American Housing and Planning Center. Bogota, Colombia.

49. CIB/RILEM. (1983). *Appropriate Building Materials for Low Cost Housing. African Region (Vol. 1 + 11).* Proceedings of an International Symposium in Nairobi, November 1983. E. & F.N. Spon. London, England.

50. Craig, R.F. (1998). *Soil Mechanics.* 6th Ed. E&FN Spon. London, England.

51. CRA-Terre-EAG. (1991). *Earth Building Materials and Techniques, Select Bibliography.* German Appropriate Technology Exchange. Eschborn, Germany.

52. Dancy, H.K. (1975). *A Manual of Building Construction.* Intermediate Technology Publications. London, England.

53. Das, B.M. (1983). *Fundamentals of Soil Dynamics.* Elsevier Science Publishing Co. New York, USA.

54. Das, B.M. (1990). *Principles of Foundation Engineering.* 2nd Ed. PWS Publishing Company. Boston, USA.

55. Das, B.M. (1994). *Principles of Geotechnical Engineering.* 3rd Ed. PWS Publishing Company. Boston, USA.

56. Darrow, K. & Saxenian, M. (1986). *Appropriate Technology Sourcebook; a guide to practical books for village and small community technology.* A Volunteer in Asia Publication. Stanford, CA 94305, USA.

57. Darve, F. (1990). *Geomaterials; Constitutive Equations and Modelling.* Elsevier Science Publishers Ltd. Essex, England.

58. Davidson, F., Payne, G. (1983). *Urban Projects Manual. A guide to preparing upgrading and new development projects accessible to low income groups.* Liverpool University Press/Fairstead Press. Liverpool, England.

59. Delarne, J. (1954). *Etude de pise de ciment au Maroc.* Bulletin, RILEM. Paris, France.

60. Denyer, S. (1978). *African traditional Architecture.* Africana. New York, USA.

61. Doat, P. et al. (1979). *Construire en terre.* Editions Alternative et Paralleles. Paris, France.

62. DoHUD. (1955). *Earth for Homes. Ideas and Methods Exchange.* Office of International Affairs. US Department of Housing and Urban Development. Washington DC, USA.

63. DoHUD. (1979). *Handbook for Building Homes of Earth.* Office of International Affairs. US Department of Housing and Urban Development. Washington DC 20410, USA.

64. Dunlap, W.A. (1975). *US Airforce Soil Stabilisation Index System.* Airforce Weapons Laboratory. New Mexico, USA.

65. Eaton, K.J. (1981). *Buildings in tropical windstorms.* Building Research Establishment. Overseas Building Notes. Garston, England.

66. Eigenbrod, D. (1969). *Quantitative Slaking Test for Mudstones.* Department of Civil Engineering, University of Alabama. Int. Note SM5. Alabama, USA.

67. Ellison, W.D. (1944). *Studies of Raindrop Erosion.* Journal of Agricultural Engineering, Vol. 25, pp 131-136 & 181-182.

68. Enteiche, G. & Augusta, D. (1964). *Soil Cement: Its Use in Building.* United Nations Publications. New York, USA.

69. ESCAP/RILEM/CIB. (1987). *Building Materials for Low-income Housing. Asian and Pacific Region.* Proceedings of an International Symposium in Bangkok (January 1987). E. & F.N. Spon. London, England.

70. Evans, R. (1980). *Mechanics of water erosion and their spatial and temporary controls: an empirical viewpoint.* In Soil Erosion. John Wiley and Sons. Chichester, England.

71. Everett, A. (1984). *Materials.* Mitchels Building Series. Bastard Academic and Educational Ltd. London, England.

72. Fathy, J.M. (1973). *Architecture for the Poor.* An experiment in Rural Egypt. The University of Chicago Press. Chicago, USA.

73. Fitch, J.M., Branch, D.P. (1960). *Primitive architecture and climate.* Scientific American. New York, USA.

74. Fitzmaurice, R. (1958). *Manual on Stabilised Soil Construction for Housing.* Technical Assistance Programme. United Nations. New York, USA.

75. Fullerton, R.L (1979). *Building Construction in Warm Climates.* Vols. 1, 2 & 3. Oxford Tropical Handbooks. Oxford University Press. Oxford, England.

76. Franklin, J.A., Chandra, R. (1972). *The Slake Durability Test.* In The International Journal of Rock Mechanics. Min. Sci. Vol. 9. pp 325-341. Pergamon Press. London, England.

77. Gamble, J.C. (1971). *Durability-Plasticity Classification of Shales and other Argillaceous Rocks*. Ph.D Thesis, University of Illinois, Chicago, USA.

78. Games, K. Dr. (1981). *History and properties of adobe bricks.* New Scientist, IPC Magazine Ltd., June 1981, pp 670-68 1.

79. GATE. (1980). *Low-cost self help housing.* German Appropriate Technology Exchange. Eschborn, Germany.

80. Glanville, J., Neville, A. (Ed) (1997). *Prediction of Concrete Durability.* Proceedings of STATS 21st Anniversary Conference. The Geological Society, London, 16 November 1995. E & FN SPON. London, England.

81. Goldsmith et al. (1998). *Tropical Rain Forests - A Wider Perspective.* Chapman and Hall Ltd. London, England.

82. Goodman, R.E. (1980). *Introduction to Rock Mechanics*. John Wiley & Sons. USA.

83. Gooding, D.E.M. (1993). *Quasi-Static Compression Forming of Stabilised Soil-Cement Building Blocks.* Working Paper No. 40, Development Technology Unit, University of Warwick. Coventry, England.

84. Gooding, D.E.M. (1993). *Soil Testing for Soil-Cement Block Preparation.* Working Paper No. 38. Development Technology Unit, University of Warwick, Coventry, England.

85. Gooding, D.E.M. (1994). *Improved Processes for the Production Soil-Cement Building Blocks.* PhD Thesis. University of Warwick. Coventry, England.

86. Gooding, D.E., Thomas, T.H. (1995). *The Potential of Cement Stabilised Building Blocks as an Urban Building Material in Developing Countries.* Working Paper No. 44. Development Technology Unit. University of Warwick. Coventry, England.

87. Guillaud, H., Joffroy, T., Odul, P. (1995). *Compressed Earth Blocks: Volume 11. Manual of design and construction.* German Appropriate Technology Exchange (GATE) Publications. Eschborn, Germany.

88. Gummerson et al. (1980). *Water movement in porous building materials-II. Hydraulic Suction and Sorptivity of brick and other masonry materials.* Building Environment Vol. 15., pp 101-108.

89. Gunn, R., Kinzer, O.D. (1949). *The Terminal Velocity of Fall for Water in Stagnant Air.* Journal of Meteorology, Vol. 6, pp 243-248.

90. Greenfield et al. (1996). *Research Methods.* Guidance for Postgraduates. Arnold/Hodder Headline Group. London, England.

91. Grimshaw, R.W. (1971). *The Chemistry and Physics of Clays and Allied Ceramic Materials.* Ernest Benn Ltd. London, England.

92. Gwosdz, W., Sekivale, M. (1998). *Brick-earth and clay resources of eastern Botswana.* Geological Surveys Department. Lobatse, Botswana.

93. Hamdi, N. (1995). *Housing Without Houses: Participation, Flexibility, Enablement.* Intermediate Technology Publications. London, England.

94. Hammond, A.A. (1972). *Lateritic soils for rural housing.* Build International, Vol. 5(3). London, England.

95. Hammond, A.A. (1973). *Prolonging the Life of Earth Buildings in the Tropics.* Building Research and Practice. May/June 1973. Building and Road Research Institute. Kumasi, Ghana.

96. Hall, C. (1989). *Water Sorptivity of Mortars and Concrete: a review.* Magazine of Concrete Research 41, No. 147, pp. 51-61.

97. Head, K.H. (1980). *Manual of Soil Laboratory Testing.* Pentech Press. London, England.

98. Herbert, M.R.M. (1974). *Some observations on the behaviour of weather protective features on external walls.* Techniques and Materials. London, England.

99. Herzog, A., Mitchell, J. (1963). *Reactions accompanying the stabilisation of clay with cement.* 42nd Annual Meeting of the HRB. Washington, USA.

100. Hilt, G.H.; Davidson, D.L. (1960). *Lime fixation in clayey soils*. HRB Bulletin, National Academy of Sciences, National Research Council, Washington DC, 262, pp 20-32, USA.

101. Hood, M. (1996). *East Africa Handbook.* Footprint Handbooks. Passport Books. NTC Publishing Group. Chicago, Illinois, USA.

102. Houben, H., Guillaud, H. (1994). *Earth Construction: A Comprehensive Guide.* CRATerre-EAG. Intermediate Technology Publications. London, England.

103. Houben, H., Rigassi, V., Garnier, P. (1996). *Compressed Earth Blocks: Production Equipment.* 2nd Edn. CRATerre-EAG. Series Technologies Nr. 5. Brussels, Belgium.

104. Hudson, N.W. (1963). *Raindrop Size Distribution in High Intensity Storms.* Rhodesian Journal of Agricultural Research, 1 (1), pp 6-11.

105. Hughes, R. (1983). *Material and structural behaviour of soil constructed walls.* In Techniques and Materials. London, England.

106. Ingles, O.G. (1962). *Bonding Forces in Soils.* In the First Conference of the Australian Road Research Board. Sydney, Australia.

107. Ingles, O.G., Metcalf, LB. (1972). *Soil Stabilisation. Principles and Practice.* Butterworths Publications. Sydney, Australia.

108. ILO/UNIDO. (1984). *Small-scale Brickmaking.* Technology Series, Memorandum No.6. International Labour Organisation. Geneva, Switzerland.

109. IIHT. (1972). *The Manufacture of Asphalt Emulsion Stabilised Soil Bricks and Brick Makers Manual.* International Institute of Housing Technology. Fresno.

110. Illston, J.M. (Ed). (1994). *Construction Materials. Their Nature and Behaviour.* 2nd Edn. E. & F.N. Spon, Chapman & Hall. London, England.

111. ICE. (1980). *Appropriate Technology in Civil Engineering.* Proceedings of the Conference of the Institution of Civil Engineers, April 1980, Thomas Telford Ltd. London, England.

112. ISRM. (1971). *Suggested Methods for Determining the Slaking, Swelling, Porosity, Density and Prelated Rock Index Properties.* Commission on Standardisation of Laboratory and Field Tests. London, England.

113. Iterbeke, M., Jacobus, P. (1988). *Soil-cement Technology for Low-cost Housing in Rural Thailand. An Evaluation Study.* PG-CHS-KULeuven. Leuven, Belgium.

114. Jackson, N., Dhir, R.K. (Ed). (1996). *Civil Engineering Materials.* 5th Edn. McMillan Press Ltd. London, England.

115. Jagadish, K.S., Ventakarama, R.B. (1981). *A Manual of Soil Block Construction. Alternative Building Series-I,* Centre for Application of Science and Technology (ASTRA), Indian Institute of Science. Bangalore, India.

116. Jones, B. (1984). *A study of the particle size of sandy soils in the Central District and their suitability for production of soil-cement bricks using the CINVA-RAM.* Gabarone, Botswana.

117. Kahane, J. (1978). *Local Materials, a self-help builders manual.* Publications Distribution. London, England.

118. Keddi, J., Cleghorn, W. (1978). *Least Cost Brickmaking.* Appropriate Technology, Vol. 5, No. 3. Intermediate Technology Publications. London, England.

119. Keddi, J., Cleghorn, W. (1980). *Brick Manufacture in Developing Countries.* Scottish Academic Press Ltd. Edinburgh, Scotland.

120. Kerali, A.G. (1994). *Construction Industry in Uganda. Country Report.* In Compendium of International Construction Management '94. Lund Centre for Habitat Studies. Lund University. Lund, Sweden.

121. Kerali, A.G., Schmetser, H. (1995). *Master Action Plan for the Physical Rehabilitation of Makerere University.* African Development Bank-11/ GTZ-Germany Project. Kampala, Uganda.

122. Kerali, A.G. (1996). *Construction of 12 No. Senior Staff Houses and Civil Works at Muganzi Awongererwa.* In Technical Report to the Engineers Registration Board of the Republic of Uganda. Makerere University. Kampala, Uganda.

123. Kirkby, M.J., Morgan, R.D. (Eds). (1980). *Soil Erosion.* John Wiley & Sons. Chichester, England.

124. Kirkby, M.J. (1980). *Modelling water erosion processes.* In Soil Erosion. John Wiley & Sons. Chichester, England.

125. Laws, O.J. (1941). *Measurement of the fall-velocity of water-drops and raindrops.* In Transactions-American Geophysical Union. ISS 22 pp 709-721, USA.

126. Lawson, W. (1991). *Materials for Low Cost Building in North Eastern Nigeria.* Working Paper No. 35, Development Technology Unit, University of Warwick. Coventry, England.

127. Lea, F.M. (1970). *The Chemistry of Cement and Concrete.* 3rd Edn. Edward Arnold (Publishers) Ltd. London, England.

128. Lea, F.M. (1976). *The Chemistry of Cement and Concrete.* Edward Arnold Publishers Ltd. London, England.

129. Linsley, R.K., Kohler, M.A., Paulhus, J.L. (1975). *Hydrology for Engineers.* 2nd edtn. McGraw-Hill Series in Water Resources & Environmental Engineering. McGraw-Hill Book Company. Toronto, Canada.

130. Lola, C.R. (1981). *Research Efforts on Soil Cement Stabilisation for Low-Cost Housing in Nicaragua.* University of Tennessee. Knoxville, USA.

131. Lucky Lowe. (1998). *The Current Review Process of the Earth Building Standards in Africa.* Enabling Housing Standards and Procedures Project/CRATerre-EAG, January 1998. ITDG Publications. London, England.

132. Lunt, M.G. (1980). *Stabilised Soil Blocks for Building.* Overseas Building Note No. 184. Building Research Establishment. February 1980. Garston, England.

133. Lunt, M.G. (1980). *A Survey of Rural Housing in Sri Lanka.* Building Research Establishment. Overseas Building Note No. N104/80. Garston, England.

134. Maclean, J.H., Scot, J.S. (1995). *Penguin Dictionary of Building.* 4th Edn. Penguin Group, London, England.

135. Markus et al. (1979). *Stabilised Soil.* University of Strathclyde. Glasgow, Scotland.

136. Massey, B., Ward-Smith, J. (1998). *Mechanics of Fluids.* 7th Edn. Stanley Thornes Publishers Ltd. Cheltenham, England.

137. McIllveen, R. (1998). *Fundamentals of Weather and Climate.* Stanley Thornes Publishers Ltd. Cheltenham, England.

138. Minke, G. (1983). *Low cost housing. Appropriate construction techniques with loam, sand, and plant stabilised earth.* In Colloque L'habitat Economique dans les PED. Presses Ponts et Chaussees. Paris, France.

139. Mitchell, J.K., EI Jack, S.A. (1978). *The fabric of soil cement and its formation.* In 14th National Conference on Clays and Minerals. Sydney, Australia.

140. Mukerji, K. (1988). *Soil Block Presses. Product Information.* German Appropriate Technology Exchange (GATE). Eschborn, Germany.

141. Mukerji, K., Worner, H. (1991). *Soil Preparation Equipment.* German Appropriate Technology Exchange (GATE). Eschborn, Germany.

142. Mukerji, K. (1994). *Stabilisers and Mortars for Compressed Earth Blocks. Product Information.* German Appropriate Technology Exchange. GATE. Eschborn, Germany.

143. MoWHPP. (1985). *Kenya Low Income Housing Project Report.* Ministry of Works, Housing and Physical Planning. Nairobi, Kenya.

144. NASA. (1971). *Foldable Patterns for Construction Blocks.* In National Agency for Space Administration Technical Brief. Langley, Virginia, USA.

145. New Vision. (2001). *600 bags of fake cement seized*. Article in the New Vision Newspaper of 12th June, 2001. New Vision Printing and Publishing. Kampala, Uganda.

146. Newey, C., Weaver, G. (Eds.). (1990). *Materials Principles and Practice.* The Open University. Betterworths/Heinmann Publications. Oxford, England.

147. Newman, A.J. (1986). *Microclimate and its effiects on durability*. Paper to SCI/BBA Symposium on Building Deteriology, April 1986. Garston, England.

148. Neville, A.M., Brooks, J.J. (1994). *Concrete Technology*. Longman Scientific and Technical. London, England.

149. Neville, A.M. (1995). *Properties of Concrete.* 4th Edn. Addison Wesley Longman Ltd. Essex, England.

150. Norton, J. (1986). *Building with Earth. A Handbook.* IT Publications Ltd. London, England.

151. Norton. J. (1997). *Building with Earth. A Handbook.* 2nd Edn. Intermediate Technology Publications Ltd. London, England.

152. Odul, P. (1984). *Case Studies on Earth Construction: Synthesis.* PGC-HS. Katholieke Universiteit. Leuven, Belgium.

153. Okello, W. (1989). *Namuwongo Low Cost Housing and Slum Upgrading Project Report.* World Bank/Ministry of Housing Project. Kampala, Uganda.

154. Ola, S.A. (1977). *The Potential of Lime Stabilisation of Lateritic Soils*. Elsevier Scientific Publishing Company. Amsterdam, Netherlands.

155. Ola, S.A., Mbata, A. (1990). *Durability of Soil-cement for Building Purposes-Rain Erosion Resistance Test.* In Construction and Building Materials. Volume 4 No. 4. December 1990. Butterworth-Heinemann Ltd. London, England.

156. Oliver, P. (Ed). (1971). *Shelter in Africa.* Barrie and Jenkins Ltd. London, England.

157. Oliver, P. (1987). *Dwellings-the House Across the World.* Phaidon Press Ltd. Oxford, England.

158. Ortega, A. (1985). *Basic Technology: Lime and its Production.* Mimar 17. Concept Media. Singapore.

159. Parma, R.P., Nimityongskul, P., Cook, D.J. (Eds). (1978). *Materials of Construction for Developing Countries.* Vol. 1 & 11. Proceedings of the International Conference at AIT. Bangkok, Thailand.

160. Parry, J. (1979). *Brickmaking in Developing Countries.* Review for Overseas Division, Building Research Establishment. Garston, England.

161. Perera, A. (1993). *Landcrete Blocks: Cement Stabilised Lateritic Soil Blocks.* Department of Civil Engineering, University of Moratuwa. Moratuwa, Sri Lanka.

162. Pettijohn, F.J. (1957). *Sedimentary Rocks.* Hopper and Brothers Ltd. New York, USA.

163. Plinchy. F. (1982). *Earth Block Manufacturing and Construction Techniques.* 2nd Regional Conference on Earthen Building Materials, University of Arizona. Tucson, USA.

164. Portland Cement Association. (1971). *Soil-cement Laboratory Handbook.* PCA. Chicago, USA.

165. Portland Cement Institute. (1970). *Soil-cement Series.* PCI Publication. Johannesburg, South Africa.

166. Portland Cement Association. (1971). Soil-cement Laboratory Handbook. PCA. Chicago, USA.

167. Ray, M.S. (1992). *Engineering Experimentation: Ideas, Techniques, and Presentation.* McGraw Hill Book Company. London, England.

168. Riedjik, W. (Ed.) (1984). *Appropriate Technology for Developing Countries.* Delft University Press. Delft, Netherlands.

169. Rigassi, V. (1995). *Compressed Earth Blocks: Manual of Production.* Vol. 1. CRATerre-EAG. Vilefontaine Cedex, France.

170. RILEM. (1972). *Baked Clay Masonry Units, Testing Methods.* Natural and Artificial Stones Technical Commission, Materials and Structures. Vol. 5, pp 231-259.

171. Ringsholt, T., Hansen, T.C. (1978). *Lateritic soil as a raw material for building blocks.* American Ceramic Society Bulletin. Vol. 57(5). Washington, USA.

172. Roberson, J.A., Crowe, C.T. (1997). *Engineering Fluid Mechanics.* 6th Edn. John Wiley & Sons Inc. Toronto, Canada.

173. Ruskulis, O. (1997). *The Basics of Concrete Building Products.* German Appropriate Technology Exchange. GATE Publications. Braunschweig. Germany.

174. Spence, R.J.S., Cook, D.J. (1983). *Building Materials in Developing Countries.* John Wiley & Sons Ltd. Chichester, England.

175. Spence, R.J.S. (1975). *Predicting the Performance of Soil-cement as a Building Material in Tropical Countries.* In Building Science. Volume 10. Pp 155-159. Pergamon Press. London, England.

176. StuIz, R., Mukerji, K. (1988). *Appropriate Building Materials: A Catalogue of Potential Solutions.* Revised & enlarged edition. Swiss Centre for Appropriate Technology. St. Gallen, Switzerland.

177. Smith, R.G., Webb. D.T.J. (1987). *Small-scale Manufacture of Stabilised Soil Blocks.* Technical Memorandum No. 12. International Labour Organisation. Geneva, Switzerland.

178. Schildermann, T. (1998). *Standards and Specifications.* In Building Advisory Service and Information Network. Basin News No. 15. May 1998. SKAT Publications. St. Gallen, Switzerland.

179. Schumacher, E.F (1973). *Small is Beautiful: A study of Economics as if People Mattered.* Blond and Briggs Ltd. London, England.

180. Scot, R.F. (1963). *Principles of Soil Mechanics.* Addison-Wesley Publishing Inc. London, England.

181. Sjostrom et al. (Ed.). (1996). *Durability of Building Materials and Components 7: Prediction, Degradation and Materials.* Proceedings of the Seventh International Conference on Durability of Building Materials and Components. 7 DBMC held in Stockholm, Sweden 19-23 May 1996, Vol. 1, May 1996. E & FN SPON. London, England.

182. Sjostrom et al. (Ed.). (1996). *Durability of Building Materials and Components 7: Testing, Design and Standards.* Proceedings of the Seventh International Conference on Durability of Building Materials and Components. 7 DBMC held in Stockholm, Sweden 19-23 May 1996, Vol. 2, May 1996. E & FN SPON. London, England.

183. Smith, R.C. (1973). *Materials of Construction.* 2nd Edn. International Student Edition. McGraw Hill Kogakusha Ltd. Sydney, Australia.

184. Smith, R.G., Webb, D.T.J. (1987). *Brepak Block Making Machine.* Building Research Establishment. Garston, England.

185. Smith, G.N., Smith, I.N. (1998). *Elements of Soil Mechanics.* 7th Edn. Blackwell Science Limited. Cambridge, England.

186. Spence, R.J.S. (1979). *Appropriate Technologies for Small Scale Production of Cement and Cementitious Materials.* Intermediate Technology and Industrial Services. London, England.

187. Spence, R.J.S. (1980). *Small-scale Production of Cementitious Materials.* ITDG Publications Ltd. London, England.

188. Spence, R. (1985). *Lime in Industrial Development.* United Nations Industrial Development Organisations. Sectoral Series No. 18. Vienna, Austria.

189. Suare, T.I. (1974). *Stabilised Soil Blocks.* Building Research Unit Data Sheet, BRU. Dar es Salaam, Tanzania.

190. Sutton, B.H.C. (1993). *Solving Problems in Soil Mechanics.* 2nd Edn. Addison Wesley Longman Limited. Essex, England.

191. Taylor, K., Cotton, A. (1994). *Urban Upgrading.* WEDC/GHK/MRM International for ODA, Loughborough University of Technology. Loughborough, England.

192. Taylor, H.F.W. (1998). *Cement Chemistry.* 2nd Edn. Thomas Telford, London, England.

193. Tetley et al. (1998). *Spectrum Guide to Uganda.* Camerapix Publishers International. Nairobi, Kenya.

194. The Underground Space Centre. (1979). *Earth Sheltered Housing Design.* Van Nostrand Rheinhold Company. New York, USA.

195. Torraca, G. (1988). *Porous Building Materials. Materials Science for Architectural Preservation.* 3rd Edn. ICCROM. Technical Notes. Rome, Italy.

196. Tibbets, J.M. (1982). *The Pressed Block Controversy.* In Adobe Today. Adobe News. Albuquerque, New Mexico.

197. United Nations. (1950). *Housing, Town and Country Planning.* Bulletin No. 4. United Nations Publications. New York, USA.

198. United Nations. (1964). *Soil-cement: Its Use in Building.* Sales No. 64. IV.6. United Nations Publications. New York, USA.

199. United Nations Conference for Human Settlements. (1976). *Report of HABITAT,* A/CONF. 70/15. Vancouver, Canada.

200. United Nations Industrial Development Organisation. (1980). *Appropriate Industrial Technology for Construction and Building Materials.* Monographs on Appropriate Industrial Technology, No.12, UNIDO. New York, USA.

201. United Nations Centre for Human Settlements. (1981). *Construction with Sisal-cement.* Technical Notes, UNCHS (HABITAT). Nairobi, Kenya.

202. Uzomaka, O.J. (1978). *Performance Characteristics of Plain and Reinforced Soil Blocks.* In Proceedings of the International Conference on Materials of Construction for Developing Countries, AIT. Bangkok, Thailand.

203. Van Den Branden, F., Hartsell, T. (1971). *Plastering Skill and Practice.* American Technical Society. Chicago, USA.

204. Van Griethuysen, A.J. (Ed.). (1988). *New Applications of Materials*, Forecasting and Assessment in Science and Technology (FAST), The Netherlands Study Centre for Technology Trends (STT), Scientific & Technical Press Limited. The Hague, The Netherlands.

205. Van Olphen, H. (1977). *An Introduction to Clay Colloid Chemistry*. John Wiley and Sons. New York, USA.

206. Vickers, B. (1983). *Laboratory Work in Soil Mechanics.* 2nd Edition. Granada Publishing Ltd. London, England.

207. Volunteers in Technical Assistance. (1977). *Making Building Blocks With the CINVA-Ram Block Press,* VITA. Mt. Rainer, Maryland, USA.

208. Vohauer, K. (1979). *Low Cost Self-Help Housing.* German Appropriate Technology Exchange, GATE-Modul 616. Eschborn, Germany.

209. Walton, D. (1995). *Building Construction Principles and Practices.* The Motwate Series. Macmillan Education Ltd. London, England.

210. Webb, D.T.J. (1983). *Stabilised Soil Construction in Kenya.* Proceedings of the International Conference on Economic Housing in Developing Countries, UNESCO/RILEM, 25-27 January 1983, Paris, France.

211. Webb, D.T.J., Lockwood, A.J. (1987). *BREPAK Operators Manual.* Overseas Development Administration. Building Research Establishment Client Report. Department of the Environment. Garston, England.

212. Webb, D.T.J. (1988). *Stabilised Soil Building Blocks.* PhD Thesis, University of Newcastle-upon-Tyne. Newcastle, England.

213. Weidemann et al. (Ed.). (1990). *Structural Materials,* Materials Department, Open University. Butterworths. London, England.

214. White, F.M. (1999). *Fluid Mechanics.* 4th Edn. McGraw-Hill Companies Inc. Boston, USA.

215. Williams, W. (1980). *Construction of Homes Using On-Site Materials.* In International Journal IAHS, Pergamon Press. New York, USA.

216. Wilson, E.M. (1993). *Engineering Hydrology*. 4th Edn. Macmillan Press Ltd. Kent, England.

217. Wischmeier, W.H., Smith, D.D. (1958). *Rainfall Energy and its Relationship to Soil Loss.* In Transac American Geophysics Union. No. 39(2). Pp 285-291. Chicago, USA.

218. Young, L.F. et. al. (1998). *The Science and Technology of Civil Engineering Materials.* Prentice-Hall, New Jersey, USA.

APPENDIX A

BASIC CHEMICAL CONSTITUENTS OF OPC

S/N	Compound Name	Shorthand Nomenclature	Mineral Name	Density Kg/m^3	Typical Quantity by Weight %	Role
1	Tricalcium silicate	C_3S	Alite	3150	55	The major constituent in OPC; involved in the initial gel formation contributing to setting; hydration products are C-S-H fibres and $Ca(OH)_2$ crystals; contribute to strength in the early stages of hardening.
2	Dicalcium silicate	C_2S	Belite	3280	20	Same hydration products as above; contributes to increase in strength at later stages of hardening due to its slower rate of hydration.
3	Tricalcium aluminate	C_3A	Aluminate	3030	12	Contributes to setting through gel and ettringite formation due to its fast rate of hydration, but little to hardening.
4	Tretracalcium alumino ferrite	C_4AF	Ferrite	3770	8	Contributes to colour of cement, but plays little part in setting and hardening
5	Hydrated calcium sulfate	$C\bar{S}H_2$	Gypsum	2320	3.5	Controls hydration rate of C_3A; own rate of hydration very fast
6	Alkali oxides, other impurities	K_2O, Na_2O, CaO	-	-	1.5	May affect the crystal structure and reactivity or both of 1-5 above; Na_2O and K_2O may react with soils containing silica to cause ASR

(Adapted from: Weidemann et al, 1990; Young et al, 1998; Lea, 1976; Taylor, 1998)

APPENDIX B

PROPERTIES OF HYDRATION PRODUCTS OF OPC AND THEIR POTENTIAL INFLUENCES ON THE DURABILITY OF CSBs

S/N	Product		Volume Fraction	Density	Particle Size Across	Particle Size Thick	Specific Surface Area	Morphology and Crystallinity	Strength	Impact on durability of CSBs
	Symbol	**Name**	**%**	**Kg/m^3**	**μm**	**μm**	**m^{2}g^{-1}**			
1	C-S-H	Calcium sulphate hydrate	65	2000	<1	0.01	400	Irregular foils Amorphous Microporous	Provides major cohesive force but is weak due to its microporosity. This is why dry blocks will be stronger than wet blocks (stronger van der Waal forces)	Very insoluble. Water loss from its micropores will cause shrinkage or drying and creep on loading even at room temperature. Responsible for drying shrinkage in CSBs and creep respectively.
2	$Ca(OH)_2$	Calcium hydroxide	20	2250	100	10	~ 0.5	Thick hexagonal plates which cleave easily and are crystalline	Contributes to strength in CSBs reducing porosity. Cleavage tends to limit levels of high strength pastes. Is dimensionally stable and will restrain C-S-H deformations.	Blocks capillary pores hence lowering permeability in blocks. Very soluble in water, especially in presence of CO_2. It is slowly leached out by water causing increase in porosity, permeability and reduction in strength.
3	$C_4A\bar{S}H_{12}$	Monosulpho-aluminate	10	1950	~ 2	~ 0.1	~ 2	Thin irregular plates and farily crystalline	Reduces porosity but not significantly. Has minimum effect on deformation	Responsible for causing sulphate attack by reforming ettringite and causing expansion.
4	UCR	Unhydrated cement residues	5	3150	~ 1	-	~ 0.1	Remnants of original cement grains	Not very significant but may restrain C-S-H deformation	Renewed hydration may cause autogeneous healing of internal micropores
5	CP GP	Capillary pores; Gel pores	-	-	-	-	-	Openings	Total porosity is the major factor influencing strength. Fine pores contribute to shrinkage and creep	Porosity influences permeability. Large interconnected pores facilitate circulation of moisture in blocks. These can catalyse chemical reactions.

(Adapted from: Young et al, 1998)

APPENDIX C

COMPARISON OF EXISTING SOIL SUITABILITY CRITERIA

S/N	Author	Year	Basis of Criteria	Details
1	Fitzmaurice	1958	Particle size distribution Plasticity Compactability Simplified particle size distribution test	Recommendation: 33-40% sand (min.) 20-30% clay (max.) Limit: not ≮ 5% clay OPC: 5-10% Liquid limit: 40-50% Plasticity index: Less than 22% and more than 2.5% Optimum moisture content: 10-14% (urban) 7-16% (rural) Limits: not >30% silt and clay not < 70% gravel and sand
2	United Nations	1964	Particle size distribution	Optimum: 75% sand 25% silt and clay clay not < 10% Limits: 45% sand (min.) 55% silt and clay (min.) 80% sand (max.) 20% silt and clay (max.) OPC: 4-12% by volume
3	Spence and Cook	1983	Particle size distribution Plasticity	Range: Sand 60-90% Silt 10-40% Clay 0-30% Range: Liquid limit 7-40% Plasticity index 0-20%
4	Webb and Lockwood	1987	Linear Shrinkage (Alcocks Mould)	Shrinkage limits: < 15 mm not recommended 15-30 mm recommended (use 1:20/C:S) 30-45 mm recommended (use 1:15 C:S or 1:7 L.S) 45-60 mm recommended (use 1:12 C:S or 1:6 L:S) > 60 mm not recommended. Insufficient sand Advantage: • Various soil combinations can be tested for shrinkage • Guidelines for stabiliser content given
5	ILO	1987	Particle size distribution	Limits: None specificed Recommendation: Well graded soil of max. size < 6 mm

6	Stulz and Mukerji	1988	Particle size distribution	Optimum: Gravel 7% Sand 53% Silt 20% Clay 20%
			Plasticity	Plasticity index 7-29% Liquid limit 25-50% Caution: Lateritic soils may not conform to these limits
7	Houben and Guillaud	1994	Particle size distribution Plasticity Compactability Cohesion	Range: Clay 5-20% Silt 5-40% Sand 40-90% Limits: Plasticity index 3-30% Liquid limit 24-37% Dry density 1700-2400 kg/m^3 Corresponding moulding moisture content: 4-10% Maximum acceptable load 0.3-0.6 MPa Cohesion 15,000-36,000 Pa
8	Rigassi	1995	Particle size distribution Plasticity	Recommended: Gravel 0-40% Sand 25-80% Silt 10-25% Clay 8-30% Stabiliser: OPC 4-8% not < 3% For clay content 30-70% use lime Plasticity index 15-20%
9	Houben et al	1996	Particle size distribution	Range: Gravel 0-40% Sand 25-80% Silt 10-25% Clay 8-30% Recommended: Other tests be done as well OPC: Optimum 5-6% Maximum 8% Minimum 2% Caution: Clay not > 30%
10	Norton	1997	Particle size distribution Plasticity	Recommended: Gravel/sand 45-70% Silt 15-30% Clay 10-30% OPC: 5-10% not > 10% Plasticity index 10-25% Liquid limit 25-42%

APPENDIX D

DETERIORATION AGENTS AND THEIR SEVERITY RANKING (UGANDA)

S/N	Category	No	Agent	Severity Ranking	Source	Type of Action	Effect	Affected Property		Speed	Common Defect Type
								Surface	Bulk		
A	Environmental	1	Water Liquid Rain	*I*	Rain	Abrasion Wetting Penetration Solvent Catalytic	Erosive wear and tear Dampness Swelling Softening Dissolution Chemical reactions	•	•	Fast	Pitting, Roughening Mass loss Volume Reduction Moulding Volume Change
			Rising damp	*III*	Ground water	Wetting Solvent Catalytic	Dampness Swelling Softening Chemical reactions				"
			Condensation	*III*	Users	Wetting Solvent Catalytic	Dampness Chemical reactions				"
			Vapour Humidity	*II*	Atmosphere	Wetting Catalytic	Creation of moisture gradient				"
		2	Temperature	*I*	Atmosphere	Reversible Warming Cooling Irreversible Catalytic Drying	Volumetric expansion and contraction Contraction Chemical Reaction Shrinkage	•		Fast	Cracks
		3	Radiation Solar Thermal	*III*	Sun CSB	Heat absorption (heating) Heat emission (cooling)	Volumetric expansion Lowering temperature	• •	•	Fast	Cracks
		4	Wind	*II*	Atmosphere	Driving rains and particles Differential pressure	Rain penetration Loosening particles	• •		Fast	Pitting, Roughening
		5	Air Carbon dioxide	*III*	Atmosphere	Acid solution formation Alkalinity Neutralisation Catalytic (leaching)	Bond weakening	•	•	Slow	Porous residues Mass loss
			Oxygen	*III*	Atmosphere	Catalytic Oxidisation	Bond weakening				
			Gases Nitrogen oxide & Sulphur dioxide	*III*	Atmosphere	Dissolution in H_2O to form acidic conditions	Bond weakening				
			Particulates dust/grit	*IV*	Atmosphere	Accumulation in pores Other chem. reactions Deposition	Bond weakening				

B	Chemical	6	Sulfates	*II*	Soil	Expanded product formation within cement paste	Build-up of internal stresses Bond weakening Disintegration	•	•	Slow	Cracks Mass loss Disintegration Porous residues
		7	AAR ASR ACR	*III*	 Sand Clay	 Gel formation, swelling in presence of H_2O "	 Build up of expansive forces Bond weakening "				
		8	Soluble Salts	II	Soil	Crystallisation within pores	Volume changes of salt crystals induce internal stresses				
		9	Acids	*I*	Soil Groundwater	Dissolution of hydrated cement and $Ca(OH)_2$	Bond weakening				
		10	Calcium Hydroxide	*II*	Cement paste	Dissolution in water followed by leaching out	Segregation Porosity increase				
		11	Clay	*II*	Soil	Hydrophilic attraction of water	Swelling Loss of bonding				
C	Biological	12	Plants	*III*	Seeds	Penetration	Bond weakening Disintegration	•	•	Slow	Surface cracks Deep cracks & crevices
		13	Insects	*III*	Larvae	Boring	Bond weakening Disintegration	•	•	Slow	Deep holes

LEGEND:	Speed		Severity Ranking
	Fast: 1-3 years	I	Very severe
	Moderate: 3-5 years	II	Severe
	Slow: >5 years	III	Low severity

APPENDIX E

RESULTS OF THE VISUAL OBSERVATION RECORD OF DEFECTS IN CSB BUILDINGS AND DIAGNOSIS OF LIKELY CAUSES (UGANDA, JANUARY-MARCH, 2000)

S/N	DEFECT TYPE	Fraction of buildings (%)	CAUSE										WALL						AGE OF BUILDING		REMARKS
			Rain abrasion	Rain softening	Temperature	Relative humidity	Chemical action	Biological action	Handling	Workmanship	Curing	Settlement	FACADE		SECTION				YEARS		
													N-S	E-W	U	M	L	C	1-5	5-12	
1	Surface erosion	75	•										***	***	*	**	***	***	•	•	Mostly in the rainy season
2	Surface pitting	72	•										***	***	*	**	***	***	•	•	"
3	Surface roughening	74	•	•									***	***	*	**	***	***	•	•	"
4	Surface spalling	16		•			•				•		***	***	*	**	***		•	•	"
5	Surface growth	3				•		•					*	*	*	*	*			•	"
6	Surface cracking	21			•		•	•			•	•	**	***	*	**	`**		•	•	All seasons
7	Surface crazing	17			•		•				•		**	***	*	**	**		•	•	"
8	Bulk cracking	21			•		•	•			•	•	*	***	*	*	*			•	"
9	Chipped edges	25							•	•			*	*	*	*	*	***	•	•	Due to handling or transportation
10	Loose material residue	16		•			•	•					*	*			***	***		•	All seasons
11	Plant growth	1				•		•					*	*	*	*	*	*		•	"
12	Peeled off plaster	9				•					•		*	*			***	***		•	"
13	Inter block/mortar cracking	2										•	*	*	*	*	*	*		•	Foundation settlement
14	Other	1																			

KEY: Denotes defect observed •
Severity ranking: * low ** medium *** high

Façade:
N-S North-South
E-W East-West

Wall section:
U Upper
M Middle
L Lower
C Corner

APPENDIX F

COMPREHENSIVE SUMMARY LIST OF CURRENTLY AVAILABLE SOIL INDICATOR TEST TYPES

S/N	TEST NAME	AUTHOR AND YEAR OF PUBLICATION										
		1	2	3	4	5	6	7	8	9	10	11
		DoUHD 1955	Fitzmaurice 1958	United Nations 1964	ILO 1987	Webb & Lockwood 1987	Webb 1988	Stulz & Mukerji 1988	Gooding 1994	Houben & Guillaud 1994	Rigassi 1995	Norton 1997
1	Visual	•	•	•				•	•	•		•
2	Odour (smell)	•			•		•	•	•	•	•	•
3	Touch	•	•					•	•	•	•	•
4	Nibble							•		•		
5	Washing	•						•		•	•	
6	Cube (disc)				•	•		•		•	•	
7	Lustre (shine)	•		•	•			•	•	•		•
8	Adhesion	•						•		•		
9	Water retention (surface water)	•		•	•		•	•	•	•		
10	Dry strength	•		•				•	•	•		•
11	Thread (rolling/consistency)	•			•	•	•	•	•	•	•	•
12	Ribbon (cohesion)	•	•	•			•	•	•	•	•	•
13	Sedimentation (jar/bottle)		•	•	•	•	•	•	•	•	•	•
14	Decantation						•		•	•		
15	Linear Shrinkage		•	•	•	•	•	•	•	•		•

Key: • indicates that the test was described by the author

APPENDIX G

SUMMARY OF FIELD INDICATOR SOIL TEST RESULTS FOR TWO CSB PROJECT SITES IN UGANDA (JANUARY-MARCH, 2000)

S/N	TEST NAME	UNITS	RESULTS		INTERPRETATION	
			NAMUWONGO (B)	MALUKHO	NAMUWONGO (B)	MALUKHU
1	Visual test	-	Dark red-brown soil Large sand content	Dark brown-grey soil Moderate sand content	Silty sand	Clayey sand
2	Smell test	-	Non-musty smell (even on wetting)	Non-musty smell (even on wetting)	No significant presence of organic matter	No significant presence of organic matter
3	Touch test	-	Rough sensation felt on rubbing Moderate cohesion	Rough sensation felt on rubbing More cohesive: lumps sticky when moist	Silty sand	clayey sand
4	Nibble test	-	Gritty sensation	Gritty and floury sensation	Silty sand	Clayey sand
5	Washing test	-	Hands easy to rinse, but powdery sensation felt	Hand difficult to rinse clean Soapy sensation	Silty sand	Clayey sand
6	Cube test	-	Forms cube on moulding Breaks easily on drying	Forms cube on moulding Breaks with difficulty on drying	Silty sand	Clayey sand
7	Lustre test	-	Freshly cut surface of ball sphere is dull	Freshly cut surface of ball sphere is shiny	Silty sand	Clayey sand
8	Adhesion test	-	Easy penetration by knife No sticking on to knife on withdrawal	Penetration of knife with difficulty. Soil sticks on to knife on withdrawal	Silty sand	Clayey sand
9	Water retention test	taps	5-10 Ball partially crumbles	20-30 Ball flattens on pressing	Fine sand and silt present	Silt and clay present
10	Thread test	-	Medium hard thread. Reconstituted ball tends to crack and crumble	Hard thread. Reconstituted ball difficult to crush. Does not crumble	Low clay content	High clay content
11	Ribbon test	cm	5-10 Short ribbon	24-30 Long ribbon	Low to medium clay content	High clay content
12	Sedimentation test	% % %	6 (gravel) 70 (sand) 24 (silt and clay)	14 (Gravel) 61 (sand) 35 (silt and clay)	Low gravel content High sand content Medium fines content	Low gravel content Moderate sand content High fines content
13	Linear shrinkage test	mm	23	45	Soil suitable for CSB production. Recommended: 1:20 cement: soil	Soil suitable for CSB production. Recommendation: 1:12 cement : soil, or 1:6 lime : soil

LABORATORY TEST RESULTS FOR NAMUWONGO CSB SLUM UPGRADING PROJECT (UGA 186/005)

S/N	TEST TYPE	UNITS	NAMUWONGO SITE		
			A	**B**	**C**
	A. Laboratory Soil Test Results				
1	Particle size distribution				
	Gravel	%	8	2	5
	Sand	%	68	70	70
	Silt	%	12	13	3
	Clay (+ fine silt)	%	12	15	22
2	Linear shrinkage (mean)	mm	21	13	10
3	Sedimentation (Bottle test)				
	Gravel	%	10	15	5
	Sand	%	60	60	75
	Silt and Clay	%	30	25	20
4	Natural moisture content	%	14	16	16
5	Soil type	-	Lateritic soil (or dark grey coffee soil	Silty sand (murrum)	Silty sand (sand)
	B. Stabiliser Selection				
1	Cement (only)	%	6	5	4
2	Lime (only)	%	5	5	5
	C. Initial Performance Tests				
1	Block sizes (mean)	mm mm	290 x 140 x 88 220 x 107 x 70	290 x 140 x 88 220 x 107 x 70	290 x 140 x 88 220 x 107 x 70
2	Wet compressive strength (mean) R_c28				
	Cement blocks	MPa	5.1	3.9	4.1
	Lime blocks	MPa	3.0	2.9	1.5
3	Water absorption (mean)				
	Cement blocks	%	12.0	8.0	10.3
	Lime blocks	%	10.2	12.3	-

(Source: Okello, 1989; MoWHUD, 1992)

APPENDIX I

SUMMARY OF FINDINGS FROM VISITS TO BLOCK PRODUCTION SITES IN UGANDA

S/N	PROCESS and SUB PROCESSES	OBSERVATIONS / NOTES	
		MALUKHU	TEMANGALO
1	Soil Extraction and Preparation		
	Extraction		
	Adequacy of soil pre-determined	x	x
	Soil test records available (field and laboratory)	x	x
	On-site soil used	●	●
	Sub-soil extraction	●	●
	Top-soil extraction	x	x
	Manual extraction	●	●
	Mechanical extraction	●	x
	Drying		
	In sheltered area	x	x
	In the open yard	●	●
	Spreading out in thin layers	x	x
	Turning over	x	x
	Uniform colour check for drying	x	x
	Supervision	x	x
	Storage		
	In open yard	●	●
	In sheltered area (ventilated)	x	x
	Pulverising		
	Manual (wooden hammers)	x	x
	Mechanised	x	x
	Screening		
	Fixed inclined screen (5-20 mm)	●	●
	Suspended screen (5-20 mm)	x	x
	Extra removal check by hand	x	x
	Stockpiling		
	Sheltered area	x	x
	Open area	●	●
	Controlled mixing to modify	x	x
2	Mixing (soil, stabiliser, water)		
	Proportioning out		
	By mass	x	x
	By volume	●	●
	Batching (per day)	●	●
	Batching (per hour)	x	x
	Levelling off	x	x
	Dry physical state (soil, stabiliser)	x	x
	Dry mixing		
	On clean/hard ground surface	x	x
	On the open yard (grass, soil)	●	●

	Mechanical	x	x
	Manual	●	●
	Spread out soil (plus stabiliser)	x	x
	Heaped soil (plus stabiliser)	●	●
	Supervision	x	x
	Wet mixing	x	x
	On clean/hard ground surface	x	x
	On open yard (grass, soil)	●	●
	Mechanical	x	x
	Manual	●	●
	Water added by spray	x	x
	Water added by pouring	●	●
	Uniform mix colour check done	x	x
	Drop test check (OMC)	x	x
	Supervision	x	x
	Reaction time		
	Moulded within 45 minutes (OPC)	x	●
	Moulded within 24 hours (lime)	●	x
	Supervision	x	x
3	<u>Compression</u>		
	Measuring out		
	Controlled amount pre-measured	x	x
	Fixed volume box used	x	x
	Protected mix	x	x
	Unprotected mix	●	●
	Supervision	x	x
	Filling		
	Mould interior cleaning	x	x
	By hand	●	●
	By spade	●	x
	In layers	x	x
	Corners checked, pressed	x	x
	Topping up, removal	x	x
	Levelling off	x	x
	Correct filling check	x	x
	Periodic repeat cleaning	x	x
	Supervision	x	x
	Moulding		
	Manual press	x	x
	Motorised press	●	●
	Mould pressure check	x	x
	Solid blocks	x	x
	Hollow blocks	x	x
	Bed frogged blocks	x	x
	Interlocking blocks	●	●
	Output > 2000 per day	●	x
	Output < 2000 per day	x	●
	Same day moulding	●	●
	Supervision	x	x

	Demoulding / handling		
	Automatic	x	x
	By hand removal	•	•
	By timber pieces removal	x	x
	By pincer removal	x	x
	Curing area close by	•	•
	Supervision	x	x
	Quality checks		
	By batch	x	x
	All blocks	x	x
	None	•	•
	Appearance	x	x
	Weight	x	x
	Dimensions	x	x
	Bulk density	x	x
	Surface penetration	x	x
	Parallelism	x	x
	Corners and edges	x	x
	Supervision	x	x
4	Curing		
	Wet curing		
	Close to mould site	•	•
	On hard surface	x	x
	Polythene sheet cover	x	x
	Elephant grass cover	•	•
	Sheltered area	x	x
	By stabiliser specification	x	x
	In separated batches	x	x
	Marking	x	x
	Dry curing		
	On open yard	•	•
	In sheltered area	x	x
	Duration check	x	x
	Supervision	x	x
	Stockpiling		
	Near machine side	•	•
	Near building side	x	x
	Covered	x	x
	Uncovered	•	•
	Testing		
	WCS, BDD, TWA	x	x
	Use		
	Walling with render	•	•
	Without render	•	•
	Without mortar	•	•
	Surface coating	x	•

Key: • process observed/noted
x process not observed/noted

FIELD AND LABORATORY TESTING

SEDIMENTATION TEST (BOTTLE)

TEST TITLE:	**Sedimentation (jar or bottle) Test**
Standard:	Stulz and Mukerji (1988), Houben and Guillaud (1994).
Objective:	To determine quantitatively the approximate relative proportions of the main fractions in a soil sample.
Precision:	Low to medium accuracy.
Limitations:	It is difficult to precisely discriminate the boundaries of the grain layers, which may not always be linear. The resettling movement of sand, but more especially silt and clay can affect the results if they are taken too early. The volume of the silt and clay is slightly increased due to swelling and expansion of the particles in water. They will therefore appear to be larger than they really are.
Duration:	3 to 24 hours

APPARATUS

1. Transparent cylindrical glass jar (65 mm diameter, of flat bottom with the top sealable by the palm of the hand).
2. Clock or stopwatch.
3. Centimetre scale.
4. Clean drinking water.

TEST PROCEDURE

(i) Take a representative sample of the soil and place it into the glass jar until it is about one quarter full. Fill some of the remaining three-quarters of the jar with clean drinking water, leaving just enough space at the top to allow agitation.

(ii) Leave the bottle and its soil and water content standing undisturbed so that the soil can soak in the water for about 60 minutes.

(iii) After 60 minutes have elapsed, firmly cover the top opening of the jar and shake vigorously for between 1 and 3 minutes, then replace the bottle and its contents on a flat horizontal surface. Repeat the process again an hour later, then leave the jar standing undisturbed for at least 45 minutes. After this time, the soil fractions should begin to segregate with the heavier fraction (fine gravel and sand) settling at the bottom of the bottle. The silt, clay and organic matter fractions will settle at the top of each other in that order of lightness. Organic matter will float at the surface of the water, while the finer colloids will remain in suspension in the water.

(iv) Allow up to 8 hours before measuring the precipitated height of the segregated layers using an accurate centimetre scale. First measure the overall depth of the sediments (100%) without including the depth of the clear water covering them. Then measure the height of each fraction layer separately and record it as a percentage of the total depth. Take three measurements for each layer and record the average for the sand, silt and clay.

Results are discussed in Chapter 4 and 5.

INTERPRETATION AND RECOMMENDATIONS

The depth of each separated layer provides an indication of the relative proportions of each of the main soil constituents in the sample tested. If the results show an even distribution of sand, silt and clay, then the soil is suitable for CSB production. If the results reveal an excess or absence of either sand, silt or clay, then the soil is unlikely to be suitable for stabilisation without further modification as before. The separation of the soil fractions can be further facilitated by using a suitable dispersant or deflocculant (ILO, 1987; Houben and Guillaud, 1994). Sodium hexametaphosphate (tannic acid) is commonly used. The use of ordinary salt is not recommended as it is a known flocculant causing the agglomeration of clay particles in water (Grimshaw, 1971).

FIELD AND LABORATORY TESTING

LINEAR SHRINKAGE TEST

TEST TITLE: **Linear Shrinkage Test (LST)**

Standard: Webb and Lockwood, 1987; ILO, 1987; Webb, 1988; Stulz and Mukerji, 1988.

Objective: To estimate the proportion of the clay fraction in a soil from its linear shrinkage value and by implication, the stabiliser type and amount.

Precision: Medium to high accuracy.

Duration: 7 to 10 days.

Limitations: Requires at least one week before results can be obtained.

APPARATUS

1. Sieve of aperture opening 6 mm (or 5 mm)..
2. Alcocks wooden mould: internal dimensions 600 x 40 x 40 mm open at the tope with formica lined walling.
3. Wooden spatula (small).
4. Accurate measuring scale (vernier calliper or rule to 0.5 mm).
5. Lubricant: mould release oil, vaseline or silicone grease.
6. Clean drinking water.

TEST PROCEDURE

(i) Take about 1.5 kg to 2.0 kg of the representative soil sample that has passed through the 6 mm sieve and moisten it. Make the soil wet enough to form a paste which when tapped brings water to the surface, thus indicating proximity to the OMC. Confirm the proximity to OMC by squeezing the damp soil lump in the hand and checking if it can retain its shape without soiling the hands. Also drop the lump from about one metre height and check if it does not break into several smaller lumps.

(ii) Measure and record the internal dimensions of the mould and lightly smear the inside with a suitable lubricant. This is done to prevent the soil adhering to the surface of the internal walls of the mould which would interfere with the movement while shrinking.

(iii) Fill the soil into the mould in three equal layers while tapping and lightly pressing it in all four corners using a wooden spatula. This is done to eliminate any trapped air pockets from the soil. Smoothen the top of the final layer using the spatula so that the soil exactly fills the mould box. This ensures that any soil that would have extended over is removed. It would have otherwise increased the drag as the sample dries out and begins to shrink.

(iv) Leave the filled box with its contents in the sun for a period of 5 to 7 days, or in a shaded area for 7 to 10 days. During this period, the mould and its contents should not be rewet, e.g. by rain or addition of more water.

(v) After the above period in (iv), the soil should have dried out and shrunk either as: a single piece, several pieces with cracks across the width; or hogged up and out of the mould in a crescent shape. If the soil dried out in several pieces, gently elevate the box to about 45° on one end and tap it to move all the cracked pieces to one end of the mould. If hogging is the result, then take the dry length as the average length of the upper and lower faces lengthwise.

(vi) Calculate the linear shrinkage by determining the shrinkage gap by deducting the length of the dry soil sample from that of the mould cavity box. The shrinkage is expressed as a percentage of the original mould cavity length, or simply in millimetres.

$$LS = \frac{L_W - L_d}{L_W} \times 100 \ (\%)$$

Where LS = linear shrinkage (%)

L_W = length of the wet bar (mm)

L_d = length of the dry bar (mm)

Results are discussed in Chapters 4 and 5.

RESULTS AND RECOMMENDATIONS

Shrinkage and severe cracking across the width of a soil is an indication of high sand content soils of low clay and silt contents. Shrinkage with hogging up and out is an indication of a high clay content soil.

Soil for CSB production should shrink or swell as little as possible. The more the clay content of the soil, the more it will tend to shrink. Such soils can be modified by controlled mixing with sand, in which case the test has to be repeated using the blended soil. The amount of linear shrinkage in soils have been used to suggest the type and amount of stabiliser to be used (Webb and Lockwood, 1987). Low shrinkage soils (high sand content) are better stabilised with OPC, while high shrinkage soils (high clay content) are better stabilised using lime.

PARTICLE SIZE DISTRIBUTION CHART FOR SOIL 'S'

APPENDIX M

LABORATORY RECORDING SHEET

MIX COMPOSITION USED FOR MCSB, CSSB AND CLSB

A. Microsilica-Cement Soil Blocks (MSCB)

SN	STABILISER PERCENTAGE USED		ACTUAL MASS IN GRAMS				
	cc	Microsilica	Fine gravel + sand + silt	Clay	Cement	Microsilica	Total
	%	%	g	g	g	g	g
1	3	0.3	6986.6	1232.9	255.0	25.5	8500.0
2	5	0.5	6827.6	1204.9	425.0	42.5	8500.0
3	7	0.7	6668.7	1176.8	595.0	59.5	8500.0
4	9	0.9	6509.7	1148.8	765.0	76.5	8500.0
5	11	1.1	6350.8	1120.7	935.0	93.5	8500.0

B. Cement-Stabilised Soil Blocks (CSSB)

SN	STABILISER PERCENTAGE USED		ACTUAL MASS IN GRAMS				
	cc	Other	Fine gravel + sand + silt	Clay	Cement	Other	Total
	%	%	g	g	g	g	g
1	3	-	7008.2	1236.8	255.0	-	8500.0
2	5	-	6863.7	1211.3	425.0	-	8500.0
3	7	-	6719.2	1185.8	595.0	-	8500.0
4	9	-	6574.7	1160.3	765.0	-	8500.0
5	11	-	6430.2	1134.8	935.0	-	8500.0

C. Cement-Lime Soil Blocks (CLSB)

SN	STABILISER PERCENTAGE USED		ACTUAL MASS IN GRAMS				
	cc	Lc	Fine gravel sand + silt	Clay	Cement	Lime	Total
	%	%	g	g	g	g	g
1	3	5`	6647.0	1173.0	255.0	425.0	8500.0
2	5	5	6502.5	1147.5	425.0	425.0	8500.0
3	7	5	6358.0	1122.0	595.0	425.0	8500.0
4	9	5	6213.5	1096.5	765.0	425.0	8500.0
5	11	5	6069.0	1071.0	935.0	425.0	8500.0

SUMMARY LIST OF CSBs PRODUCED

S/N	SPECIMEN REFERENCE	CEMENT CONTENT	NUMBER OF BLOCKS	NUMBER OF THE DIFFERENT SPECIMEN SIZES OBTAINED			
			290 X 140 X 100 mm	100 x 100 x 100 mm	100 x 100 x 90 mm	100 x 100 x 40 mm	100 x 90 x 40 mm
		%	No.	No.	No.	No.	No.
A	COMPACTED AT 6 MPa						
1	MCSB 116	11	3	6	3	6	3
2	MCSB 096	9	3	6	3	6	3
3	MCSB 076	7	3	6	3	6	3
4	MCSB 056	5	3	6	3	6	3
5	MCSB 036	3	3	6	3	6	3
6	CSSB 116	11	3	6	3	6	3
7	CSSB 096	9	3	6	3	6	3
8	CSSB 076	7	3	6	3	6	3
9	CSSB 056	5	3	6	3	6	3
10	CSSB 036	3	3	6	3	6	3
11	CLSB 556	5	3	6	3	6	3
12	CLSB 356	3	3	6	3	6	3
	SUBTOTAL A	**-**	**36**	**72**	**36**	**72**	**36**
B	COMPACTED AT 10 MPa						
13	CSSB 1110	11	3	6	3	6	3
14	CSSB 0910	9	3	6	3	6	3
15	CSSB 0710	7	3	6	3	6	3
16	CSSB 0510	5	3	6	3	6	3
17	CSSB 0310	3	3	6	3	6	3
	SUBTOTAL B	-	15	30	15	30	15
	GRAND TOTAL (A + B)	**-**	**51**	**102**	**51**	**102**	**51**
C	COMPARABLE MATERIALS						
18	CBS	12-18	-	6	-	-	-
19	FBS	-	-	6	-	-	-
20	RBS	-	-	6	-	-	-
	TOTAL C			**18**			

Reference key:

MCSB 116	=	Microsilica cement soil block compacted at 6 MPa (11% cement)
CSSB 116	=	Cement stabilised soil block compacted at 6 MPa (11% cement)
CLSB 556	=	Cement lime soil block compacted at 6 MPa (5% cement, 5% lime)
CBS	=	Concrete block sample
FBS	=	Fired brick sample
RBS	=	Rock block sample

Note: Includes list of comparable materials obtained from the laboratory.

WET COMPRESSIVE STRENGTH TESTING

TEST TITLE: **Wet Compressive Strength Test (WCS)**

Standard:	BS 3921: 1985; BS 6071: Parts 1 & 2: 1981; Neville 1995
Objective:	To determine the wet compressive strength of various categories of blocks
Precision:	High accuracy (BS 1610: 1964 Grade A or B)
Delimitations:	Results can be affected by the sample size, moisture condition, curing age, the rigidity of the testing machine, type of end preparation used, and the rate of application of the load.
Duration:	2 to 5 minutes per test
Specimen description:	Various CSB categories: MCSB, CSSB, CLSB cut to cube size 100 x 100 x 100 mm, 28 days old, pre-immersed in water for 24 hours prior to testing.

APPARATUS

1. Compression Testing Machine: Denison 7231, machine number T91080/ES 8171, calibration certificate number 04818 (re-calibrated December 1998, 1999, 2000). The machine has the means of providing the rate of loading, capacity 100-300 KN. Accuracy complies with BS 1610 grade A and B. The upper platen of the machine is able to align freely with the specimen as contact is made. The lower platen bearing the sample is plain and non-tilting.
2. Plywood packing 105 x 105 x 20 mm; free from knots and new for each sample tested.
3. Masonry saw machine (concrete lathe cutting machine); trademark Clipper, model (t W 2-40-3), MS 27, serial number 606726, 4Kw 50Hz T/M 2900 (Luxembourg). Used to reduce blocks of 290 x 140 x 100 mm to 100 x 100 x 100 mm prisms.
4. Water tank 2000 x 1000 x 600 mm with provision for free circulation of water at bottom of samples (to immerse and soak blocks overnight)
5. Laboratory balance: accuracy up to 0.1% of the mass of the specimen.

TEST PROCEDURE

(i) Take three samples each cut from the various categories of block types; measure and record their area and volume individually.

(ii) Immerse the samples in a water filled tank (temperature 10-25°C) provided with a free circulation frame at the bottom for 24 hours.

(iii) Remove and leave to drain on a stillage or damp sacking until the blocks stop dripping (about 30 minutes).

(iv) Wipe clean the bearing surface of the platens to remove any loose grit. Place the specimen between two new 4 to 20 mm plywood sheets with an over-hang allowance of 5 mm along each edge. Make sure the centre of the mass of the specimen coincides with the axis of the machine.

(v) Make a final check of the correct positioning and packing, then apply the load without shock at a rate of 15 KN/min. Maintain the load up to failure (1 to 5 minutes).

(vi) Record the maximum load at failure and as well as the rate of loading (these were recorded automatically by the machine and a printout obtained).

(vii) Note the type of failure mode and calculate the crushing strength as below:

$$\mathrm{WCS} = \frac{M_L \ (\mathrm{KN})}{A_S \ (\mathrm{mm}^2)}$$

Where: WCS = wet compressive strength (MPa)

M_L = maximum load (KN)

A_S = cross section area (mm^2)

(viii) Calculate the average of 3 tests done on each category of material from the same mix batch and processing method.

Repeat the same procedure to determine the dry compressive strength (DCS) value, except that the samples do not need to be soaked in water for 24 hours as before. Instead they are oven-dried till constant mass and tested as described above. Results are discussed in Chapter 6.

LABORATORY RECORDING SHEET: WET COMPRESSIVE STRENGTH

SN	SPECIMEN REFERENCE A_{CS} = 10,000 mm^2 L_R = 15 KN/min	CC	MAXIMUM LOAD	WCS (28 DAY)	MEAN WCS (28 DAY)
		%	KN	MPa	MPa
1	CSSB 361	3	14.7	1.47	
2	CSSB 362	3	14.4	1.44	1.43
3	CSSB 363	3	14.1	1.41	
4	CSSB 561	5	22.5	2.25	
5	CSSB 562	5	27.7	2.77	2.48
6	CSSB 563	5	24.2	2.42	
7	CSSB 761	7	45.4	4.54	
8	CSSB 762	7	43.3	4.33	4.57
9	CSSB 763	7	48.4	4.84	
10	CSSB 961	9	64.2	6.42	
11	CSSB 962	9	62.9	6.29	6.54
12	CSSB 963	9	69.1	6.91	
13	CSSB 1161	11	90.6	9.06	
14	CSSB 1162	11	88.8	8.88	8.99
15	CSSB 1163	11	90.3	9.03	

Key: CSSB 363 = Cement stabilised soil block / 3% cc / 6 MPa / sample no. 3
A_{CS} = Cross-section area
L_R = Loading rate
CC = Cement-content

LABORATORY RECORDING SHEET: WET COMPRESSIVE STRENGTH

SN	SPECIMEN REFERENCE A_{CS} = 10,000 mm^2 L_R = 15 KN/min	CC	MAXIMUM LOAD	WCS (28 DAY)	MEAN WCS (28 DAY)
		%	KN	MPa	MPa
1	MCSB 361	3	31.9	3.19	
2	MCSB 362	3	30.7	3.07	3.12
3	MCSB 363	3	31.0	3.10	
4	MCSB 561	5	53.3	5.33	
5	MCSB 562	5	61.5	6.15	5.76
6	MCSB 563	5	58.0	5.80	
7	MCSB 761	7	96.8	9.68	
8	MCSB 762	7	106.7	10.67	10.11
9	MCSB 763	7	99.8	9.98	
10	MCSB 961	9	143.6	14.36	
11	MCSB 962	9	140.1	14.01	14.19
12	MCSB 963	9	142.0	14.20	
13	MCSB 1161	11	181.0	18.10	
14	MCSB 1162	11	181.3	18.13	18.30
15	MCSB 1163	11	186.7	18.67	

Key: MCSB 761 = Microsilica-cement soil block / 11% cc / 6 MPa / sample no. 1
A_{CS} = Cross-section area
L_R = Loading rate
CC = Cement-content

LABORATORY RECORDING SHEET: WET COMPRESSIVE STRENGTH

SN	SPECIMEN REFERENCE A_{CS} = 10,000 mm^2 L_R = 15 KN/min	CC	MAXIMUM LOAD	WCS (28 DAY)	MEAN WCS (28 DAY)
		%	KN	MPa	MPa
1	CSSB 3101	3	19.8	1.98	
2	CSSB 3102	3	18.7	1.87	1.89
3	CSSB 3103	3	18.2	1.82	
4	CSSB 5101	5	31.5	3.15	
5	CSSB 5102	5	32.9	3.29	3.21
6	CSSB 5103	5	31.9	3.19	
7	CSSB 7101	7	52.4	5.24	
8	CSSB 7102	7	53.3	5.33	5.29
9	CSSB 7103	7	53.0	5.30	
10	CSSB 9101	9	74.8	7.48	
11	CSSB 9102	9	75.0	7.50	7.51
12	CSSB 9103	9	75.5	7.55	
13	CSSB 11101	11	98.1	9.81	
14	CSSB 11102	11	98.4	9.84	9.84
15	CSSB 11103	11	98.7	9.87	

Key: CSSB 9102 = Cement stabilised soil block / 9% cc / 10 MPa / sample no. 2
A_{CS} = Cross-section area
L_R = Loading rate
CC = Cement-content

LABORATORY RECORDING SHEET: DRY COMPRESSIVE STRENGTH

SN	SPECIMEN REFERENCE A_{CS} = 10,000 mm^2 L_R = 15 KN/min	CC	MAXIMUM LOAD	DCS (28 DAY)	MEAN DCS (28 DAY)
		%	KN	MPa	MPa
1	CSSB 361	3	27.3	2.73	
2	CSSB 362	3	26.9	2.69	2.70
3	CSSB 363	3	26.8	2.68	
4	CSSB 561	5	46.6	4.66	
5	CSSB 562	5	45.7	4.57	4.61
6	CSSB 563	5	46.0	4.60	
7	CSSB 761	7	73.1	7.31	
8	CSSB 762	7	73.0	7.30	7.33
9	CSSB 763	7	73.8	7.38	
10	CSSB 961	9	96.9	9.69	
11	CSSB 962	9	96.4	9.64	9.66
12	CSSB 963	9	96.5	9.65	
13	CSSB 1161	11	122.5	12.25	
14	CSSB 1162	11	123.6	12.36	12.3
15	CSSB 1163	11	122.9	12.29	

Key: CSSB 562 = Cement stabilised soil block / 5% cc / 6 MPa / sample no. 2
DCS = Dry compressive strength
A_{CS} = Cross-section area
L_R = Loading rate
CC = Cement-content

LABORATORY RECORDING SHEET: DRY COMPRESSIVE STRENGTH

SN	SPECIMEN REFERENCE A_{CS} = 10,000 mm^2 L_R = 15 KN/min	CC	MAXIMUM LOAD	DCS (28 DAY)	MEAN DCS (28 DAY)
		%	KN	MPa	MPa
1	MCSB 361	3	39.8	3.98	
2	MCSB 362	3	39.1	3.91	3.94
3	MCSB 363	3	39.3	3.93	
4	MCSB 561	5	70.0	7.00	
5	MCSB 562	5	71.0	7.10	7.09
6	MCSB 563	5	71.7	7.17	
7	MCSB 761	7	121.5	12.15	
8	MCSB 762	7	119.7	11.97	12.02
9	MCSB 763	7	119.4	11.94	
10	MCSB 961	9	162.2	16.22	
11	MCSB 962	9	161.2	16.12	16.18
12	MCSB 963	9	162.0	16.20	
13	MCSB 1161	11	206.0	20.6	
14	MCSB 1162	11	207.0	20.7	20.5
15	MCSB 1163	11	202.0	20.2	

Key: MCSB 761 = Microsilica-cement soil block / 7% cc / 6 MPa / sample no. 1
A_{CS} = Cross-section area
L_R = Loading rate
CC = Cement-content

LABORATORY RECORDING SHEET: BLOCK DRY DENSITY (BDD)

(CSSB Specimens)

SN	Sample Ref.	cc	Dimensions				Oven dry mass			Block dry density	
			L	W	H	Gross volume	1	2	3	Gross	Mean
		%	mm	mm	mm	m^3 (x 10^{-3})	g	g	g	Kg/m^3	Kg/m^3
1	CSSB 361	3	101.2	99.5	101.1	1.01802	2120.1	2118.6	2118.5	2081	
2	CSSB 362	3	101.3	99.6	101.2	1.02106	2130.3	2130.0	2129.9	2085	2084
3	CSSB 363	3	101.4	99.8	101.1	1.02310	2134.0	2133.4	2133.2	2085	
4	CSSB 561	5	101.1	99.6	101.4	1.02105	2153.9	2153.4	2153.4	2109	
5	CSSB 562	5	101.2	99.6	101.4	1.02206	2146.8	2146.3	2146.3	2100	2102
6	CSSB 563	5	101.4	99.7	101.3	1.02410	2147.9	2147.6	2147.5	2097	
7	CSSB 761	7	101.1	99.7	101.1	1.01905	2153.1	2152.3	2152.2	2112	
8	CSSB 762	7	101.0	99.6	101.0	1.01602	2149.6	5149.2	2148.9	2115	2114
9	CSSB 763	7	101.0	99.8	101.0	1.01806	2153.4	2153.2	2153.2	2115	
10	CSSB 961	9	101.3	99.5	101.1	1.01902	2172.8	2172.6	2172.6	2132	
11	CSSB 962	9	101.2	99.7	101.1	1.02006	2167.9	2167.6	2167.6	2125	2127
12	CSSB 963	9	101.2	99.5	101.3	1.02003	2167.8	2166.8	2166.5	2124	
13	CSSB 1161	11	101.1	99.9	101.2	1.02211	2179.2	2178.2	2178.1	2131	
14	CSSB 1162	11	101.4	99.8	101.1	1.02310	2185.8	2185.3	2185.3	2135	2132
15	CSSB 1163	11	101.3	99.9	101.1	1.02312	2178.6	2178.2	2178.2	2129	

Key: CSSB 361 = Cement stabilised soil block (3% cc / 6 MPa / sample no. 1)
L = Length
W = Width
H = Height

LABORATORY RECORDING SHEET: BLOCK DRY DENSITY (BDD)

(CSSB Specimens)

SN	Sample Ref.	cc	Dimensions				Oven dry mass			Block dry density	
			L	W	H	Gross volume	1	2	3	Gross	Mean
		%	mm	mm	mm	m^3 (x 10^{-3})	g	g	g	Kg/m^3	Kg/m^3
1	CSSB 3101	3	101.0	99.8	101.1	1.0191	21525.5	2152.3	2152.3	2112	
2	CSSB 3102	3	101.1	99.9	101.2	1.0221	2164.9	2164.9	2164.8	2118	2113
3	CSSB 3103	3	101.0	99.8	101.2	1.0201	2151.3	2151.3	2151.3	2109	
4	CSSB 5101	5	101.2	99.5	101.3	1.0200	2171.7	2171.6	2171.6	2129	
5	CSSB 5102	5	101.1	99.7	101.3	1.0211	2168.9	2168.9	5168.8	2124	2128
6	CSSB 5103	5	101.2	99.7	101.4	1.0231	2180.3	2180.3	2180.2	2131	
7	CSSB 7101	7	100.9	99.6	101.3	1.0180	2170.6	2170.5	2170.4	2132	
8	CSSB 7102	7	101.0	99.5	101.3	1.0170	2174.5	2174.4	2174.4	2138	2136
9	CSSB 7103	7	101.1	99.6	101.2	1.0190	2178.9	2178.8	2178.8	2138	
10	CSSB 9101	9	101.3	99.7	101.1	1.0211	2193.4	2193.3	2193.3	2148	
11	CSSB 9102	9	101.3	100.1	101.0	1.0242	2201.9	2201.9	2201.9	2150	2149
12	CSSB 9103	9	101.1	99.9	101.2	1.0221	2196.8	2196.6	2196.5	2149	
13	CSSB 11101	11	101.2	99.8	101.3	1.0231	2205.9	2205.8	2205.8	2156	
14	CSSB 11102	11	101.2	99.6	101.4	1.0221	2202.8	2202.5	2202.5	2155	2157
15	CSSB 11103	11	101.3	99.9	101.2	1.0241	2212.4	2212.2	2212.1	2160	

Key: CSSB 3101 = Cement stabilised soil block (3% cc / 10 MPa / sample no. 1)
L = Length
W = Width
H = Height

LABORATORY RECORDING SHEET: BLOCK DRY DENSITY (BDD)

(MCSB SPECIMENS)

SN	Sample Ref.	cc	Dimensions				Oven dry mass			Block dry density	
			L	W	H	Gross volume	1	2	3	Gross	Mean
		%	mm	mm	mm	m^3 ($x\ 10^{-3}$)	g	g	g	Kg/m^3	Kg/m^3
1	MCSB 361	3	101.2	99.6	101.1	1.01904	2197.5	2197.0	2197.0	2156	
2	MCSB 362	3	101.1	99.6	101.2	1.02211	2199.8	2199.6	2199.6	2152	2153
3	MCSB 363	3	101.0	99.7	101.3	1.02006	2194.3	2194.2	2194.2	2151	
4	MCSB 561	5	101.1	99.8	101.2	1.02109	2221.3	2221.0	2220.9	2175	
5	MCSB 562	5	101.3	99.7	101.3	1.02309	2230.5	2230.3	2230.0	2180	2176
6	MCSB 563	5	101.2	99.6	101.2	1.02005	2216.7	2216.7	2216.6	2173	
7	MCSB 761	7	101.2	99.6	101.4	1.02514	2247.3	2247.2	2247.1	21912	
8	MCSB 762	7	101.1	99.8	101.3	1.02209	2247.9	2247.8	2247.6	2199	2194
9	MCSB 763	7	101.4	99.8	101.4	1.02614	2248.5	2248.4	2248.3	2191	
10	MCSB 961	9	101.3	99.5	101.1	1.01902	2255.8	2251.1	2255.1	2213	
11	MCSB 962	9	101.4	99.7	101.0	1.02107	2267.2	2266.9	2266.8	2220	2218
12	MCSB 963	9	101.1	99.6	101.3	1.02005	2265.7	2265.6	2265.5	2221	
13	MCSB 1161	11	101.2	99.8	101.0	1.02008	2284.4	2283.9	2283.9	2239	
14	MCSB 1162	11	101.2	99.8	101.0	1.02008	2297.7	2292.2	2292.1	2247	2242
15	MCSB 1163	11	101.3	99.6	101.1	1.02005	2285.3	2284.9	2284.9	2240	

Key: MCSB 361 = Microsilica-cement soil block (3% cc / 6 MPa / sample no. 1)
L = Length
W = Width
H = Height

LABORATORY RECORDING SHEET: BLOCK DRY DENSITY (BDD)

(CLSB SPECIMENS)

SN	Sample Ref.	cc	Dimensions				Oven dry mass			Block dry density	
			L	W	H	Gross volume	1	2	3	Gross	Mean
		%	mm	mm	mm	m^3 (x 10^{-3})	g	g	g	Kg/m^3	Kg/m^3
1	CLSB 3561	3	101.2	99.5	101.1	1.01802	2085.3	2084.9	2084.9	2048	
2	CLSB 3562	3	101.0	99.6	101.0	1.01602	2089.1	2088.9	2088.9	2056	2051
3	CLSB 3563	3	101.3	99.6	101.2	1.02106	2092.4	2092.2	2092.1	2049	
4	CLSB 5561	5	101.1	99.8	101.3	1.02209	2109.8	2109.6	2109.6	2064	
5	CLSB 5562	5	101.1	99.8	101.1	1.02008	2109.9	2109.6	2109.5	2068	2066
6	CLSB 5563	5	101.3	99.8	101.2	1.02311	2113.8	2113.8	2113.7	2066	
7	CLSB 761	7	101.0	99.9	101.1	1.02009	2113.9	2113.7	2113.6	2072	
8	CLSB 762	7	101.4	99.9	101.1	1.02413	2127.3	2127.2	2127.1	2077	2074
9	CLSB 763	7	101.1	99.7	101.2	1.02006	2114.8	2114.6	2114.6	2073	
10	CLSB 961	9	101.2	99.5	101.3	1.02003	2121.9	2121.8	2121.7	2080	
11	CLSB 962	9	101.2	99.8	101.3	1.02311	2126.2	2126.1	2126.0	2078	2081
12	CLSB 963	9	101.3	99.6	101.2	1.02106	2129.4	2128.9	2128.9	2085	
13	CLSB 1161	11	101.4	99.9	101.1	1.02413	2147.9	2147.6	2147.6	2097	
14	CLSB 1162	11	101.2	99.8	101.0	1.02008	2132.5	2131.9	2131.9	2090	2095
15	CLSB 1163	11	101.3	99.9	101.0	1.02211	2144.7	2144.5	2144.4	2098	

Key: CLSB 3561 = Cement-lime stabilised soil block (3% cc / 5% lc / 6 MPa / sample no. 1)
L = Length
W = Width
H = Height

TOTAL WATER ABSORPTION AND VOLUME FRACTION POROSITY

TEST TITLE: **Total Water Absorption Test (TWA)**
Total Volume Porosity (TVP)

Standard:	BS 3921: 1985; BS 1881: Part 122: 1983; ASTM C 642 90
Objective:	To determine the water absorption values of blocks and to calculate the total volume porosity.
Precision:	Medium to high accuracy
Delimitations:	By using the cold immersion method, some air may still remain entrapped in the pores.
Duration:	24 hours
Specimen description:	Various CSB categories (as before); fired brick samples and concrete block samples.

APPARATUS

1. Ventilated drying oven (BS 2648).
2. Tank with bottom grid to ensure free circulation of water.
3. Electronic weighing scale (accurate to 0.1% of the specimen mass).

TEST PROCEDURE

(i) Dry the specimens from each category of blocks to constant mass in the oven at temperatures between 110°C and 115°C.

(ii) When cool, weigh each specimen to an accuracy of 0.1% of the specimen mass.

(iii) Immerse the specimens in a single layer tank immediately after weighing so that water can circulate freely on all sides and bottom of the sample. Leave a space of about 10 mm between adjacent samples in the tank.

(iv) After 24 hours, remove the specimens, wipe off the surface water while shaking lightly with a damp cloth and reweigh each specimen within 2 minutes of removal from the water tank.

(v) Calculate the water absorbed by each sample (TWA) expressed as a percentage of the dry mass using the equation:

$$TWA = \frac{(M_W - M_D)}{M_D} \times 100$$

Where: TWA = total water absorption (%)
M_W = wet mass (g)
M_D = dry mass (g)

Obtain the mean of three samples of the same mix and processing category (Chapter 5).

(vi) Calculate the total volume porosity using the formula.

$$n = \frac{(TWA)\,\rho}{100\,\rho_W}$$

Where: n = porosity (fraction)
ρ = block dry density (kg/m^3)
ρ_W = density of water (kg/m^3)
TWA = total water absorption (%)

The results obtained are discussed in Chapter 6.

LABORATORY RECORDING SHEET: TOTAL WATER ABSORPTION AND TOTAL VOLUME POROSITY

			SAMPLE TYPE														
			Cement stabilised soil block (CSSB) : 6 MPa block samples														
			3% cc			5% cc			7% cc			9% cc			11% cc		
SN	ITEM	UNITS	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
1	Pre-test dry mass (1)	g	724.7	704.1	711.3	722.9	722.5	731.7	740.8	741.3	742.5	770.4	761.7	763.9	781.8	782.6	796.9
2	Pre-test dry mass (2)	g	724.5	703.9	710.6	722.8	722.2	721.6	739.9	741.0	742.3	770.1	761.5	763.5	781.7	782.4	796.9
3	Pre-test dry mass (3)	g	724.5	703.8	710.6	722.8	722.2	731.6	739.8	740.9	742.3	770.1	761.5	763.4	781.7	782.4	796.8
4	Post-test wet mass	g	818.7	784.7	794.5	792.2	791.5	806.2	801.2	800.2	795.0	827.1	812.5	816.1	835.6	838.0	847.8
5	Total water absorption	%	13.0	11.5	11.8	9.6	9.6	10.2	8.3	8.0	7.1	7.4	6.7	6.9	6.9	7.1	6.4
6	Mean TWA	%	12.1			9.8			7.8			7.0			6.8		
7	Volume fraction porosity	%	25.3			20.6			16.5			14.9			14.4		

APPENDIX T (2)

LABORATORY RECORDING SHEET: TOTAL WATER ABSORPTION AND TOTAL VOLUME POROSITY

			SAMPLE TYPE														
			Cement stabilised soil blocks (CSSB) : 10 MPa block samples														
			3% cc			5% cc			7% cc			9% cc			11% cc		
SN	ITEM	UNITS	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
1	Pre-test dry mass (1)	g	721.3	722.8	726.9	750.6	744.8	754.4	775.7	763.9	760.9	781.3	776.7	782.7	789.3	793.6	796.0
2	Pre-test dry mass (2)	g	720.9	721.7	726.5	749.9	744.7	753.4	775.5	763.9	760.8	781.3	776.6	782.4	788.9	793.4	795.2
3	Pre-test dry mass (3)	g	720.8	721.7	726.4	749.8	744.6	753.3	775.5	763.9	760.8	781.1	776.5	782.4	788.9	793.4	795.1
4	Post-test wet mass	g	795.8	793.1	799.0	807.5	801.9	806.8	823.6	809.0	812.5	820.9	821.5	821.5	827.6	837.0	836.4
5	Total water absorption	%	10.4	9.9	10.0	7.7	7.7	7.1	6.2	5.9	6.8	5.1	5.8	5.0	4.9	5.5	5.2
6	Mean TWA	%	10.1			7.5			6.3			5.3			5.2		
7	Volume fraction porosity	%	21.4			16.0			13.5			11.5			11.1		

LABORATORY RECORDING SHEET: TOTAL WATER ABSORPTION AND TOTAL VOLUME POROSITY

			SAMPLE TYPE														
			Microsilica cement soil block (MCSB) : 6 MPa block samples														
			3% cc			5% cc			7% cc			9% cc			11% cc		
SN	ITEM	UNITS	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
1	Pre-test dry mass (1)	g	741.6	743.3	757.1	760.4	765.3	763.5	770.6	769.9	772.5	788.4	786.5	786.5	791.8	795.7	799.9
2	Pre-test dry mass (2)	g	740.8	742.9	755.9	760.1	764.8	763.5	770.4	769.7	771.8	787.9	786.5	786.1	791.5	795.6	799.5
3	Pre-test dry mass (3)	g	740.7	742.8	755.9	760.1	764.8	763.4	770.3	769.5	771.8	787.9	786.3	786.0	791.5	795.6	799.4
4	Post-test wet mass	g	794.8	782.2	801.3	789.0	803.0	796.2	801.9	797.2	801.1	817.1	818.5	814.3	821.6	824.2	830.6
5	Total water absorption	%	7.3	5.3	6.0	3.8	5.0	4.3	4.1	3.6	3.8	3.7	4.1	3.6	3.8	3.6	3.9
6	Mean TWA	%	6.2			4.4			3.9			3.8			3.8		
7	Volume fraction porosity	%	13.3			9.5			8.5			8.4			8.4		

APPENDIX T (4)

LABORATORY RECORDING SHEET: TOTAL WATER ABSORPTION AND TOTAL VOLUME POROSITY

			SAMPLE TYPE														
			Cement-lime soil blocks (CLSB) : 6 MPa block samples (5% cc)														
			3% cc			5% cc			7% cc			9% cc			11% cc		
SN	ITEM	UNITS	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
1	Pre-test dry mass (1)	g	674.9	699.6	693.8	707.2	713.7	725.9	731.3	727.6	728.7	738.6	740.9	741.8	764.6	756.8	772.5
2	Pre-test dry mass (2)	g	674.7	699.1	693.5	706.8	713.5	725.2	731.1	726.9	728.5	738.5	740.8	741.7	764.1	756.7	772.3
3	Pre-test dry mass (3)	g	674.6	699.1	693.5	706.8	713.4	725.2	731.1	726.9	728.4	738.5	740.8	741.6	763.9	756.7	772.3
4	Post-test wet mass	g	767.0	792.8	794.8	801.5	799.0	812.9	817.4	789.7	802.0	810.1	810.4	809.1	828.8	826.3	844.1
5	Total water absorption	%	13.7	13.4	14.6	13.4	12.0	12.1	11.8	9.9	10.1	9.7	9.4	9.1	8.5	9.2	9.3
6	Mean TWA	%	13.9			12.5			10.6			9.4			9.0		
7	Volume fraction porosity	%	28.4			25.8			22.0			19.6			18.9		

APPENDIX T (5)

LABORATORY RECORDING SHEET: TOTAL WATER ABSORPTION AND TOTAL VOLUME POROSITY

			SAMPLE TYPE														
			Fired bricks (FB); Concrete blocks (CBS); and Rock block samples														
			FB			CBS			RBS (sandstone)								
SN	ITEM	UNITS	1	2	3	1	2	3	1	2	3						
1	Pre-test dry mass (1)	g	163.9	188.6	207.1	229.7	216.9	212.3	236.8	224.5	241.9						
2	Pre-test dry mass (2)	g	163.6	188.3	207.1	229.4	216.9	211.9	236.5	223.9	241.7						
3	Pre-test dry mass (3)	g	163.6	188.3	207.1	229.4	216.8	211.8	236.5	223.9	241.7						
4	Post-test wet mass	g	179.3	204.7	225.7	237.0	227.0	219.6	247.1	235.5	252.3						
5	Total water absorption	%	9.6	8.7	9.0	3.3	4.7	3.7	4.5	5.2	4.4						
6	Mean TWA	%	9.1			3.9			4.7								
7	Volume fraction porosity	%	-			-			-								

THIN SECTION MICROGRAPH OF CSB SURFACES

EVALUATION OF SURFACE PERFORMANCE

SLAKE DURABILITY TEST

TEST TITLE:	**Slake Durability Test (SDT)**
Standard:	ISO (1967); ISRM (1971); Gamble (1971); Franklin and Chandra (1972).
Objective:	To monitor the performance of surfaces of various block samples when subjected to wetting, abrasion and drying.
Precision:	Very high accuracy
Delimitations:	Results can be affected by sample shape, size, weight and number; sieve mesh size, drum size and speed of rotation; state of sample moisture condition; duration of slaking; chemistry of the slaking liquid.
Duration:	10 minutes
Sample description:	Soil type (soil 'S'); sample types (IPD and TDB of varied cc 3% to 11% compressed at 6 MPa and 10 MPa; curing age (7 days, 14 days, 28 days, 56 days). FBS, CBS and RBS also tested

APPARATUS

1. Slake durability test equipment: sieve mesh opening 2mm, drum size (140 mm diameter), 100 mm (long); speed of rotation (20 revolutions per minute); electrically operated.
2. Electronic weighing scale.
3. Standard laboratory oven (105°C)
4. Timer (clock).
5. China clay dish containers (90g to 300g).
6. Laboratory tap water (Coventry).
7. Hand-held magnifying glass.

TEST PROCEDURE

(i) To represent each specimen sample, select 4 or 5 pre-cut samples each weighing between 115g and 125g with a total mass of between 450g and 550g. Oven dry the samples overnight to constant mass.

(ii) Weigh and mark the dish containers separately and then together with their contents from (i) above.

(iii) Place the pre-weighed and oven-dried samples into the drums. Couple the drums to the mortar drive, making sure they are connected in the correct order.

(iv) Fill the tanks with laboratory tap water (about 20°C) to the level indicated on the side of the tanks and immediately set the test in motion using the switch. Run and time the test for 10 minutes.

(v) At the end of 10 minutes, switch off the drive, remove the drums and record the state of the water in each bath and the type of sediments deposited at the bottom of each one. Examine the worn samples using a hand-held magnifying glass.

(vi) Place the removed specimens into their respective china containers and dry them to constant mass using the oven set at 105°C. When successive weighings yield the same result, record the dry mass.

(vii) The slake durability index (SDI or I_d) is then given in percent terms by the ratio of the final to original mass:

$$SDI = \frac{M_f}{M_O} \times 100$$

Where: SDI = slake durability index (%)

M_f = final mass (g)

M_O = original mass (g)

The container mass should be deducted before determining the SDI in all cases.

(viii) Repeat steps (i) to (vii) for all other samples to be tested.

CLASSIFICATION OF RESULTS

Existing and proposed classifications and grading are described in Chapter 7, the results obtained are also discussed in Chapter 7.

APPENDIX W (1)

LABORATORY RECORDING SHEET: SLAKE DURABILITY TEST

			SAMPLE TYPE														
			Cement-Stabilised Soil Blocks (CSSB) – 28 days (6 MPa)														
			3% cc			5% cc			7% cc			9% cc			11% cc		
SN	**Item**	**Units**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**
1	Container Reference	-	CSS31	CSS32	CSS33	CSS51	CSS52	CSS53	CSS71	CSS72	CSS73	CSS91	CSS92	CS593	CS111	CS112	CSS113
2	Container mass	g	274.5	257.3	296.7	136.2	136.0	155.5	156.7	155.8	157.9	171.4	157.5	157.9	257.3	274.5	182.4
3	Container + pre-test dry mass (1)	g	717.8	712.9	789.7	624.3	611.5	633.0	644.1	657.3	668.6	679.9	681.7	696.9	798.8	821.3	721.7
4	Container + pre-test dry mass (2)	g	716.2	711.2	789.4	622.9	611.3	631.7	643.5	655.4	666.3	679.4	681.1	696.8	798.4	820.6	721.3
5	Container + pre-test dry mass (3)	g	716.2	711.2	789.1	622.8	611.0	631.3	643.4	655.2	666.1	678.9	681.0	696.8	797.7	820.4	721.3
6	Container + pre-test dry mass (4)	g	716.1	711.2	789.1	622.5	611.1	631.3	643.3	655.2	666.0	678.9	680.9	696.7	797.7	820.4	721.0
7	Pre-test dry mass less container mass	g	441.6	453.9	492.2	486.3	475.1	475.8	486.6	499.4	508.1	507.5	523.4	538.8	540.4	545.9	538.6
8	Container + post-test dry mass (1)	g	551.6	549.7	597.4	549.1	515.5	534.9	587.3	600.7	592.4	632.2	617.5	642.8	755.5	778.1	693.7

Continued on next page

W (1)

SN	Item	Units	3% cc			5% cc			7% cc			9% cc			11% cc		
			1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
9	Container + post-test dry mass (2)	g	551.2	548.2	596.5	547.6	514.9	534.6	586.4	600.2	590.9	630.8	617.1	642.5	755.3	778.5	692.2
10	Container + post-test dry mass (3)	g	551.2	548.2	596.3	547.6	514.8	534.4	586.4	600.2	590.8	630.6	617.1	642.3	755.0	778.5	691.9
11	Container + post-test dry mass (4)	g	550.9	548.2	596.2	547.6	514.7	534.2	586.4	600.3	590.8	630.7	617.0	642.3	755.0	778.4	691.9
12	Post-test dry mass less container mass	g	276.4	290.9	299.3	411.4	378.7	378.7	429.7	444.5	432.9	459.3	459.5	484.4	497.7	503.9	509.5
13	Slake durability Index	%	62.6	64.1	60.8	84.6	79.7	79.6	88.3	89.0	85.2	90.5	87.8	89.9	92.1	92.3	94.6
14	Mean SDI	%		62.5			81.3			87.5			89.4			93.0	
15	Mean total mass loss	%		37.5			18.7			12.5			10.6			7.0	

CSSB (28-days, 6MPa)

LABORATORY RECORDING SHEET: SLAKE DURABILITY TEST

			SAMPLE TYPE														
			Cement-lime Soil Blocks (CLSB) – 28 days (6 MPa)														
			3% cc			5% cc			7% cc			9% cc			11% cc		
SN	**Item**	**Units**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**
1	Container Reference	-	C31	C32	C33	C51	C52	C53	C71	C72	C73	C91	C92	C93	C111	C112	C113
2	Container mass	g	157.5	156.8	136.0	296.9	257.3	274.5	155.5	171.4	136.2	182.4	157.5	156.7	155.8	157.9	155.7
3	Container + pre-test dry mass (1)	g	613.8	618.6	595.9	774.7	759.4	771.6	662.5	667.8	588.1	707.9	623.0	631.8	662.7	695.5	615.3
4	Container + pre-test dry mass (2)	g	613.3	617.9	595.1	773.9	758.3	770.5	661.9	667.3	587.5	707.8	622.7	630.9	662.3	694.8	614.6
5	Container + pre-test dry mass (3)	g	612.8	617.4	594.7	772.0	757.3	769.8	661.4	667.3	587.3	707.8	622.7	630.6	662.3	694.6	614.5
6	Container + pre-test dry mass (4)	g	612.8	617.4	594.7	772.0	757.3	769.8	661.4	667.2	587.2	707.7	622.7	630.6	662.2	694.6	614.5
7	Pre-test dry mass less container mass	g	455.3	460.6	458.7	475.1	500.0	495.3	505.9	495.8	451.0	525.3	465.2	473.9	506.4	536.7	458.8
8	Container + post-test dry mass (1)	g	411.6	391.1	379.6	650.0	637.8	642.9	592.8	584.7	513.3	638.5	573.9	566.2	607.6	635.4	559.4

continued on next page

W (2)

			3% cc			5% cc			7% cc			9% cc			11% cc		
SN	**Item**	**Units**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**
9	Container + post-test dry mass (2)	g	410.9	390.3	378.7	648.8	636.4	642.6	592.5	584.1	512.1	636.9	573.7	565.5	607.1	635.3	558.7
10	Container + post-test dry mass (3)	g	410.7	389.4	378.7	648.5	636.4	642.5	591.2	584.0	512.1	636.8	573.4	564.3	607.1	635.2	558.5
11	Container + post-test dry mass (4)	g	410.7	389.4	378.7	648.5	636.3	642.5	591.1	583.9	511.9	636.8	573.4	564.3	607.0	635.0	558.5
12	Post-test dry mass less container mass	g	253.2	232.6	242.7	351.6	379	368.0	435.6	412.5	375.7	454.4	415.9	407.6	451.2	477.1	402.8
13	Slake durability Index	%	55.6	50.5	52.9	74.0	75.8	74.3	86.1	83.2	83.3	86.5	89.4	86.0	89.1	88.9	87.8
14	Mean SDI	%		53.0			74.7			84.2			87.3			88.6	
15	Mean total mass loss	%		47.0			25.3			15.8			12.7			11.4	

CLSB (28 days, 6 MPa)

APPENDIX W (3)

LABORATORY RECORDING SHEET: SLAKE DURABILITY TEST

			SAMPLE TYPE														
			Microsilica-cement soil blocks (MCSB) – 28 days (6 MPa)														
			3% cc			5% cc			7% cc			9% cc			11% cc		
SN	**Item**	**Units**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**
1	Container Reference	-	MC31	MC32	MC33	MC51	MC52	MC53	MC71	MC72	MC73	MC91	MC92	MC93	MC111	MC112	MC113
2	Container mass	g	136.0	157.9	155.6	136.1	155.8	156.7	156.8	296.9	274.5	157.5	156.8	155.5	171.4	182.4	257.3
3	Container + pre-test dry mass (1)	g	623.3	619.6	634.7	643.9	683.8	735.4	657.5	810.6	810.8	650.1	647.7	649.9	695.5	729.8	803.6
4	Container + pre-test dry mass (2)	g	622.2	617.8	633.3	643.7	682.6	735.1	656.9	810.4	809.9	649.7	647.3	649.2	694.5	729.7	802.7
5	Container + pre-test dry mass (3)	g	622.2	617.8	633.3	643.5	682.3	735.1	656.7	810.3	809.9	649.6	647.2	649.2	694.3	729.6	802.7
6	Container + pre-test dry mass (4)	g	622.1	617.8	633.2	643.5	682.3	735.1	656.6	810.2	809.9	649.4	647.0	649.2	694.0	729.6	802.4
7	Pre-test dry mass less container mass	g	486.1	459.9	477.6	507.4	526.5	578.4	499.8	513.3	535.4	491.9	490.2	493.7	522.6	547.2	545.1
8	Container + post-test dry mass (1)	g	513.7	512.5	526.6	577.7	599.4	647.1	628.7	789.6	775.0	643.5	626.6	644.7	693.9	725.1	796.5

continued on next page

W (3)

			3% cc			5% cc			7% cc			9% cc			11% cc		
SN	**Item**	**Units**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**
9	Container + post-test dry mass (2)	g	512.8	511.3	526.4	576.6	598.8	647.0	628.3	788.4	771.3	642.3	625.8	643.5	692.6	723.9	795.8
10	Container + post-test dry mass (3)	g	512.8	511.1	526.4	576.5	598.7	646.6	628.2	788.1	773.7	642.1	625.0	643.3	692.5	723.6	795.4
11	Container + post-test dry mass (4)	g	512.7	511.1	526.2	576.5	598.6	646.6	628.1	788.1	773.5	642.0	624.9	643.3	692.4	723.6	795.3
12	Post-test dry mass less container mass	g	376.7	353.2	370.6	440.4	442.8	489.9	471.3	491.2	499.0	484.5	468.1	487.8	521.0	541.2	538.0
13	Slake durability Index	%	77.5	76.8	77.6	86.8	84.1	84.7	94.3	95.7	93.2	98.5	95.5	98.8	99.7	98.9	98.7
14	Mean SDI	%		77.3			85.2			94.4			97.6			99.1	
15	Mean total mass loss	%		22.7			14.8			5.6			2.4			0.9	

MCSB (28 days, 6 MPa)

LABORATORY RECORDING SHEET: SLAKE DURABILITY TEST

			SAMPLE TYPE															
			Fired brick samples (FBS); Concrete block sample (CBS); Rock block sample (RBS)															
			FBS (0% cc)			CBS (12-18% cc)			RBS (sandstone)									
SN	**Item**	**Units**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**							
1	Container Reference	-	Fb1	Fb2	Fb3	Cb1	Cb2	Cb3	Rb1	Rb2	Rb3							
2	Container mass	g	136.1	155.7	155.5	296.9	257.3	274.5	156.7	157.9	136.2							
3	Container + pre-test dry mass (1)	g	587.8	619.4	437.5	821.6	783.7	789.7	594.4	598.1	582.7							
4	Container + pre-test dry mass (2)	g	587.3	618.1	436.6	821.4	783.2	789.7	593.3	597.5	581.9							
5	Container + pre-test dry mass (3)	g	587.3	618.1	436.6	821.3	783.2	789.5	593.3	596.9	581.8							
6	Container + pre-test dry mass (4)	g	587.2	618.1	436.5	821.3	783.2	789.3	593.3	596.9	581.7							
7	Pre-test mass less container mass	g	451.1	462.4	481.0	524.4	525.9	514.8	436.6	439.0	445.5							
8	Container + post-test dry mass (1)	g	587.9	618.3	635.9	802.6	771.1	768.9	590.9	587.3	571.7							

continued on next page

			FBS (0% cc)			CBS (12-18% cc)			RBS (sandstone)								
SN	**Item**	**Units**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**						
9	Container + post-test dry mass (2)	g	587.4	617.2	635.6	802.2	770.5	768.8	590.8	587.1	571.5						
10	Container + post-test dry mass (3)	g	587.2	616.9	635.1	801.9	770.2	768.8	590.8	587.1	571.6						
11	Container + post-test dry mass (4)	g	587.2	616.7	635.1	801.9	770.1	768.7	590.7	587.2	571.5						
12	Post-test dry mass less container mass	g	451.1	461.0	479.6	505.0	512.8	494.2	434.0	429.3	435.3						
13	Slake durability Index	%	100	99.7	99.7	96.3	97.5	96.0	99.4	97.8	97.7						
14	Mean SDI	%		99.8			96.6			98.3							
15	Mean total mass loss	%		0.2			3.4			1.7							

FBS; CBS; RBS

APPENDIX W (5)

LABORATORY RECORDING SHEET: SLAKE DURABILITY TEST

			SAMPLE TYPE														
			Cement Stabilised Soil Blocks (CSSBs): 6 MPa; 5% cc														
			7 days			14 days			21 days			28 days			56 days		
SN	Item	Units	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
1	Container Reference	-	CS571	CS572	CS573	CS5141	CS5142	CS5143	CS5211	CS5212	CS5213	CS5281	CS5282	CS5283	CS5561	CS5562	CS5563
2	Container mass	g	136.0	155.5	136.2	297.6	296.9	297.0	257.3	297.6	275.1	274.5	313.9	257.4	171.4	157.5	156.8
3	Container + pre-test dry mass (1)	g	640.7	673.5	659.0	791.5	786.9	793.4	790.5	836.6	815.9	732.4	776.8	730.5	683.5	677.8	683.7
4	Container + pre-test dry mass (2)	g	640.1	671.8	658.1	791.3	786.7	791.6	789.6	836.3	815.3	731.2	775.1	730.2	683.2	677.6	683.3
5	Container + pre-test dry mass (3)	g	639.9	671.7	658.2	790.9	786.7	791.6	789.6	836.2	815.1	730.1	774.6	730.2	683.2	677.4	683.3
6	Container + pre-test dry mass (4)	g	639.8	671.7	658.1	790.9	786.7	791.5	789.6	836.2	815.1	730.1	774.6	730.2	683.2	677.4	683.2
7	Pre-test dry mass less container mass	g	503.8	516.2	521.9	493.3	489.8	494.5	532.3	538.6	540.0	455.6	460.7	472.8	511.8	519.9	526.4
8	Container + post-test dry mass (1)	g	372.3	386.4	378.8	586.5	583.7	594.5	635.9	686.1	672.8	641.5	695.5	644.7	587.8	591.6	585.3

continued on next page

W (5)

SN	Item	Units	7 days 1	7 days 2	7 days 3	14 days 1	14 days 2	14 days 3	21 days 1	21 days 2	21 days 3	28 days 1	28 days 2	28 days 3	56 days 1	56 days 2	56 days 3
9	Container + post-test dry mass (2)	g	371.9	385.9	378.4	585.9	583.4	594.3	635.8	685.5	672.1	640.3	695.3	644.3	587.5	591.2	584.9
10	Container + post-test dry mass (3)	g	371.8	385.8	377.9	585.7	583.4	594.3	635.7	685.4	672.1	640.3	694.9	644.1	587.5	591.1	584.8
11	Container + post-test dry mass (4)	g	371.8	385.7	377.8	585.7	583.4	594.2	635.8	685.4	672.1	640.3	694.9	644.2	587.5	591.1	584.8
12	Post-test dry mass less container mass	g	235.8	230.2	241.6	288.1	286.5	297.2	378.5	387.8	396.9	365.8	381.0	386.8	416.1	433.6	428.0
13	Slake durability Index	%	46.8	44.6	46.3	58.4	58.5	60.1	71.1	72.0	73.5	80.3	82.7	81.8	81.3	83.4	81.3
14	Mean SDI	%		45.9			59.0			72.2			81.6			82.0	
15	Mean total mass loss	%		54.1			41.0			27.8			18.4			18.0	

CSSB (6 MPa)

APPENDIX W (6)

LABORATORY RECORDING SHEET: SLAKE DURABILITY TEST

			SAMPLE TYPE														
			Microsilica-cement Soil Blocks: 6 MPa: 5% cc														
			7 days			14 days			21 days			28 days			56 days		
SN	**Item**	**Units**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**
1	Container Reference	-	MS571	MS572	MS573	MS5141	MS5142	MS5143	MS5211	MS5212	MS5213	MS5281	MS5282	MS5283	MS5561	MS5562	MS5563
2	Container mass	g	171.4	156.8	157.5	313.8	257.3	274.5	136.0	136.2	155.5	297.6	296.9	297.0	157.8	155.7	158.0
3	Container + pre-test dry mass (1)	g	633.0	626.9	611.3	864.3	808.4	811.4	630.8	627.5	644.7	834.6	834.5	822.9	649.8	646.7	654.6
4	Container + pre-test dry mass (2)	g	632.7	626.7	611.2	864.2	807.2	809.1	629.9	627.3	644.4	834.2	833.8	822.8	649.7	646.4	654.5
5	Container + pre-test dry mass (3)	g	632.5	626.7	611.2	863.9	807.2	809.1	629.8	627.3	644.3	834.1	833.9	822.7	649.7	646.3	654.4
6	Container + pre-test dry mass (4)	g	632.5	626.6	611.1	863.9	807.2	809.1	629.8	627.3	644.3	834.1	833.8	822.7	649.7	646.2	654.4
7	Pre-test dry mass less container mass	g	461.1	469.8	453.6	550.1	549.9	534.5	493.8	491.1	488.8	536.5	536.9	525.7	491.9	490.5	496.4
8	Container + post-test dry mass (1)	g	440.4	430.1	414.8	696.9	634.5	647.9	546.7	548.8	569.4	761.5	765.8	755.7	592.3	577.9	589.5

continued on next page

W (6)

SN	Item	Units	7 days			14 days			21 days			28 days			56 days		
			1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
9	Container + post-test dry mass (2)	g	439.9	429.8	414.5	696.8	634.1	647.7	546.3	548.3	568.6	761.3	765.2	755.6	592.2	577.1	588.9
10	Container + post-test dry mass (3)	g	439.8	429.8	414.4	696.7	634.0	647.6	545.9	548.1	568.5	761.2	765.1	754.9	592.2	577.1	588.9
11	Container + post-test dry mass (4)	g	439.8	429.8	412.4	696.7	634.0	647.6	545.9	548.2	568.5	761.1	765.1	754.9	592.1	577.0	588.9
12	Post-test dry mass less container mass	g	268.4	273.0	254.9	382.9	376.7	373.1	409.9	412.0	413.0	463.5	468.2	457.9	434.3	421.3	430.9
13	Slake durability Index	%	58.2	58.1	56.2	69.6	68.5	69.8	83.0	83.9	84.5	86.4	87.2	87.1	88.3	85.9	86.8
14	Mean SDI	%		57.5			69.3			83.8			86.9			87.0	
15	Mean total mass loss	%		42.5			30.7			16.2			13.1			13.0	

MCSB (6 MPa)

LABORATORY RECORDING SHEET: SLAKE DURABILITY TEST

			SAMPLE TYPE														
			Cement-Lime Soil Blocks CLSB): 6MPa; 5% cc														
			7 days			14 days			21 days			28 days			56 days		
SN	**Item**	**Units**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**	**1**	**2**	**3**
1	Container Reference	-	Cl5571	Cl5572	Cl5573	Cl55141	Cl55142	Cl55143	Cl55211	Cl55212	Cl55213	Cl55281	Cl55282	Cl55283	Cl55561	Cl55562	Cl55563
2	Container mass	g	297.6	296.9	257.3	155.5	156.7	157.8	274.5	313.8	257.4	171.4	157.6	156.8	136.0	136.2	157.5
3	Container + pre-test dry mass (1)	g	814.4	817.9	767.9	619.2	629.9	639.9	780.6	827.8	777.9	647.7	621.1	639.9	561.8	566.5	603.7
4	Container + pre-test dry mass (2)	g	813.4	817.1	767.8	618.9	629.7	639.6	780.5	827.5	777.5	646.9	620.9	639.7	561.7	566.2	603.3
5	Container + pre-test dry mass (3)	g	813.3	817.0	767.7	618.8	629.5	639.4	780.4	827.5	777.3	646.9	620.7	639.6	561.6	566.3	603.3
6	Container + pre-test dry mass (4)	g	813.3	817.0	767.7	678.8	629.6	639.4	780.4	827.4	777.2	646.9	620.7	639.6	561.6	566.2	603.4
7	Pre-test dry mass less container mass	g	515.7	520.1	510.4	463.3	472.9	481.6	505.9	513.6	519.8	475.5	463.1	482.8	425.6	430.0	445.9
8	Container + post-test dry mass (1)	g	508.6	495.7	463.7	400.5	401.4	438.2	596.5	645.3	590.3	527.7	510.4	520.6	472.3	477.4	506.7

continued on next page

SN	Item	Units	7 days 1	7 days 2	7 days 3	14 days 1	14 days 2	14 days 3	21 days 1	21 days 2	21 days 3	28 days 1	28 days 2	28 days 3	56 days 1	56 days 2	56 days 3
9	Container + post-test dry mass (2)	g	507.5	495.2	463.6	400.3	401.2	437.9	596.4	646.9	589.9	527.7	501.9	520.1	471.9	477.2	506.3
10	Container + post-test dry mass (3)	g	507.0	495.2	463.6	400.1	401.2	437.8	596.3	646.8	589.6	527.6	501.7	519.9	471.9	477.2	506.2
11	Container + post-test dry mass (4)	g	507.0	495.1	463.5	400.1	401.2	437.8	596.3	646.6	589.6	527.5	501.7	519.9	471.8	477.2	506.2
12	Post-test dry mass less container mass	g	209.4	198.2	206.2	244.6	244.5	280.0	321.8	332.8	332.2	356.1	344.1	363.1	335.8	341.0	348.7
13	Slake durability Index	%	40.6	38.1	40.4	52.8	51.7	51.5	63.6	64.8	63.9	74.9	74.3	75.2	78.9	79.3	78.2
14	Mean SDI	%		39.7			52.0			64.1			74.8			78.8	
15	Mean total mass loss	%		60.3			48.0			39.5			25.2			21.2	

Attitudes towards Rainwater Harvesting

0. Summary

- increasing interest and formation of national rainwater associations in East and Southern Africa
- NGOs in West Africa starting to realise the potential of rainwater harvesting; some local cultural obstacles seem to exist
- Donor organisations funding development aid are generally interested in rainwater harvesting. In cases where organisations have specific water policies and strategies, rainwater harvesting is either specifically mentioned as an "alternative water source" or implied under "appropriate technologies"
- Many of the donor organisations who responded to our questionnaire have already funded in one way or the other rainwater harvesting
- Potentials of rainwater harvesting are seen in areas
    - that cannot be served by standard technologies such as groundwater and handpumps or
    - have contaminated/unusable groundwater
- Obstacles to rainwater harvesting are
    - the unconventional approach needed for dissemination and small and very local investment on the household level
    - finance of investment cost for the water storage facility
- Water legislation in South Africa specifically mentions rainwater as a permissible source of water but forces at the same time users to use only water from an authorised service provider

1. Introduction: New developments in rainwater harvesting in Africa

Interest in rainwater harvesting is on the increase in many countries of Eastern and Southern Africa. National rainwater harvesting associations are being formed in Eastern and Southern Africa. The first one of these national associations was the Kenya Rainwater Association (KRA), founded in 1995. Three national associations have been formed in 1998 alone: in Ethiopia, Uganda and Zimbabwe. The associations are usually started at national rainwater workshops. They see their tasks in information dissemination, lobbying and networking.

For SIDA, the Swedish International Development Agency, the

> ***potential for household-level rainwater harvesting in Ethiopia and Eritrea is considered to be enormous and will be investigated.***

There is a serious initiative in Southern Africa to promote knowledge on rainwater harvesting and the implementation of it. Participating countries are Namibia, South Africa, Mozambique, Zimbabwe and Zambia.

2. Local NGOs and rainwater harvesting in West Africa

While a lot of interest is being noticed in Eastern and Southern Africa, no important activities in rainwater harvesting are reported from West Africa. Therefore several video shows and discussions have been organised in selected countries (Senegal, Ghana, Togo, Benin) to see the reactions of NGOs re. rainwater harvesting[i],[ii],[iii].

Some general observations by the organisers of the video shows on rainwater harvesting:

Interest

- Women groups and women departments of NGOs are much more interested than general development departments of NGOs (where usually many men are represented)
- Interest was greatest in areas where there were real (seasonal) shortages of water. This was esp. true for Togo and Ghana with very low water levels of lake Volta leading to rationing of water and electricity. This also affected water pumping and caused many shortages in the distribution system!

Problems

- Rainwater harvesting was sometimes associated with traditional or pagan culture esp. in urban environments
- As rain and rainmaking is often associated with witchcraft, some church groups have difficulties also with rainwater harvesting, being seen as pre-Christian, "backward" practice

Utilisation

- The potential of rainwater harvesting in mobilising own resources, materials and skills was seen by many groups, but not always
- The potential for local artisans and small industry opens chances for local employment
- The improvement of general living conditions by rainwater harvesting was appreciated
- Many people saw rainwater harvesting as an ideal supplement to the public (and unreliable) water supply
- There was a great demand of how-to-do materials (booklets, brochures) to try and put rainwater harvesting into practice as seen in the video.

3. Water Professionals and rainwater harvesting

A letter with an associated questionnaire has been designed. A representative sample of water professionals has been selected. The mailing is being done at present.

4. Donor organisations and rainwater harvesting

4.1. Survey

Donor organisations have been contacted in order to find out their policy and associated strategy, their intervention tools and their partners in project planning and implementation. Questions related also to previous involvement in rainwater harvesting as well as potential advantages perceived as well as objections. Letter and questionnaire sent are given as annex 1, the list of donor organisations contacted as annex 2 (The Collaborative Council made it clear that they are not a donor organisation).

A total of 18 donor organisations were contacted:
- 6 mulitlateral aid organisations
- 8 bilateral aid agencies
- 4 "donor NGOs"

Answers came from 9 out of 18 contacted organisations with varying degree of detail.
Sending questionnaires to unknown people has very little chance of getting an answer, as was also the case in this situation. At least one reminder is necessary. Organisations generally responded to e-mail inquires more readily than to letters sent by fax or mail. Organisations involved in rainwater harvesting were more readily giving some feedback than those not involved.

4.2. Results

4.2.1. Water policies / strategies

Every donor organisation who responded showed interest in rainwater harvesting and often asked for more information or expressed interest in the results of the sudy!
Not every donor organisation has its own sectoral policy for water. Organisations like Water Aid have a strategic framework, which lays out general aims and strategies. But each country office has its own strategies which states how that country office will contribute to the general objectives. So WaterAid could easily have many different views on the appropriateness on rainwater harvesting and find no contradiction in that! Similar approaches hold true for UNICEF (...”our country offices independently design strategy and implementation methodologies”) and many donor NGOs (e.g. church donor organisations).

Out of the four policy documents of contacted donor organisations(see Annex 3), two of them specifically mention rainwater harvesting:
UNICEF gives support to specific “mini-projects” in addition to standard water and sanitation interventions within its “programme strategies”, inter alia:

> ***Rainwater harvesting, using roof tops as well as through the construction of underground check dams and dikes***

NEDA assigns an importance to our topic by stating in the Technology part of the Policy section:

> ***The development of techniques for harvesting rainwater for drinking purposes will be fostered,...***

The two other documents not yet quoted mention explicitly the promotion and support of appropriate technology (or appropriate products). For SDC, rainwater harvesting is included in this context:

> ***In view of our technical strategies related to the use of appropriate technology, rainwater harvesting is very much in line with our sector policy (quote from letter of .SDC to the author).***

SDC further elaborates its strategies in the economic field by stating:

A reliable and sustainable water and sanitation infrastructure depends on

- ***appropriate technology (see above)***
- ***promotion of local construction.***

  <u>Household</u> and community-level water and sanitation installations will stand a better chance of being used in a sustainable manner if they are built and maintained with local material and know-how,...

The emphasis on household solutions is not found in the other documents studied but clearly indicates an important prerequisite for the widespread use of roofwater harvesting.

4.2.2. Current involvement in rainwater harvesting programmes

Half of the organisations (UNICEF, SDC, WaterAid, SIDA) who answered stated that they already had funded projects which included at least a rainwater component. SIDA has – according to the documents received – a defined strategy to promote and spread rainwater harvesting in East and Southern Africa.

4.2.3 Potential applications, obstacles to rainwater harvesting

UNICEF mentions the need to supply people under difficult conditions, where rainwater harvesting will become important:

> ***...as safe water coverage increases, the remaining unserved people tend to be those that cannot be served by more traditional technologies (notably groundwater and handpumps) and thus alternative technologies such as rainwater harvesting will become more important.***

The other important area where rainwater harvesting is seen as a viable solution is in areas, ***where groundwater contamination is on the rise*** (as seen by UNICEF and SDC). SDC mentions the example of Bangladesh with about 2 million handpumps which lay in areas where the groundwater is containing arsenic!

...rainwater catchments maybe another interesting alternative

SIDA talks about HRWH, household level rainwater harvesting. They quote Ministers (e.g. of Uganda) asking donor agencies to consider and give more attention to HRWH. But they also see some difficulties:

- The participatory approach required for dissemination (of HRWH) is unconventional and investments are small, numerous and very local. The existing institutions are not geared to working with this kind of projects and may be reluctant to do so.
- Another main constraint is finance of the investment cost for the water storage facility. Many households have great difficulties to accumulate funds for the investment and have no access to credit.

5. Water legislation in view of rainwater harvesting

5.1 South Africa

Water legislation of South Africa was considered relevant, since it is very recent (1997/1998). South Africa has tried to incorporate present knowledge of water resources and their sustainable use into the new legislation (for details of acts referred to, see Annex 4).

Chapter 1 (Interpretation and fundamental principles) of the National Water Bill (1998) defines the entitlement to water use (chapter 4) and refers to Schedule 1. This schedule (permissible use of water) is mentioning roofwater as a permissible water use:

A person may, subject to this Act -
(c) store and use run-off water from a roof.

However this right is made relative in the Water Services Act of 1997, stating in chapter 1 (introductory provisions), paragraph 6: Access to water services through nominated water services provider:

... no person may use water services from a source other than a water services provider nominated by the water services authority...

Users of rainwater can come into conflict with the water services act, if there is a nominated water services provider in the area. Clarification of this possible conflict will be sought together with the new initiative of rainwater harvesting in Southern Africa.

[i] Günther Rusch: Rain is water: Bericht über Filmvorführungen und Diskussionen während einer Westafrikareise (Report on video shows and discussions during a visit to West Africa)
[ii] Pierre Jekinnou: Quelques observations (some observations...on discussions with NGOs on rainwater harvesting)
[iii] Personal communication with Günther Rusch

Attitudes towards Rainwater Harvesting

Annex 1: Letter and questionnaire to donor organisations

Dear water professional,

I'm writing this letter in the hope that you may be able to assist us in our research programme, called „Domestic Roofwater Harvesting in the Humid Tropics". It is a 3-year-programme to generate reliable information for water policy planners, water supply professionals and ultimately householders. The programme just started and is funded by the EU. It involves 4 partners from India, Sri Lanka, England and Germany. Links are being developed with practitioners in Central America and East Africa.
The programme will examine literature and practice from many parts of the world, but it is expected that those from humid tropical areas will be of most use, since Domestic Rainwater Harvesting technology and economics are dominated by factors like climate and culture.

In the view of water resources getting scarce, it is becoming obvious that we should use every available water resource as e.g. rainwater. Rainwater harvesting has been and is successfully practised for millennia around the Mediterranean as a supplementary source of water or the only one available. In many countries of Asia, Africa and Latin America, it is currently newly introduced or its use widened. One of several components of the programme is to define the information needs of organisations active in the water sector. We would therefore like to ask you about your funding policy for the water sector and how it is implemented in the different countries.
We will compile our research findings (we just have started) and will make them available to you if you wish so. They will also be available on a web site, which will soon be established. Please feel free to contact me for any additional information you might want to get.
Thanking you in advance for your time and efforts invested

- also in the name of the other partners involved

Hans Hartung
Responsible Task Manager for Task B: Institutional Values and Decision Making

Domestic Roofwater Harvesting in the Humid Tropics

A. Water programmes

We would be grateful, if you could give us some details about the general way of your funding of water projects
– maybe documents are available which would help us to explain the procedures:

a) Aid policy and strategy related to water
   - in general
   - is rainwater harvesting in line with your funding policy or not?
     If not, why not, are there any objections?

b) Intervention tools for the water sector within you organisation
   - on the policy level
   - on the project level
   - in technical assistance in general

c) Partners in project planning and implementation
   What kind of partners are you working with?

B. Rainwater harvesting programmes

a) Have you already (co)funded projects where rainwater harvesting was involved?

If yes:

Would it be possible to get details about the funded programmes?
(reports if possible?)
What kind of funding strategy did you use?
Do you have general documents on rainwater harvesting considering their successes and failures, highlighting components of the project to be considered in similar projects?

If no:

Do you have general objections to rainwater harvesting and if so, what are they?
In particular do you consider financing single households to be in line with your house rules?
What kind of information on rainwater harvesting would you be interested in receiving?

Annex 2: Organisations contacted

Title	Participant	Function	Organisation	Address	City
Mr.	John Briscoe	Division Chief	The World Bank, Water and Sanitation Division, Transport, Water and Urban Development	1818 H Street N.W.	Washington D.C. 20433
Mr.	Frank Hartvelt	Deputy Director	Division for Global and Interregional Programmes, United Nations Development Program	One UN Plaza	New York NY 10017
			Water and Environmental Sanitation Section UNICEF (DH-40B)	3, United Nations Plaza	New York NY 10017
Mr.	Liabaert	VIII E4	European Commission	200 rue de la Loi,	1049 Brussels
Mr.	S.A. Baha	Director	Asian Development Bank Infrastructure Department	P.O.Box 789	1099 Manila
Mr.	Dennis Carroll		US AID Bureau for Global Programmes, Field Support and Research,Office of Health		Washington DC 20523-1817
Mr.	A. Hartmann		Swiss Development Corporation SDC Water and Infrastructure Service		CH-3003 Bern
Mr.	Ingvar Andersson and Mrs. Margaretha Sundgren		SIDA Infrastructure Division	Birger Jarlsgatan 61	S-105 25 Stockholm
Mr.	Henning Jensen	Senior Technical Adviser	Ministry of Foreign Affairs, Danida	2 Asiatisk Plads	DK-1448 Copenhagen K
Mr.	Joep Blom		Ministry of Foreign Affairs Directorate General for International Cooperation	P.O.Box 20061	2500 EB The Hague
Mr.	Guy Carrier	Senior Adviser	Canadian International Development Agency (CIDA) Water and Sanitation Policy Branch	200, Promenade du Portage	Hull (Quebec)
Mr.	J. Hodges and Mr. H.B. Jackson		DFID	94, Victoria Street	London SW1E 5JL
Mr.	Roy Hewson and Mr. John Casey		Australian International Development Assistance Bureau (AIDAB), Development and Coordination Section	GPO Box 887	Canberra ACT 2601
		Technical Adviser, Water and Infra-structure	Christian Aid	P.O. Box 100	London SEI 7RT
Mr.	Van Damme		Water & Sanitation Collaboration Council	1 Poellaan 59	Lisse 2161
Herrn	Pankert		Misereor Bauabteilung	Postfach 40 50	52064 Aachen
Mr.	Dave Mather		Water Aid	Prince Cousort House, 27-29 Albert Embankment	London SE 1 7 UB
		Technical Adviser	OXFAM	274 Bambury Road	Oxford OX2 7DZ

Annex 3: Water Policy Documents

1. SDC Sector Policy on Water Supply and Sanitation, Series SDC Sector Policies, Swiss Development Cooperation, Berne May 1994

2. Water supply and Sanitation in Developing Countries, Sectoral Policy Document of Development Cooperation, NEDA (Netherlands Development Assistance), Ministry of Foreign Affairs, The Hague, 1998

3. UNICEF Strategies in Water and Environmental Sanitation, UNICEF, New York, 1995

4. Water Policy Issues, prepared by J. Winpenny for Department for International Development (DFID), July 1997

Annex 4: Water Laws

Republic of South Africa:

- National Water Bill (as amended by the Portfolio Committee on Agriculture, Water Affairs and Forestry (National Assembly)), [B 34B-98]
- Water Services Act, 1997 (Act No. 108), as published in the Government Gazette Vol. 390, No. 18522, Cape Town 19.12.1997

Republic of Kenya

- The Water Act, Chapter 372, revised edition 1972

Domestic Rainwater Harvesting:
Perceptions of Water Professionals and the way forward

1. Survey

A survey of water professionals and their perceptions on rainwater harvesting was done, using a questionnaire and personal interviews

The experience from previous surveys was taken into account: No fax or mail questionnaires were sent out. The distribution of the questionnaires was exclusively done by e-mail. Around 73 professionals of the a.m. groups were contacted. The result was not very encouraging, only around 12% (9 persons) of the contacted people answered, most of them already being familiar with rainwater harvesting.

The mailing list of GARNET, the Global Applied Research Network in Water Supply and Sanitation (called: "water-and-san-applied-research) was also used to distribute the questionnaire to more than 200 practitioners in the water field. Only three people responded.

Beside the mail survey professionals met at conferences (WEDC-conference 1999, Addis Ababa; World Water Forum 2000 in The Hague) or during travels in Africa (Southern Africa, May 2000) were interviewed.

2. Results

2.1 Respondents to the Water Professionals Questionnaire

A total of 26 people answered either orally or by writing.
More than half (14) of the people who answered orally or by writing use either rainwater harvesting in their house or are involved with rainwater harvesting programmes or projects.
Three of them came to rainwater harvesting when assessing water resources of small islands.
It is obvious from the answers, that water professionals are not aware of the full range of possibilities of rainwater harvesting. Instead, they discuss the topic only for a specific application they know (e.g. only rainwater harvesting in the rainy season, only for a desert location,...). Rainwater harvesting is mostly associated with drylands by them so that even the title of the questionnaire/project (Domestic Roofwater Harvesting in the humid tropics) is overlooked.

2.2 Future role of Domestic Roofwater Harvesting

The wording of the question was as follows:

> *"Will domestic roofwater harvesting play an important role for specific areas in securing drinking water needs of the future ?"*

Three people who responded were not in favour of roofwater harvesting: they either had enough water within their own area, did not know anything about it or saw it as only a last resort if everything else fails
All others saw an important role of DRWH for the future in a resource threatened world. Specific geographical areas were mentioned:

- small islands
- peri-urban areas: the opinion on peri-urban areas was divided. While many saw a great potential for rainwater harvesting (corrugated iron roofs, water by vendors very costly, reduces flooding and can increase groundwater recharge) one person opposed: people do not own their shack and its owner does not own the land it is built on, there is generally very little space for storage in these surroundings
- dry (arid and semi-arid) areas (specifically in India, where rainwater harvesting was traditionally practised on a large scale, declined in the last decades and is now starting to be revived).

Problems experienced with water were cited for which rainwater harvesting could make a contribution towards a solution:
- **decreasing water availability** (i.e. in ground- and surface water sources). One person (of several who cite this problem) suggested that by using rainwater for non-potable purposes, drinking water sources could be conserved! (while others see the value of rainwater especially in its good quality!)
- **bad infrastructure** with unreliable supplies, esp in peri-urban areas
- **no access to ground- or surface water**
- **rising water prices** for piped water supplies (esp. in European countries)
- **long distances and time spent** to fetch water in rural areas of the South
- **decreasing water quality** due to contamination of water sources.

3.3 Advantages of Domestic Roofwater Harvesting

Many advantages of domestic roofwater harvesting are seen:
- **closeness** to the home. This aspect was mentioned most.
- **simple technology** and consequently simple operation and maintenance, an advantage esp. for rural areas with weak infrastructure
- **reliable and controllable source (because of individual ownership)** of good quality water. This statement is not shared with other respondents. Some of them point out the reasonable quality of rainwater or the easiness to improve the quality by simple means. Other see (bacteriological) quality as the main concern with respect to rainwater (see chapter 3.4). Good chemical quality is especially valued in areas with highly contaminated groundwater (such as with fluoride or arsenic). The soft rainwater is very suitable for washing with little soap.
- **economic advantages** (compared to standard solutions or water to be bought from vendors). This is certainly site specific and must be established at the location of the intended use of rainwater. The collected rainwater can also be used for kitchen gardens and animals and thus improve the household economy.
- **increases the available options** to a community (in some areas it might be the only option)
- **reduction of soil erosion,** flood water retention, can improve ground infiltration, but also protection of house walls from rainstorms

3.4 Concerns with regard to Domestic Roofwater Harvesting

The wording of the question:

"What are your concerns with regard to domestic roofwater harvesting:
- *water quality?*
- *water quantity?*
- *other concerns?"*

Water quantity was voiced 6 times as a concern. Many respondents are fixed to rainwater harvesting during the rainy season, since otherwise the storage cost would get too high. This shows the very different and often limited knowledge of rainwater harvesting. Reliability of the water source was seen as a draw back as rain does not follow a regular pattern.
In-county information on rainfall data and specific water demand (of different water user groups for different purposes) was considered difficult to obtain in many cases.
Construction skills was seen as a serious concern, because of much damage to rainwater harvesting having been done in the past by bad design and construction. Tank building will always be needed so that specific and thorough training for artisans was demanded.
Maintenance of roofs and tanks was seen missing, i.e. essentially cleaning of these two components in regular intervals.
No standard solutions are available or are considered possible at all by one water professional. Conditions for the use of rainwater are so much different and rainwater harvesting could be serving different purposes in different surrounding so that site specific designs have to developed and experimented with. No design criteria to incorporate the varying parameters are available.
Water quality was the concerned voiced by half of the respondents. This refers to the bacteriological quality. While 3 water professionals are convinced of the "excellent" quality of rainwater (or that its

quality can be kept high), others see rather problems with the (bacteriological) quality of rainwater when its use is intended for drinking purposes:

- problems of birds and their droppings especially on islands
- intrusion of small mammals in the storage container
- cleanliness of roof (and storage)
- collection from thatched and earthen roofs (still used as an important roofing material for houses in rural areas).

There is a persistent misconception of water getting bad over time when stored! This is actually not true; research shows that rainwater quality gets better with time when certain criteria are met (no light into the storage, protection from whatsoever animals, aeration).
For one professional, it is without doubt that the quality of rainwater cannot satisfy drinking water requirements and suggests solutions: solar heating of the water as a treatment process.

3. The next steps

Rainwater harvesting is most often associated with

- rural areas and
- (semi)-arid climates.

The title of the research project "Domestic roofwater in the humid tropics" shows already that the scope for rainwater harvesting is much larger. The humid tropics have indeed some advantages for rainwater harvesting (e.g. small storage size)! At the same time the need for water is getting obvious in towns and cities and also here, rainwater harvesting could play a role.

3.1 Peri-urban areas

Although the introduction of rainwater harvesting into peri-urban areas was discussed very controversial and some respondents were opposed to it, it seems that there is an enormous need for water and a great potential for rainwater harvesting.
Urban water supply organisations seem rather helpless of how to tackle the problem of ever increasing peri-urban areas and their water supply. In the meantime street vendors sell water of doubtful quality at very high prices.
The advantages of domestic roofwater harvesting such as closeness to the home, economic advantages, simple technology and individual ownership (in an environment where often people come from different backgrounds and regions and are not willing to co-operate) combine in favour of its use in this environment. Further action research and closely monitored implementation of larger scale field trials are important!

3.2 Water quality

Half of all people interviewed voiced their concern on water quality of rainwater harvesting. This is the first argument that come to peoples mind when they are asked about rainwater. Quality means in this context always bacteriological quality. (Rainwater is one of the important options in areas with serious groundwater pollution with arsenic of fluorides).

Information on bacteriological quality of stored rainwater but also suggestions for treatment (before entering a storage tank, in it of after its withdrawal) must be made available as clear and simple messages.

3.3 Closing the information

Many people interviewed had never heard of rainwater harvesting as an option in water supply: Yes, in their village at home they tried to collect water from the roof when it rained with a pot but can this seriously contribute to household water supply? Half of all people of the survey specifically asked for more information, some for examples of where it was used with success. How can awareness on rainwater harvesting be created among water professionals? How can information reach the people in demand of water? In what form, in what messages should the information be conveyed?

Annex 1
Domestic Roofwater Harvesting in the Humid Tropics, a short questionnaire

Will domestic roofwater harvesting play an important role for specific areas in securing drinking water needs of the future?

yes, because of

I'm specifically thinking of these areas

no, because of

In order to make an informed decision, I would need reliable information on

What are your concerns with regard to domestic roofwater harvesting:

water quality, please specify

water quantity, please specify

other concerns like

or

Advantages often associated with domestic roofwater harvesting are its being close to the house and the individual ownership. Such harvesting may be operated for full or partial water supply. What do you consider as advantages?

Have you had any close experience with domestic roofwater harvesting?

Yes, I use it in my house

Yes, I have been involved in projects where roofwater harvesting played a role.

We, the Rainwater Harvesting Research Group would appreciate, if you could give us details

No, never had experience but I'm interested

No, I can't think that it can be useful

Yes, I'm interested in roofwater harvesting, specifically in

Name/Organisation

Address

Phone/Fax

E-mail

I'm expressing my personal opinion

I'm expressing the organisation's opinion

Thank you very much for your time!

Hans Hartung, FAKT, Kanalstr.23, D-97990 Weikersheim, Germany, fax +49.7934.990031

The inclusion of domestic roofwater harvesting (drwh) in a national water legislation framework

esp. looking at Botswana, Ethiopia, Kenya, Lesotho, Namibia, South Africa, Tanzania, Uganda and Zambia

by Hans Hartung & Christine Patschull, FAKT, Germany

The major objective of water law is to establish a framework for the protection and control of water resources in a country. A Water Law defines the legal entitlement to water and identifies the rights and obligations tied to water use and thus provides the prescriptive parameters for its development.

Until now, water laws don't deal with **domestic roofwater harvesting (drwh)** as the direct collection of roof runoff and storage for later use. So far, drwh is working outside the legal framework on project-level. Practitioners are often rather glad to not wake up sleeping dogs, instead of working on appropriate legislation, and follow a strategy to first create facts without too much government involvement.
This practice implies a lot of limits to drwh. While individual village-level drwh projects may succeed in the short term, their long terms sustainability may be severely tested in the absence of an appropriate institutional and legal framework at all levels. It is also unlikely that widespread replication of appropriate technologies and community-based implementation strategies will be achievable in the absence of supportive institutions at higher levels, even if isolated project success may be possible. Moreover, we will show that the organisation of the water sector has severe direct and indirect implications for the promotion of drwh and thus can't be ignored.

WATER LEGISLATION AND ITS INDIRECT IMPACT ON DRWH

Water legislation of 10 countries in Sub-Saharan Africa was reviewed, looking specifically at how it relates to domestic roofwater harvesting The national water laws of *Botswana, Ethiopia, Kenya, Lesotho, Namibia, South Africa, Tanzania, Uganda, and Zambia* reveal that drwh is nowhere part of a national water management strategy. SA is the only country, where drwh is mentioned as a possible supply for private households. Thus, drwh plays no role in water legislation, but as we will see, the legislation has lots of indirect impacts on drwh.

- **the legal status of (rain-)water**

Except in SA, where everyone has a right to water supply, all water resources including rainwater are treated as a property.

	Water Resources owned by	**Controlled by**	***Quote national water law***
Bots wana	Public	Govern-ment	*There shall be no right of property in public water (II.4)*
Ethio-pia	People		*All water resources are the collective property of the Ethiopian people*

Kenya	Govern-ment	Minister	*The water of every body of water or upon any land is vested in the government. The control of every body of water shall be exercised by the minister in accordance with this Act. The right to the use of every body of water is hereby declared to be vested in the Ministry. (II.3, II.4)*
Lesotho	Nation	Minister	*The ownership of all water within Lesotho is vested in the Basotho Nation. The Power to control and regulate the use of water shall be exercised by the Minister. (5.1., 5.2.)*
South Africa	No pro-perty, but a public right	Govern-ment	*Everyone has a right to basic water supply and basic sanitation (Water Services Act 97, 3.1.) The national Government, acting through the Minister, has the power to regulate the use, flow and control of all water in the Republic. (3.3.)The Minister is ultimately responsible to ensure that water is allocated equitably and used beneficially in the public interest, while promoting environmental values (3.2., National Water Act 98).*
Uganda		Govern-ment	*All right to investigate, control, protect and manage water in Uganda for any use, is vested in the government and shall be exercised by the Minister and the Director in accordance with the provisions of the part of the schedule. (II.I.5)*

- **extra government-permits/ land ownership required**

No matter whether someone is supplied with public water or not, extra permits are required in some countries for the private installation of drwh-tanks on household-level. Public allocation of water resources dominates in these countries through the granting of permits on water use. The state holds water resources as an aspect of sovereignty and cannot alienate such ownership of the basic resource and concomitant responsibility. In some countries you also need to own the land, which probably makes drwh impossible in peri-urban areas

	Permit/ Ownership required	***Quote***
Botswana	No, if you own the land	*The owner or occupier of any land may, without a water right construct any works thereon for the conservation of public water, and abstract and use public water so conserved, for domestic purposes. (II,6,b)*
Ethiopia	**No**	*No person may construct waterworks or withdraw water from a water resource, either for its own use or for the supply to others (5.1./1998) No permit shall be required for domestic use (21.1)*
Kenya	Yes, for the waterworks, you have to own the land	*A permit is not required for the abstraction or use of water from any body of water for domestic purposes by any person having lawful access thereto, if such abstraction is made without the employment of works (VIII, 38.)*

Lesotho	Yes, for the waterworks, you have to own the land	*Any person having lawful access to water may abstract and use such water for domestic purposes (3.2.)* *Whenever a water use requires the construction of water works the water officer shall grant the water use permit subject to the construction of such waterworks within the specified time under specified conditions. (4.6)*
Namibia	(forbidden)	-
South Africa	**No**	*A person may, subject to this Act* *take water for a reasonable domestic use in that person's household, directly from any water resource to which that person has lawful access* *(c) store and use run-off water from a roof*
Uganda	Yes, for the waterworks, no matter if you own the land	*No person shall acquire or have a right to (a) use water, (b) construct or operate any works. (II.I.6)* *A person may while temporarily at any place, or being the occupier of or resident to any land, where there is a natural source of water, use that water for domestic use. (II.I.7,1)* *No person shall construct or operate any works unless authorised to do so under this Part of the Statute. A person wishing to construct any works or to take and use water may apply to the Director in the prescribed form for a permit to so.*

In Europe**, Germany** is leading the way in encouraging widespread utilisation of rainwater catchment systems both for domestic supply and other purposes. Interest in household rainwater catchment focuses mainly on non-potable uses such as garden watering, toilet flushing and use of washing machines (Gould, S. 219).
There is no comprehensive and/or systematic legislation across Germany. Rainwater harvesting in German legislation is hindered through a general obligation to connect to and use mains water supplied by the local service provider (based on local government regulations which originated from health precautions). Service providers can make it difficult to use rainwater on the basis of these regulations. Clients of the service providers have however a right to be granted a partial lifting of the obligation to be connected and use the mains water with arguments of appropriateness and economic reasoning (Koenig, S. 76/77).
Many court cases favour these arguments. There is also a DIN (German standard) being at approval stage at the moment (mid-2001) to standardise rainwater utilisation technically.

The city of **Bangalore, South India** will be the first city in the country to have a rainwater harvesting policy. With an average rainfall of 900 till 970 mm over 7 months, Bangalore is at an elevation of 900 MSL and water has to be pumped in from 400 MSL. Thus, the pumping costs are enormous and so are power charges. Water rates are the highest in the country.
Without back-up by any legal provisions, 500 to 600 houses have drwh systems in place now and the number is growing. The Rainwater Club, a local NGO, developed not only very innovative approaches like the rendering of roofs for rainwater harvesting, but handed in now a draft policy mapping all possible sources for

harvesting rainwater. Cities like Chennai and Hyderabad already have rainwater harvesting regulations incorporated in the city municipal bye-laws, but only for multi-storeyed buildings. The Bangalore approach proposes to incorporate this into bye-laws for all new constructions and has taken into consideration all land uses – residential, institutional and commercial. Some of the government buildings will be used for demonstration. (For more information: www.inika.com\rainwaterclub)

WATER SECTOR ORGANISATION AND ITS DIRECT IMPACT ON DRWH

Each of the reviewed countries experienced a **water sector reform** during the 90s and thus, modernised the water law. The new laws reflect the global trend to decentralisation and commercialisation/privatisation of management, operation and maintenance of water services. The new laws will have an impact on the attitude (and the fact) that water is no longer supplied by the central government, but it will not necessarily get down to influence the traditional practice of multiple sourcing. There is a tendency that the newly established regulators will push for 100% metering and billing of water according to consumption without leaving customers the choice to select their water source (according to purpose or availability) and develop their own initiatives. The decentralised and privatised water sector doesn't solve the problem of access to water in rural areas.

- None of these reforms includes rwh as element of a multi sourcing-strategy. As legal schemes remain only pipelines, wells and tankers. One element of each reform is the push for 100% metering and billing without choice.
- Decentralisation took place only as far as Operation + Maintenance is concerned. The service functions became local while decision and regulatory functions - i.e. the power to implement a drwh-policy - remain central.
- Still communities are not allowed to develop their own drwh-policy.

In **Ethiopia**, the water sector was reorganised in 1992 with the establishment of federal regional water bureaus nominated by the Minister as one major renewal. The ownership as well as operation of maintenance of water services were thus decentralised on regional level, and the introduction and implementation of rwh-policies are legally up to the concerned regional bureaus since "no municipality or town shall engage in the planning and development of water supplies from any water resource or in the construction of water works without the permit issued by the implementing organ." (22.1a) Hence, there still are no self-reliant community-water-politics in Ethiopia possible that would allow users to be supported (at least with a legal framework) in having multiple sources to have a secure household water supply.

In 1999 **Bolivia** privatised Cochambamba's water system under instruction from the World bank. The British-based company that took over increased water prices by up to 200 %. Even collecting rainwater in rooftop tanks became illegal without a special permit. The crippling price rise sparked mass protests, in which tens of thousands of people took to the streets. In the end the government took back control of the water supply and re-legalised rwh. (For more information see www.peopleandplanet.org/tradejustice/briefing.asp)

	Water Sector Reform	Implementation of rwh-politics by
Ethiopia	Reorganisation of the water sector in 1992 establishment of regional water bureaus, nominated by the Minister, which own and operate supply systems	the implementing organ is the ministry as regards transboundary rivers and water bodies that flow across more than one regional government a, and the concerned regional water bureau as regards regional water resources (Art.2 def.) No municipality or town shall engage in the planning and development of water supplies from any water resource or in the construction of water works without the permit issued by the implementing organ. (22.1a)
Kenya	Decentralisation currently in progress: The role of the government will be redefined with emphasis on the regulatory and enabling functions as opposed to direct service provision. The government will ensure private sector participation (psp) and community management of services backed by measures to strengthen local institutions and sustaining water and sanitation programmes. Development of an institutional capacity building policy for the entire water sector, Legislation will be reviewed and updated, Of major concern will be the legislation as regards transfer of water facilities from one institution to another.	Defined by the government Community participation in O+M: the Government will endeavour to hand over Urban Water Supplies and Sanitation facilities to autonomous departments within the Local Authorities and Rural Water Supplies to the communities.
South Africa	Before 1994 no national institution was responsible for ensuring equitable and sustainable access to water supply or sanitation services and no structured national legislation existed regulations the provision of these services. Now, the South African constitution guarantees everyone the right to access to sufficient water and states that one of the objectives of local government is to ensure the provision of services to communities in a sustainable manner. The act says that everyone has a right to basic water supply and places a duty on all water services institutions to take reasonable measures to realise this right.	Water services are a local government function, (ownership and O+M) The minister of water affairs and forestry sets national standards to ensure enough continuous, affordable, and equitable water services Schedule 4B identifies water and sanitation services limited to potable water supply systems and domestic waste water and sewerage disposal systems

	Water Sector Reform	Implementation of rwh-politics by
Tanzania	In 1999 the responsibility for O+M was being moved to local communities realized through water committees or water user associations (wuas). The policy requires that all villages with or without facilities should establish a water fund. For urban centres the government envisages facilitation of PSP Major parameters of national water policy are community participation, community-based management, cost sharing for rural, full cost recovery for urban, full ownership , involvement of the private sector	Water supply systems are owned by the central government, while O+M is run by local communities Communities are encouraged to organize themselves in managing their own water supplies in order to reduce the dependency on the government. The government is to promote the private sector and individuals to participate in the planning, construction, supply of materials and equipment as well as in the management of water supply schemes
Uganda	Decentralization program started in 1999. The 1999 national water policy's key directives are an integrated, sustainable management with full participation of all stakeholders based on management responsibility and ownership by all users within a decentralized. Water resources in rural areas belong to the users, while urban and sewerage systems remain with the central government	In contrast to the 1995 Water Statute all water sources in rural areas now belong to the users. For urban and sewerage systems, the system ownership remains entrusted to the Central Government.
Zambia	Since 1994 Local Councils are operating water schemes or in most of the cases now own the infrastructure by holding shares. PSP und inter-community cooperation is possible, local authorities may resolve to establish a utility (PPP, together with other communities, joint ventures with other local authority or several local authorities)	A local authority shall provide water supply and sanitation services to the area falling under its jurisdiction, expect in any area where a person provides such services solely for that person's own benefit or a utility or a service provider is providing such services. (IV.10.1) A utility or service provider may construct any facility within or outside its area (24.1)Everyone can apply to be a service provider, a service provider may construct any facility within or outside its area.

SUMMARY OF SURVEY RESULTS

A survey was conducted among water professionals and activists in all 10 countries. For the complete answers see Annex II.

- Small-scale traditional practices of drwh as a precondition for the technology's acceptance exist in nearly each of the reviewed countries.
- There is little tradition of community level initiatives taking up its own water affairs.
- Awareness creation is generally weak, activists are mainly non-governmental organisations and depend on donor-funds.
- Water professionals pay little attention to drwh.
- Training: there is often too little capacity in training or implementing drwh installations: in Uganda there were funds available but not enough expertise.
- Drwh has a project image, it seems to survive mostly in donor sponsored projects.
- The perception that the water sector organisation has an impact on the spread of drwh is very weak
- There is very little experience in legal matters; conscious battling through legal conflicts would depend on donor funds
- The potential of drwh is perceived as nearly unlimited both in urban and rural areas.

EMAIL-CONFERENCE

A summary of contributions to the GARNET email conference on Policy and Professional Attitudes on rainwater harvesting, held between 10 and 24.11.2000 reveals that a comprehensive framework for legal and policy issues regarding drwh is still missing. Meanwhile there are lots of - often isolated - experiences in the regulation of drwh. Their benefits need to be highlighted now with regard to mainstreaming drwh in other countries and regions.

CONCLUSIONS

There is still no professionalisation of drwh within the scope of national water management strategies. Drwh does not play a role here since it is simply not considered. Traditions in drwh remain informal and the best way of promoting drwh at the moment is still the neighbour's tank.
While water laws are surely not the key to promoting drwh, they can certainly hinder it. Although there is a growing demand for domestic roofwater harvesting on local level, the laws remain silent on this subject.

- **All water laws except the one in South Africa totally ignore rainwater harvesting**. Neither is it part of a national water management strategy, nor it is mentioned as a way in which unconnected and/or connected households could be provided/supplemented with water for their domestic use. In Namibia drwh even seems to be forbidden (according to our correspondence with Namwater, the countries bulk supplier).
- All water laws except in Ethiopia and South Africa treat **(rain-)water as a national property**, not as a major human need or even as a right.
- The private installation of "rainwater-works" (rainwater catchment systems) for domestic uses legally requires the **ownership of the land** plus an **extra permit** in those countries where rainwater (just like all other water resources) legally belong to the state: Kenya, Lesotho, Tanzania (and Ethiopia). Independent from the land-ownership a permit is required in Uganda. No permit is required in Ethiopia and South Africa.
- Major policy functions like the power to define **legal public water schemes** - as a major precondition for the inclusion of drwh in the national water supply strategy - remain vested in the central Ministry of Uganda and Tanzania and in regional authorities of Ethiopia. In South Africa public water services are limited to potable water.
- An **independent communal drwh-policy** would legally be possible only in Kenya.

RECOMMENDATIONS

A structual change of the water sector and its process of decision-making is necessary in order to achieve long term effects. If drwh shall be more than a vague option for the left-offs in water supply, there's a need for both bottom-up and top-down approaches

- National governments should consider the role of drwh and its contribution to the overall supply.
- For the **mainstreaming of drwh**, projects and activities should concentrate more on **institution-building processes** to gain sustainability.
- A designated water services provider should not be granted the right of having to use "his water" only for all possible uses.
- More flexible approaches in water legislation. The idea of one water source for all uses was devised in water rich countries and doesn't make sense in many water scarce countries. It doesn't follow present practices. Water professionals need to take this into account.
- The principle of reasonable and equitable utilisation of water as a customary rule that governs the legitimacy of uses proved to be successful at international level. This principle can also be used at local level. The flexibility is its real strength, because the principle requires that all factors be considered in the assessment of a reasonable and equitable use when contesting for water and thus deterring the legitimacy of new or increased uses.
- In a time of increasing private sector participation (rain-)water is to be considered not as a property but as a basic human right. An example of best practice regarding the legal status of water is SA.
- Private drwh should be made possible without extra permits and
- Independent from land-ownership. Best practice here is the roof-rendering in Bangalore.
- On community-level there is a need for building a strong force to manage local resources through networking with NGOs, donors, banks and water user associations (wuas).
- Local empowerment-activities are important to push the government to allow an independent community rwh-policy (implicits a chance in laws).

A Watershed-management-project was established in Maharashtra/ India aiming at qualifying local self-help-groups through an Indian NGO. The next step was the establishment of a network between sector specific institutions: NGOs, banks and donors with the target to enforce independent local watershed-management-institutions. The result was not only the sustainable development of the *watershed* but a fundamental change in water law: Since March 2000 there is a "common guideline for watershed development" regulating. Kochendoerfer-Lucius, G. & van de Sand, K, 2000.

Literature/ Water Laws

Gould, J. & Nissen-Petersen, E., Rainwater Catchment Systems for Domestic Supply: Design, Construction and Implementation, London, 1999
Kochendoerfer-Lucius, G. & van de Sand, K., Entwicklungshilfe auf den Kopf stellen. Institutionenfoerderung statt Projektfoerderung. In: Entwicklung + Zusammenarbeit 4, 2000. S. 96 – 99.
Koenig, K., Handbuch der Regenwassertechnik. Dortmund 2001

Botswana: The Water Act, Ch. 34:01, 1968.
Ethiopia: Proclamation No. 92/ 1994 – A Proclamation to provide for the utilization of water resources; Federal Water Resource Code of Ethiopia, 1998.
Kenya: The Water Act, Ch. 372, 1972; Sessional Paper No.1 of 1999 on National Policy on Water Resources Management and Development.
Lesotho: Water Resources Act No. 28 of 1978.
Namibia: Water Act, 1956.
South Africa: National Water Bill (B34B – 1998), water services Act No. 18522, Dec 1997.
Tanzania: Water Act Bill (1999), Water Utilization Act No. 42 of 1974.
Uganda: Water Statute No. 9 (Dec. 1995), Ministry of Water, Land and Environment: A national water policy (1999).
Zambia: Ministry of Energy and Water Development: National Water Policy, Nov. 1994; The Water Supply and Sanitation Act No. 28 of 1997.

ANNEX I:

The Water Laws and their Indirect Impact on Domestic Rainwater Harvesting

BOTSWANA: The Water Act, Ch. 34:01, 1968.

1) Is rainwater utilisation mentioned at all in the legislation ?	No
2) If yes, in which context? (Give §§ and cite)	-
3) Is the legislation - helpful for rainwater h.? - restricting the use of rainwater ? - indifferent ?	Legislation is rather unhelpful as far as rwh is concerned Because it is unclear whether rainwater legally belongs to public water. "water works" is defined as works constructed for or in connexion with the (..) storage(..) of public water, (..) or the conservation of rain water (I.2) As far as public water is concerned, Part II describes the ownership of and inherent right to the use of the water as follows: "there shall be no right of property in public water" (4). "any person may, without a water right, while he is at any place where he has lawful access to a public stream or to a natural lake, pan or swamp, take and use public water therein for the immediate purpose of (a) watering stock (b) drinking, washing, and cooking; or (c) use in a vehicle, but nothing in this section shall be construed as authorizing the construction of any works (II.5.) So, **the owner or occupier of any land may, without a water right** (b) construct any works thereon for the conservation of public water, and abstract and **use public water** so conserved, **for domestic purposes** (II.6.)
4) Are there any §§ which make rainwater utilisation impossible ? In which context ? Why ?	-
5) Are there any direct or indirect prohibitions of rainwater utilisation ? How do they argue ?	-

***ETHIOPIA**: Proclamation No. 92/ 1994 – A Proclamation to provide for the utilization of water resources; Federal Water Resource Code of Ethiopia, 1998.*

Ownership	All water resources are the collective property of the Ethiopian people
1) Is rainwater utilisation mentioned at all in the legislation ?	No
2) If yes, in which context? (Give §§ and cite)	-
3) Is the legislation - helpful for rainwater h.? - restricting the use of rainwater ? - indifferent ?	No permit shall be required for domestic use (21.1) In order to minimize and misuse of water for domestic uses the implementing organ may, as necessary, issue appropriate directives and restrictions. (21.2.) ("Implementing organ " means ministry as regards transboundary rivers and water bodies that flow across more than one regional government, and the concerned regional water bureau as regards regional water resources. (Art.2. definitions)) (21.2.) No municipality or town shall engage in the planning and development of water supplies from any water sources or in the construction of water works without a permit issued by the implementing organ (22.1.a)
4) Are there any §§ which make rainwater utilisation impossible ? In which context ? Why ?	(5.1.) 1998: subject to the exceptions stated in 21.1 and 27 of this code no person may (a) construct waterworks, or (b) withdraw water from a water resources, either for its own use or for supply to others, or may supply water, whether withdrawn by him from a water resource or received by him from another supplier 3.1. (1994) A permit issued by the appropriate authority shall be required to use water resources for the following purposes: f) municipal and urban water supply, j) any use requiring construction of water works (1998) No person may engage, for profit or otherwise, in the business of (a) the construction of waterworks (as defined as " ..)) or consultancy services related thereto without holding a license duly by the implementing organ in the consultation with the concerned professional organisation, if there are any.
5) Are there any direct or indirect prohibitions of rainwater utilisation ? How do they argue ?	

***KENYA**: The Water Act, Ch. 372, 1972; Sessional Paper No.1 of 1999 on National Policy on Water Resources Management and Development*

1) Is rainwater utilisation mentioned at all in the legislation ?	No.
2) If yes, in which context? (Give §§ and cite)	-
3) Is the legislation - helpful for rainwater h.? - restricting the use of rainwater ? - indifferent ?	The 1972 Water Act states the following: The water of every body of water under or upon any land is vested in the government, subject to any rights of users in respect thereof which, by or under this Act or any other written law, have been or are granted, or recognized as being vested, in any other person. (II.3.) The control of every body of water shall be exercised by the minister in accordance with this Act (4.II) The purposes for which a permit may be acquired are as follows – (a) domestic purpose, which expression means the provision of water for household and sanitary purposes and the watering and dipping of stock; (35, VIII) In all cases of proposed diversion, abstraction, obstruction, storage of water from a body of water other than those referred to in section 38 of this Act, application must be made in the manner prescribed by this Act for a permit for such diversion, abstraction, obstruction or storage of water from or in such body of water (36.1.VIII) (a permit is not required – (a) for the abstraction or use of water from any body of water for domestic purposes by any person having lawful access thereto, if such abstraction is made without the employment of works; (38.VIII) works means any structure, apparatus, contrivance, device or thing for carrying, conducting, providing or utilizing water, excepting hand utensils or such contrivances as may be specified by the Water Apportionment Board)) **Thus,** according to the Water Act all water is vested in the government, and for rwh for domestic purposes a permit is required.
4) Are there any §§ which make rainwater utilisation impossible ? In which context ? Why ?	
5) Are there any direct or indirect	

prohibitions of rainwater utilisation ? How do they argue ?	

LESOTHO: Water Resources Act No. 28 of 1978.

Ownership	5.1. the ownership of all water within Lesotho is vested in the Basotho Nation 5.2. the power to control and regulate the use of water shall be exercised by the Minister.
1) Is rainwater utilisation mentioned at all in the legislation ?	no
2) If yes, in which context? (Give §§ and cite)	-
3) Is the legislation - helpful for rainwater h.? - restricting the use of rainwater ? - indifferent ?	3.2. any person having lawful access to water may abstract and use such water for domestic purposes 4.6.whenever a water use requires the construction of water works that water officer shall grant the water use permit subject to the construction of such waterworks within the specified time under specified conditions
4) Are there any §§ which make rainwater utilisation impossible ? In which context ? Why ?	
5) Are there any direct or indirect prohibitions of rainwater utilisation ? How do they argue ?	

SOUTH AFRICA: Water Services Act No. 18522, Dec 1997.

1) Is rainwater utilisation mentioned at all in the legislation ?	No.
2) If yes, in which context? (Give §§ and cite)	-
3) Is the legislation	(3.1.) Everyone has a right to basic water supply and basic

- helpful for rainwater h.? - restricting the use of rainwater ? - indifferent ?	sanitation. (11.1) Every water services authority has a duty to all consumers or potential consumers in its area of jurisdiction to progressively ensure efficient, affordable, economical and sustainable access to water services. (11.2) This duty is subject to – (a) the availability of resources; (…) (e) the duty to conserve water resources. (11.3) In ensuring access to water services, a water services authority must take into account, among other factor – (a) alternative ways of providing access to water services. One of the limitations that applies to the right to basis water supply is that access to water services must be gained through the water services authority and that nobody may utilise water services from another source without the authorities approval. Exactly what basic water supply is will be prescribed by the Minister.
4) Are there any §§ which make rainwater utilisation impossible ? In which context ? Why ?	
5) Are there any direct or indirect prohibitions of rainwater utilisation ? How do they argue ?	(6.1.) no person may use water services from a source other than a water services provider nominated by the water services authority having jurisdiction in the area in question, without approval of that water services authority. (6.2.) A person who, at the commencement of this act, was using water services from a source other than one nominated by the relevant water services authority, may continue to do so – (a) for a period of 60 days after the relevant water services authority has requested the person to apply for approval; and (b) if the person complies with a request in terms of paragraph (a) within the 60 days period, until – (i) the application for approval is granted, after which the conditions of the approval will apply; or (ii) the expiry of a reasonable period determined by the water services authority, if the application for approval is refused. (8.1.) a water services authority whose approval is required in terms of sections 6 or 7 – (a) may not unreasonably withhold the approval; and (b) may give the approval subject to reasonable conditions.

SOUTH AFRICA: National Water Bill (B34B – 1998).

1) Is rainwater utilisation mentioned at all in the legislation ?	Yes
2) If yes, in which	Schedule 1.1.: A person may, subject to this Act – (a) take water

context? (Give §§ and cite)	for a reasonable domestic use in that person's household, directly from any water resource to which that person has lawful access (c) store and use run-off water from a roof
3) Is the legislation - helpful for rainwater h.? - restricting the use of rainwater ? - indifferent ?	(4.1.) A Person may use water in or from a water resource for purposes such as reasonable domestic use, domestic gardening, animal watering, fire fighting and recreational use, as set out in Schedule 1. (12.1.) As soon as is reasonably practicable, the Minister must prescribe a system for classifying water resources. (2) The system for classifying water resources may – (a) establish guidelines and procedures for determining different classes of water resources; (3.2.) the Minster is ultimately responsible to ensure that water is allocated equitably and used beneficially in the public interest, while promoting environmental values. (3.3.) The National Government, acting through the Minister, has the power to regulate the use, flow and control of all water in the Republic.
4) Are there any §§ which make rainwater utilisation impossible ? In which context ? Why ?	
5) Are there any direct or indirect prohibitions of rainwater utilisation ? How do they argue ?	

TANZANIA: Water Act Bill (1999), Water Utilization Act No. 42 of 1974.

1) Is rainwater utilisation mentioned at all in the legislation ?	No.
2) If yes, in which context? (Give §§ and cite)	-
3) Is the legislation - helpful for rainwater h.? - restricting the use of rainwater ? - indifferent ?	In Tanzania water is regulated through two broad legal frameworks namely: I) water resources management, governed by the **water utilization act No. 42 of 1974**, which relates to the granting of rights to the user of water and ii) water supply governed by the **Urban water act No. 7 of 1981**. the acts provides for the regulation of the water utilization in urban areas

	through the National Urban Water Authority. Other relevant legislation exist including international river basin treaties (?) The **National Water Policy** is based on the following development objectives: to identify and preserve water sources and catchment areas and to increase the population's health through the provision of safe and adequate water supply. Major parameters addressed in this policy are: community participation, community-based management, cost sharing for rural and full cost recovery for urban, full ownership of the projects, involvement of the Private sector, water resources. The policy aims at ensuring that all sources are protected and optimally utilized. In the past, the responsibilities of operation, maintenance, and administration of water schemes was vested in local councils. This is being moved to local communities. **Community-based management of schemes is either realized through water committees or wuas**. The policy requires that all villages with or without facilities should establish a water fund to demonstrate their willingness to sustain water facilities. Communities are encouraged to organize themselves in managing their own water supplies in order to reduce the dependency on the government, and subsequently consumer will take full control of their water supplies and ensure sustainability. The government is to promote the private sector and individuals (!) to participate in the planning, construction, supply of materials and equipment as well as in the management of water supply schemes.
4) Are there any §§ which make rainwater utilisation impossible ? In which context ? Why ?	
5) Are there any direct or indirect prohibitions of rainwater utilisation ? How do they argue ?	

UGANDA: Water Statute No. 9 (Dec. 1995), Ministry of Water, Land and Environment: A national water policy (1999).

1) Is rainwater utilisation	No

mentioned at all in the legislation ?	
2) If yes, in which context? (Give §§ and cite)	-
3) Is the legislation - helpful for rainwater h.? - restricting the use of rainwater ? - indifferent ?	The 1995 Water statute restricts the use of rainwater as follows: All right to investigate, control, protect and manage water in Uganda for any use, is vested in the Government and shall be exercised by the Minister and the Director in accordance with the provisions of this Part of the Schedule (II, I, 5). Notwithstanding any other law to the contrary, no person shall acquire or have a right to – (a) use water, (b) construct or operate any works. (II, I, 6) Subject to Section 8 a person may – (a) while temporarily at any place, or (b) being the occupier of or a resident to any land, where there is a natural source of water, use that water for domestic use, fighting fire or irrigating a subsistence garden. (II, I, 7; 1) In addition to that right to use water the occupier of land or resident on land may, with the approval of the authority responsible for the area, use any water under the land occupied by him or is resident on any land adjacent to that land. (II, I, 7, 2) The rights under subsections 1 and 2 do not per se authorise a person to construct any works. 8. The Minister may, in relation to any water source where the situation so requires, by notice published in a manner appropriate for the area as the Minister may see fit – (a) prescribe places from which water may be extracted for use, (b) prescribe the time and manner in which water may be used, (c) at times of shortage or anticipated shortage – (i) regulate water to be used for particular purposes, or (ii) regulate, restrict or prohibit the application of a water permit (..), (iii) on the advice of the water Policy Committee, declare any part of Uganda to be a controlled area and establish a comprehensive and integrated plan for managing land, water and other natural resources within the area, (d) temporarily or permanently prohibit the use of water from a given source on health grounds. (8, 2) No person shall extract water unless authorised under this Part of the Statute.
4) Are there any §§ which make rainwater utilisation impossible ? In which context ? Why ?	
5) Are there any direct or indirect prohibitions of rainwater utilisation ? How	3, 18,1: No person shall construct or operate any works unless authorised to do so under this Part of the Statute. (2) A person wishing to construct any works or to take and use water may apply to the Director in the prescribed form for a permit to do so. (5) the director may grant the permit under sub-section (4) on

do they argue ?	conditions that he may think fit and such conditions may – (a) require the payment of fees or charges that may be prescribed under this Statute – (6.32.1) The Minister may .. fix fees and charges for (b) the taking or use of water under a water permit granted under this Part of the Statute

ZAMBIA: Ministry of Energy and Water Development: National Water Policy, Nov. 1994; The water Supply and sanitation Act No. 28 of 1997.

1) Is rainwater utilisation mentioned at all in the legislation ?	No.
2) If yes, in which context? (Give §§ and cite)	-
3) Is the legislation - helpful for rainwater h.? - restricting the use of rainwater ? - indifferent ?	IV.10.1. notwithstanding any other law to the contrary and subject to the other provisions of this act, a local authority shall provide water supply and sanitation services to the area falling under its jurisdiction, except in any area where a person provides such services solely for that person's own benefit or a utility or a service provider is providing such services. IV. 12.1. Everyone Can apply to be a service provider.
4) Are there any §§ which make rainwater utilisation impossible ? In which context ? Why ?	
5) Are there any direct or indirect prohibitions of rainwater utilisation ? How do they argue ?	

ANNEX II:

The Survey – questions and answers

	I. People often know little about the possibilities of a decentral harvesting of rainwater (rwh), at least in countries without a tradition in rwh like India or a lot of smaller islands. ***- If there is some traditional rwh practises in your country, could you describe them in a few words?***
ETHIOPIA	Yes, in Botswana the practise that has been commonly adopted and used is the roof catchment and the threshold (ground catchment. The roof became common in use when other people begun construction using tin roofs. Otherwise from the thatched roofs, people are often have been discouraged by the quality (brownish colour etc) of the water harvested from the thatched roofs. Some have even preferred to just put their buckets or drum in the open space than along a thatched roof. The other method that has been commonly used in some areas of Botswana is the one in which the threshold is used as the catchment. In all of these methods, the storage facilities range from the use of buckets, drums and some ferrocement tanks. But in most cases people have found considered the tanks expensive and are therefore not willing to invest in them. Therefore their supply is usually of a very short term.
KENYA	Yes, there are many types of traditional RWH technologies in Kenya, difficult to describe without diagrams, but they include using clay pots and using natural depressions such as rock crevices.
LESOTHO	Well, a great deal of work has been done to equip school buildings and others with rwh systems. From my experience, however, the water is poorly managed and usually wasted. The episodic tendency of the weather leads to a condition where: you do not have water when you need it; you do not need water when you have it. There must be a much broader strategy implemented with respect to water, and rwh (→ Roof water harvesting, GN) is only one component. On our campus we have a well, 2 spring supplies, a stream supply, lots of tanks, 3 rwh systems, plus an elaborately designed landscape of earthworks, terraces, diversions, and stone retaining walls, and swales. Earlier this week we had very heavy rains, and the earthworks were working wonders. They are cheap, efficient, and should be installed prior to rwh. Roofs are only a small part of the potential catchment. (IY) Generally spoken, rwh is unknown and not practised. The only exceptions are the dams' structures, manly for irrigation purposes. (GN)

NAMIBIA	Yes, traditionally roof water harvesting is known and practised on a small scale throughout the country, where people actually try to conserve every drop of water they can get hold of, but it is illegal. All the water during a rainy season - when occurring - is supposed to recharge the aquifers (Grundwasserleiter) instead of evaporating in drums. Furthermore in the cities rainwater is supposed to reach the drainage of the sewage systems, because water is hold back that might cause damage due to too little water to flush the systems. In Windhoek for example we are the only city world-wide cleaning sewage to drinking water quality, therefore an diluting rain is usually very welcome, but therefore it is also illegal to do roof water harvesting.
SOUTH AFRICA	What would you regard as traditional? All the practices I have seen in South Africa emanate from having access to manufactured products. These products were often developed for other uses but used for RWH by innovative individuals. Most systems in South Africa are straightforward guttering leading to some form of surface mounted tank. In Namaqualand in the older houses the tank has been built underneath the house and is raised out using a simple pump.
ZAMBIA	No, there seems to be no extended tradition of rwh in Zambia, although, drums to collect rainwater are used by individuals in the southern part of the country where the rain season is much shorter then in the rest of Zambia. additionally farmers dig shallow wells (in southern province called "chikala") for domestic water supply. Shallow wells are the main source of domestic water supply between December and June/July. This supply is dependent on the water table fluctuations. The other common method is to collect rainwater directly during a rainstorm using a bucket or a drum.

	II. Once people are confronted with the idea of dwh, they usually start to show much interest. And if dwh is already realised locally, most of the people are very satisfied. Thus, rwh for domestic uses often seems a question of awareness. - ***Are there any actions taken in Lesotho to increase the use of and awareness for dwh (by government, development cooperation, ngos)? Could you describe their major achievements or bottlenecks ?***
BOTSWANA	Yes, recently there has been a lot of activities geared towards the promotion of rainwater harvesting in Botswana. My organisation has been pioneering in most of these activities. There are some government bodies which have

	also been involved in the practise of harvesting rainwater especially the local authorities at household level and the ministry of agriculture at agricultural level (livestock watering and irrigation). Well I will start the bad news first, yes there has been a lot of bottlenecks, obviously. Some of the tanks which were constructed some years back have now become white elephants simply because people have alternative piped water supplies. There is poor maintenance of rwh facilities with gutter falling down or the water left to rot for ages inside the tanks. In some villages which we visited people complained of the bad odour of the water after having stored for long. Some have even cited that tadpoles even develop inside the water tanks. In areas of very severe or less severe water scarcity the tanks that have been provided in schools and clinics are still being utilised fully. It's only that there was no involvement of end-users so most people do not know how to take care of the facilities to ensure that there are no contaminants into the water as result of foreign material going inside the tanks such as bird droppings, dead leaves etc Attempts have been made to formalise a Botswana Rainwater Harvesting Association, which will involve or bring together various practitioners in this field. The good news is that I think there is a possibility of getting government funding to support some of the rwh programs such as pilot projects etc. There needs to be countrywide campaigns to assist in the promotion of this practice.
KENYA	No. Awareness creation in Kenya is very weak, partly due to the costs involved and the thin number of active professionals in RWH. However, Awareness creation has been made by: Demonstration training sessions, usually by NGOs such as KRA, Mass media (mostly radio), Pamphlets and newsletters (mostly by NGOs)
LESOTHO	I think also the technical problems have not really been addressed. It is not simple to collect water off very large roofs. The guttering and piping has to be very carefully installed. On large roofs, the gutters often sag after a few years and start tearing off, and are not maintained. On a long roof the gutters are carrying a heavy load and have to be carefully set to gradient. In addition, they need reinforcing. For example, just last week we put up gutters and tanks on a new building. I used aluminium strapping to tie back the gutter brackets onto the roof. This adds enormous rigidity and strength to the installation. My office is above a roof water cistern on my own house, and getting the water around a big building is a thorny problem. I used a funnel set-up, buried pipe (40mm) to get the water around the back of the house, and this system works very efficiently. (IY) Awareness building will be a starting point, for rwh as any

	other appropriate technology. The technical problems should not be underestimated, neither the institutional requirements involving any innovation. (GN) I don't know of any Governmental initiative on rwh (but this does not mean that nothing is ongoing). In our Helvetas NRM programme (1997-2005), we have defined water management practices as one key component. Rwh is part of it. BBCDC is one partner in that programme and can be considered as a leading competence centre in water management in Lesotho, including rwh. The main achievements: 7 years of experiences on site based water harvesting techniques and training of hundreds of rural students. Establishment of the competence centre, various action-research activities, holistic approach. Bottlenecks: lack of professionals/committed people in action-research, lack of medium or long-term commitment by donors, weak institutions on NGO and Government side. We from Helvetas don't have the means to fully promote the concept as we would like. Our programme will be phased-out in 2005. (GN) Generally open earth dams have been since the fifties/earlier the main approach. But GoL with its water diversions encouraged loss of water rather than its collection. Lately, roof water harvesting has been popularised in drier south of Lesotho through the Soil + Water Conservation Division of government in collaboration with donor supported projects (MDP/GTZ; Swedeforest & PTC II projects now adopted by RSDA in the permaculture intervention). (LM)
NAMIBIA	.
SOUTH AFRICA	As far as I am aware only one NGO has tried to raise awareness around RWH. The problem with their approach is that they tried to provide in one go the full requirement. This proved too expensive for the householders. I think one of the manufactures of the galvanised tanks has given some support as in the Eastern Cape a number of small businesses developed assembling the tanks and transporting them to the houses. Most awareness has been created by a household successfully implementing rwh and neighbours copying.
ZAMBIA	1) We have no information on concrete actions on rwh in Zambia but have been informed that some officials have participated in regional conferences on this subject i.e. a workshop in Tanzania was attended by Zambians from MEWD, National Institute for Scientific and Industrial Research and from World Vision 2) A number of RWH activities have been initiated in Zambia through governmental and non-governmental organisations. An assessment of RWH activities in Zambia (Malesu, Phiri and Muzyamba, 1999) conducted in

	December 1999 revealed that the concept is relatively new and practised by a handful of households and institutions (schools and clinics). **A training of trainers course in Rainwater Harvesting was conducted between 10th April and 28th May 2000. The Regional Land Management Unit (RELMA) based in Kenya in conjunction with MAFF/LM&CF sponsored this training. Thirteen (13) participants were drawn from southern, eastern and Lusaka provinces. The training took place in Choma and four demonstrations were set up at farm level. The main focus of this training was on imparting practical skills in designing and construction of RWH structures.** Follow up on the trainees to this course has revealed that the knowledge gained is used in different ways. Most interesting is one case in Southern province where an officer has used the knowledge gained to construct sinking wells for farmers at a fee. The officer has been motivated to carry on practicing rainwater harvesting to supplement his income. One trainee in the Eastern province mobilised a community interested in constructing a roof catchment structure and asked them to contribute bricks, sand and stones. The community is currently scouting for funds to purchase cement and start construction of the tank. Most trainees have sat down waiting for the Project Support Office to provide funds for them to implement rainwater harvesting. Formation of the Zambia Rainwater Harvesting Association (ZARHA) The LM&CF programme facilitated the formation of ZARHA towards the end of November 2000. In view of the training in RWH conducted by the programme it was important to form an association that would fully facilitate the implementation of RWH activities in Zambia. The second national workshop was sponsored by RELMA and it was at this workshop that an executive committee was put in place. ZARHA will utilise the capacity built to enhance RWH activities in Zambia. Three ZARHA members, namely Mrs. Glenda Mulenga Kasuba, Mr. Bob Muzyamba and Mr. Maimbo Malesu were invited to attend the Southern and Eastern Africa Rainwater harvesting Network (SEARNET) conference held between 10 and 14 December 2000 in Nazareth, Ethiopia. The conference was a forum for exchange of information and experiences as well as planning for country activities. Zambia was proposed as the venue for the next conferences in December 2001. Constraints • *Inadequate co-ordination among co-operating institutions,* • *Inadequate training in RWH* • *Inadequate of government policy on RWH* • *Poverty*

	• *High cost of construction materials* • *Inadequate community organisation*

	III. Full rwh often seems too expensive for the poorer half of the population except under the most favourable climatic conditions. So either partial rwh can be used, or it s introduction is part of an aid package, or it is accepted to be the means by which (richer ?) households increase their water security. - ***Could you shortly describe the attitudes of water professionals on (multiple sourcing of water and therefore of) partial rwh ?***
BOTSWANA	There has been only a few believers in the practice of rainwater harvesting. However, other professionals have seen the need to revisit this practice which was done by our ancestors. It is not a new practice. Therefore, with the move towards being a water conservative society most professionals cannot ignore the fact that rainwater harvesting is a very powerful water conservative tool given the country's finite water resources. This would be useful for both urban and rural Botswana. Most of our rural population rely on borehole water which is a very unreliable resource taking into account that the recharge rates of groundwater are very low in most parts of the Botswana. Some rural areas have very saline water from the boreholes which is even unsuitable for livestock watering too. With the new Water Conservation Policy there has been some aspects of rainwater harvesting included in it. Other professionals need a push or need to seed something tangible out of the benefits of rainwater harvesting. Sometimes it is difficult to promote something unless people see and start getting affected by a problem that is how most of us tend to learn.
KENYA	Traditionally, the attitudes in Kenya have been that water is supplied by the Government. However, due to the failure of the Government to meet these expectations, communities have learnt to either form groups through which they work collectively at RWH projects or seek donor funding (mostly cost-sharing). The latter option has been more successful. But even then, only a very small proportion of the potential households in Kenya that could benefit from RWH has been lucky to benefit.
LESOTHO	The earth dams promoted by the soil/water conservation division (MOA) is hidden under their objective of soil and water conservation. (LM) For example, the Governmental Department dealing with rural water supply (DRWS)- a long- term partner of Helvetas- has not yet started with

	including rwh into the technical packages. Rwh and a more holistic water management concept are important topics to be included in its future business. So generally, I think rwh are more or less unknown and/or not systematically promoted up to now. And that in a country known for its regular water shortages/droughts! (GN)
NAMIBIA	.
SOUTH AFRICA	Most professionals in South Africa totally ignore rwh and concentrate on unsustainable piped schemes, including desalination plants. Very few look at using different sources of water for different purposes as the communities are doing at present. Although only the rich can afford custom installations because of the large manufacturing base in SA and the mobility of the population, it is relatively easy to obtain old containers. For example in the Eastern Cape a 200 l chemical drum costs R80. Before it is used it is cleaned using boiling water to get rid of the chemical residue. Using a number of these containers significant storage can be built up over a period without incurring huge expense. **- What are in your opinion the strongest arguments for rwh in South Africa ?** the fact that we have a well developed manufacturing base, good distribution but large numbers of rural people without access to sufficient water supply is a compelling case for promoting rwh. Coupled with water conservation and re-use of water rwh could quickly raise the standard of living of many people without incurring vast O&M costs.
ZAMBIA	Water professional in Zambia seem not to pay much attention to rwh

	IV. How do you estimate the influences of good legislation and a decentrally organized water sector on the promotion of rwh?
BOTSWANA	This question is not very clear to me. Could you kindly try and explain this one to me
KENYA	1. Good legislation is very important, as until recently, the water Act was silent on RWH. Even now, this needs strengthening. 2. Kenya is in the process of decentralising the water sector and letting communities manage their water. This is in the right direction, so long as water resources are looked at from a hydrological point of view. One advantage in using RWH is that there are few conflicts of hydrological nature, as RWH systems are mainly household based. This strength should be highlighted.

LESOTHO	Legislation is absolutely irrelevant. Dissemination of current best practices is what is required. (IY) Legislation focussing on rwh is irrelevant at this stage. We need to look at it in the context of water affairs whose legislation is available but rather unhelpful in our case. (LM) Legislation is just one element. It should reflect the real intentions and commitments of the stakeholders. Legislation alone is irrelevant, if no institutions and people are behind it. (GN)
NAMIBIA	.
SOUTH AFRICA	legislation is not the key to promoting rwh. SA law has no problem with domestic rwh but at the moment there is a huge debate over free water. This will have a bigger effect, especially on poor communities as they will reason why invest in rwh when the government has promised the water for free.
ZAMBIA	1) The new water policy and the activities of the new Regulator NWASCO will have an effect on the operation of the provider of water and sanitation services. One of the main issues is the reduction of unaccounted for water, which means the reduction of wastage by the consumers. The Regulator will push for 100% metering and billing according to consumption implying that consumers will not have the choice to water their vegetable garden or lawn with tap water unless there are prepared to pay a cost covering price, and possibly a subsidies for the consumption of the poor. This might have an impact on the need to harvest rainwater 2) The decentralised water sector still favours supply of good quality water to the urban areas and less to the rural areas where clean water is an issue. The private sector is only interested in supplying water to the consumers at a cost (which is more sustainable). Water providers in the rural areas concentrate on drilling boreholes and constructing earth dams for communal use. These options are expensive to implement and maintain. RWH has not been clearly defined in the current water policy as an option for water supply. However, the current policy presents an opportunity for ZARHA to lobby for the strengthening of RWH policy in Zambia.

	V. Is the current national water-legislation helpful for rwh or rather restricting it ? in practice, how relevant is it ?
BOTSWANA	Besides the Water Act, there is also the Waterworks Area Act which defines areas that have been declared as

	catchment areas for some of the dams here. Therefore if there is any activity going on within this area the government is bound to intervene especially that involving surface water. There is apparently not much restriction in using any borehole water within your own plot, and again I think that there is no borehole water act! One thing that I have observed also is the clashing of water development guidelines especially that of the department of water affairs and the ministry of agriculture. What is happening is that, a syndicate of farmers after having acquired a land right can approach the ministry of agriculture and request to be built a small earth dam which they could either use for livestock watering and/or irrigation. However, before this can be done the department of water affairs would have to be contacted to ensure that this dam is within designated catchment areas for bigger dams. The farmers have to pay 10% of the total cost for this dam. I do know if I am actually answering your question or have gone off tangent. Otherwise water legally belongs to the government, but within you own plot you can sink a borehole depending on whether there are no alternative piped supplies nearby especially in areas that are not serviced or very remote areas where you have been allocated a farm or whatever.
KENYA	There has been a more recent water policy (1990), which mentions RWH. This is better than nothing, but more legislation is necessary to include emerging technologies and modern thinking. I will need to do more research on the legal meaning of the Water Act (1972) -as I'm not a lawyer to understand it fully. However, to answer your questions on the other aspects: 1. A permit is not required for RWH, so long as no pumping device is used. However, if a pump is used, a permit is required. Very confusing. For instance, if two farmers make RWH systems of similar capacity say 150 cum., one has tanks above the ground and therefore doesn't need a pump, while the other lets the water flow into a dam where he pumps it up. The latter requires a permit.
LESOTHO	It is overly negative. Lesotho allows water to blow out of it at a massive rate each rain event. The opportunity to reduce this without depriving anyone else of water, is virtually infinite. Witness current flooding in Mozambique. Government does a poor job of innovating or implementing; it should mediate and disseminate, and outside that, interfere as little as possible. (IY) I see no impact (see above). (GN) - In practice, how relevant is this for rwh? Irrelevant. (IY, GN)
NAMIBIA	.
SOUTH AFRICA	None
ZAMBIA	6) The Act 28 gives exclusivity of service provision to the providers and the right for individuals to generate these

	services for themselves, but does not exclude the delegation of these services by the providers to third parties. The Regulator will put this in practice as we can see from his policy.

	VI. On local level, communities have the possibility to raise their own regulations and bylaws. ***- Do you know cases where communities took use of their right to develop their own rainwater policy ? what where the results?***
BOTSWANA	No. unfortunately there has not been something like that here. And like I have indicated most communities are of a very marginal status to be able to afford bigger tanks or facilities for harvesting rainwater at household level. But where there are community structures built for rainwater harvesting especially for livestock watering, The community seems not eager to carry their responsibilities of maintaining the structures. After completion of the structure it is handed over to the community for operation and maintenance. I think that probably this may be due to lack of operation guidelines.
KENYA	Many of the RWH projects in Kenya have been achieved through community efforts, and they have been successful (Govt projects usually involve river developments).
LESOTHO	Our campus has a very elaborate reticulated multiple source water supply system. There has never been any clash between local or national water regulation, and no one has had to concern themselves. Our efforts are making more water available to the general community, and not less. Following recent rains, the level of water in our 5 meter deep well, rose by 3 meters...all due to careful landscape design. (IY) With regard to earth dams, ownership can be of 2 categories (even on public land) – individual or community. I am not sure how one can own a dam privately on public land. (LM)
NAMIBIA	
SOUTH AFRICA	No, don't know of any council which has taken this step. just one clarification only Local government can promulgate bylaws, it is not a national competency.
ZAMBIA	No. We have no knowledge of communities developing a rwh-policy, but as the councils are operating the water schemes or in most of the cases now own the infrastructure by holding shares in a CU, I can see no obstacle for them to develop such a policy if it is of benefit to their community.

	VII. Where do you see the best chances for rwh in Lesotho and why: in urban, peri-urban or rural areas ?
BOTSWANA	Both. the urban and peri-urban areas together with the rural areas have an equal chance into rwh. The water tariffs in the urban are quite high because there has been a lot of expenditures towards the development of the water infrastructure involving a more 400km water pipeline from north of the country down south to the capital city Gaborone. This is reason for increased water tariffs. Now, really I think that society can move from using clean piped water for gardening or washing cars or other external uses and instead harvest rainwater and use it for those activities to reduce strain on the already existing water supply resources . As mentioned earlier, there are some rural areas in which there can be an absence of water for more a two days or so. This at times may be due to a breakdown inside the borehole, or that the water level has gone much lower and so on. Now with rainwater as an alternative resource this can reduce the impact of these kind of incidents on the communities affected. Rainwater harvesting also can reduce the impact of drought on especially the most vulnerable communities. The health/socio-economic status of people in rural areas who are very vulnerable because most are unemployed can improve as they eat healthier food which they could have grown out of rainwater or even use the money for income generation.
KENYA	RWH in urban and peer-urban areas can be successful among the wealthy and middle class. This is because they live in spacious compounds where RWH can be practised. The poor live in slums, where RWH infrastructure is either not practical or there are dangers of pollution.
LESOTHO	Virtually unlimited potential for exploitation. Earlier this week, small stream beside our campus was estimated to be flowing at a rate of 20 cubic meters / second during rainstorm......Water scarcity is a myth.... poor management is a fact. (IY) Urban and peri-urban areas are excellent for roofwater harvesting for obvious reasons; while the general rwh as put to us in this questionnaire has unlimited potential in rural communities. (LM) rwh can be multiplied faster in urban and peri-urban areas because of the better conditions for awareness creation and service delivery. But the chances for improved rwh in general are very good anywhere. It’s just a question of organisation (commitment, communication, project management, fund raising). Helvetas would be keen to do more, but time and money are running out…(GN)

NAMIBIA	
SOUTH AFRICA	In rural areas where there the possibilities of getting communal water systems to household level remain remote. However in urban areas this might change as water becomes more scarce in SA and the marginal cost rises making it more attractive to recycle grey water for irrigation/flushing of toilets and using rwh to reduce demand from the municipal system
ZAMBIA	1) The best changes to introduce rwh in Zambia a probably in the south of the country in rural areas, in low income areas where a network or Kiosks do not exist and in urban areas where big plots are maintained (gardens and lawns) 2) The promotion of RWH has the best chance of success in the rural and peri-urban areas of the country. Dwellers in these areas are faced with serious water shortages especially during the dry season. In the rural areas people have to walk long distances to access relatively clean drinking water whereas the peri-urban communities are also faced with problems of contaminated drinking water caused by poor sanitation and high population density.

DTU

Ram Pump Programme

Ram pump system design notes

Suitable areas for a ram pump system

Although all watercourses slope downwards to some degree, the gradient of many is so shallow that many kilometres of feed pipe or canal would be needed to obtain a fall of water large enough to power a ram pump. Ram pumps can be made to run with drive heads of less than one metre but they are not normally considered viable unless heads of two metres or more are available. If it would take a long length of feed pipe or canal to achieve this head, a ram pump system would be prohibitively expensive. The best geographical area for ram pumps is one which is hilly, with rapidly dropping watercourses and, ideally, springs.

In some areas of the world good regional records of rainfall and flow from springs and in watercourses are kept in government offices and libraries. In others, another agency may have carried out recent relevant studies. If any hydrological studies are available for the region in which you plan to install ram pump systems, you can save time, effort and costly mistakes by consulting the records and using their findings in your site design.

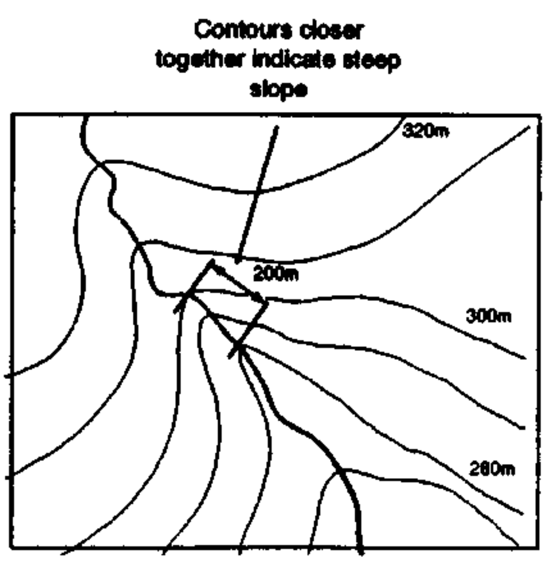
After potential sites have been identified, they must be surveyed. The survey yields information about site dimensions and the materials required to construct the site as well as, when more than one site is surveyed, yielding a cost and performance comparison.

Designing a good drive and pump layout is crucial to achieving good system performance and limiting the amount of maintenance required. The aim is to be able to achieve a large head of water between the drive tank and pump, while using a short drive pipe to connect them. The best and cheapest sites are those where the land falls rapidly, allowing all pipework to be short.

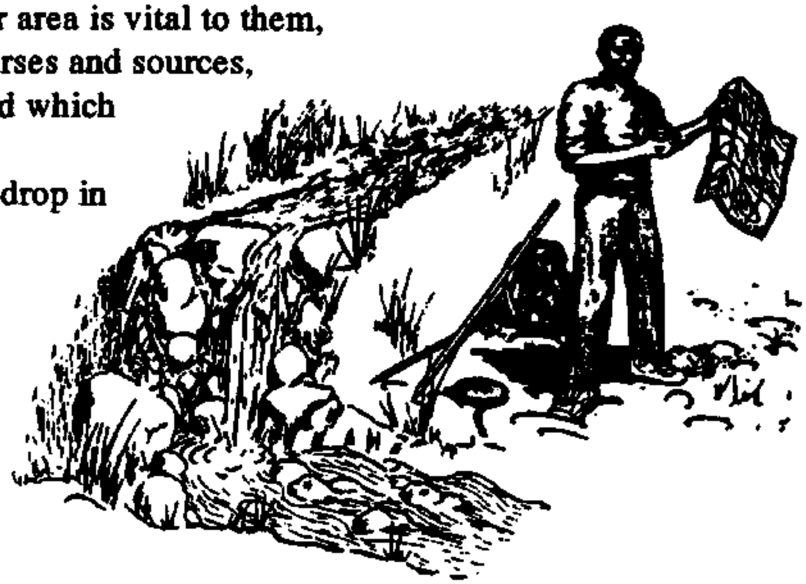
What to look for

Before arriving at a community to survey for a ram pump system, consult any available large-scale contour maps of the area. Look for places near to the community where streams cross contour lines and measure the distance along the stream between two contours. The average stream gradient can be obtained by dividing the height change between contours by this distance. An average gradient of at least 0.02 (0.02 or 2% is the same as 1 in 50) is needed for a water supply ram pump system. The pump will often be installed alongside rapids or a waterfall. If stream flow data for the area is available, look for seasonal variations, particularly during the dry months, and also for any data on the increase of flow during floods.

The section of stream shown in the map alongside falls quite rapidly. Where the contours are closest together on the stream's course, there is a drop of 20m over a distance of 200m, giving an average slope on that section of 10%. This site would be worth looking at.



People living in a community are frequently the best source of reliable, up-to-date information about that particular area. The water in their area is vital to them, so ask them about local watercourses and sources, which ones they prefer to use, and which ones regularly dry up.

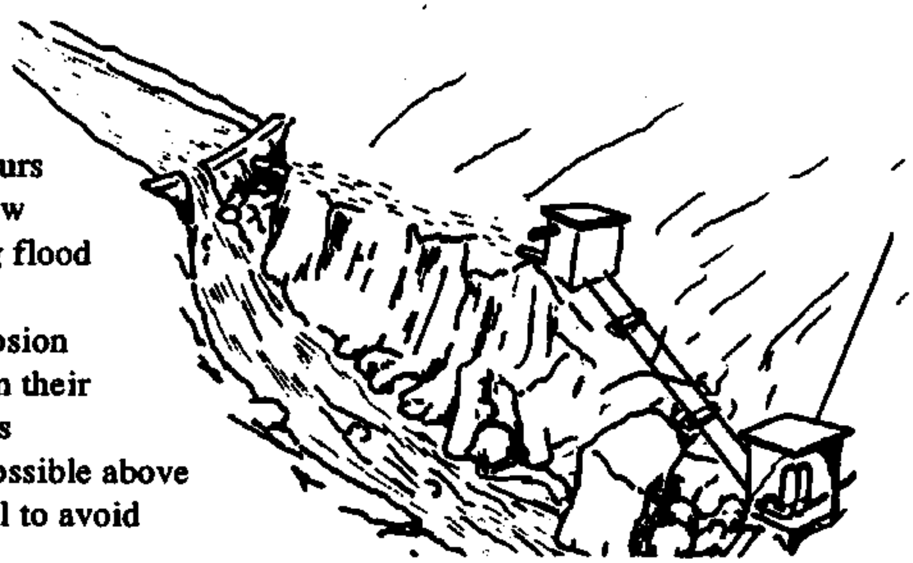


Sound is a good indicator of a drop in water. Where a watercourse is noisy and turbulent it will be dropping rapidly. It can save surveyors a lot of time to ask local people to show them places where the water is noisy, where it falls rapidly, and where there are rocks in the stream bed. Also ask to be shown any springs in the vicinity, particularly any with a source at the side of a valley along which a stream flows.

Allowing for floods

It is important to find out how the watercourse changes during a flood. Although spring-based systems are less susceptible to the extremes of flooding, it is still worth investigating and being aware of flooding as a potential problem when using a spring.

In very heavy rainfall, watercourses can swell to many times their normal flow within a short space of time. Design of a drive system must take account of these worst weather conditions, even if they only last a few hours each year. Ask local people how far the water rises during a big flood and look for evidence such as deposited rocks and stream erosion on the banks to help to confirm their reports. Keep the pumps and as much of the drive system as possible above the maximum flood water level to avoid unnecessary damage.



In the system sketched here, the main flow of flood water and debris is guided past the pumps by the large boulder or outcrop. Whilst the pumps may be submerged in a very high flood they will be protected from damage by most water-borne debris.

The one part of the system that will always be in the path of a flood is the dam or diversion structure. Bearing the expense in mind, this must either be built to withstand the worst flood conditions or built to be easily repairable. Some protection against the worst damage to a dam should be included in the design, including a cage protection for the system intake, a floodwater overflow in the dam wall and boulders anchored into the stream bed above the dam to reduce the impetus of rolling stones.

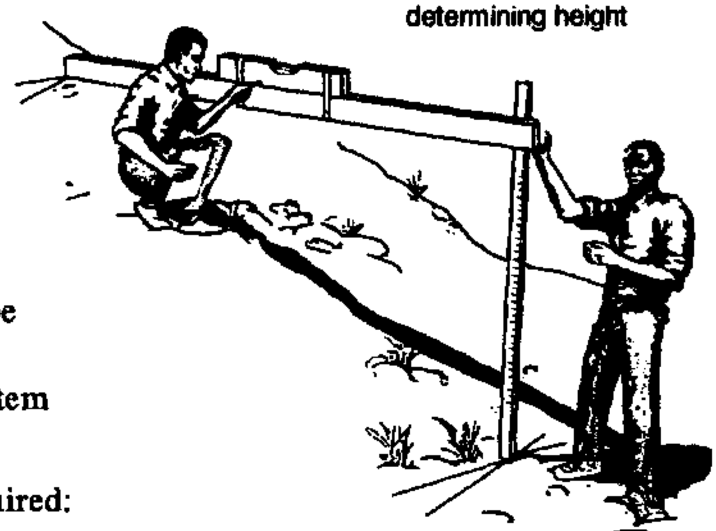
Normally the pump(s) should be sited above floodwater level, but in some cases (particularly where the drive head available is marginal) this can be impractical. In these circumstances the pumps need to be sited so that they are protected from any debris in the flood, or so that they can be easily removed when a flood is expected.

Site surveying

Surveying should be in two stages, a preliminary assessment to locate potential sites, and a full survey to select the best option. In brief, the *preliminary survey* will involve the following:

- consulting any relevant maps and hydrology studies;
- travelling to potential sites to make a preliminary assessment and consult local people;
- selecting one or more sites with a high potential that will be surveyed fully.

Simple method of determining height



The *full survey* of selected sites must be sufficiently detailed to provide all the information necessary for accurate system design.

The following information will be required:

Water source flow This figure must reflect the minimum flow of the water source in a normal year and so the measurements must be taken during the dry season. In order to ensure that the figure represents a typical minimum, local people should be consulted about the flow patterns over recent years. Some indication of maximum expected flows should also be obtained.

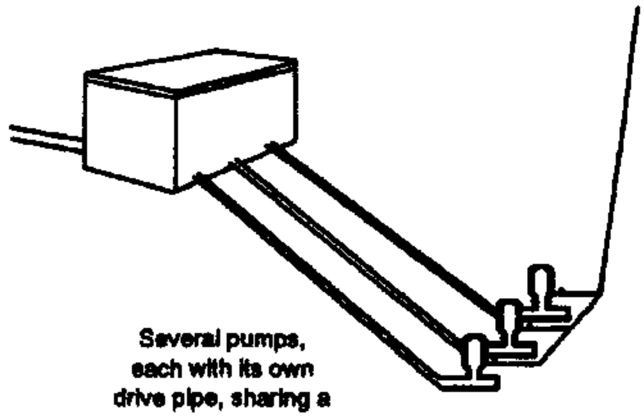
Drive head The maximum possible drive head of the site or the average angle of the slope below the water source should be measured.

Delivery head The height from the water source to the expected point of delivery should be measured. This will provide a rough estimate of delivery head but to be accurate the drive head should be added to this figure to give the actual delivery head.

Water requirement This should be estimated with reference to the population to be served (allowing for growth) or the area of land to be irrigated.

Pump selection

When selecting a ram pump there are often a number of decisions to be made concerning both its type and size. To make these decisions wisely, it is helpful to have some information about the site where the pump will be used. An estimation of the available drive flow and delivery head are needed, along with the anticipated delivery flow requirement. Knowing these will help to ensure that the correct pump is chosen. The main decisions to be made are:



Several pumps, each with its own drive pipe, sharing a common drive tank

- whether to buy an imported pump or to manufacture one locally;
- the size and number of pumps to be used.

Pumps with drive pipes of between ¾" and 4" are often commercially available or can be imported fairly quickly from manufacturers in Europe, N. America, etc. Larger models, with up to 12" drive pipes are also obtainable but usually have to be manufactured to order. Choosing between pumps from different manufacturers is not always straightforward. Comparisons of flow and efficiency can be made if the data is available. Cost, however, is normally very important, and varies considerably from pump to pump and manufacturer to manufacturer. The need to import spare parts and their cost should also be carefully considered. Where the source flow is large the choice is often between using one large pump or a number of smaller ones to meet the required delivery. Each model of ram pump with its recommended drive pipe will operate over a specified range of drive flows. In general, pumps are more efficient towards the lower limit of their drive flow, but produce a greater power output with large flows. A number of pumps can be used in parallel sharing a single drive tank and delivery pipe, but they must have separate drive pipes.

Using several small pumps rather than one large one can have the following advantages:

- each pump can be set for high efficiency;
- if the source flow falls below the required drive flow one pump can be stopped, allowing a reduced delivery flow to be maintained;
- a single pump can be stopped for maintenance work to be carried out without stopping all delivery flow;
- smaller sizes of pipe are often more readily available, whereas large steel pipe may have to be imported at high cost: it is also harder to work with large diameter pipe and to transport it;
- the maximum delivery head is generally higher on smaller pumps.

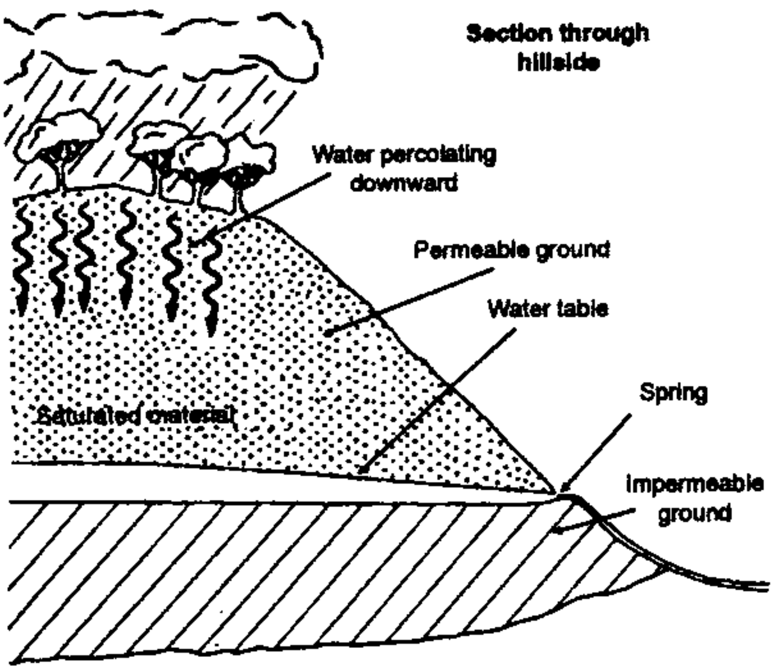
The cost of using a number of smaller pumps may be greater than using one large one but the benefits mentioned above need to be balanced against the extra cost.

Intake design

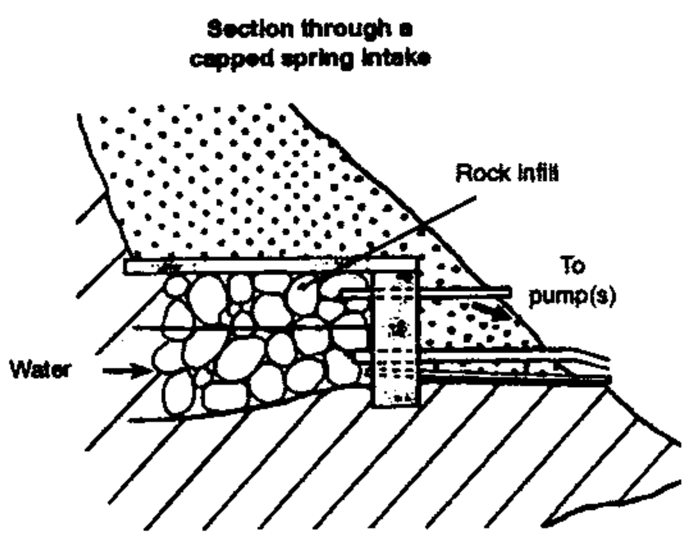
The drive water for ram pump systems is normally drawn from a spring or a stream and diverted into a feed pipe. An intake structure is required at the chosen water source to collect the water and keep the feed pipe properly supplied.

Spring intakes

When rain falls on high ground, some of the water soaks into the ground and seeps down through the permeable soil until it reaches an impermeable layer — usually clay or rock. The water collects on this layer, forming a water table of saturated material. The term water table refers to the upper level of the saturated material, and may rise and fall to reflect variations in rainfall throughout the year. As the level of the water table rises, the water spreads out until it reaches the side of the hill, then flows out as a spring.



The nature of the permeable layer influences the kind of spring that occurs. If it is soft, sandy soil through which the water can travel easily, there may be many small springs as the water emerges all along the hillside. If the permeable layer is rock, with cracks through which the water sinks, there is usually one distinct spring.



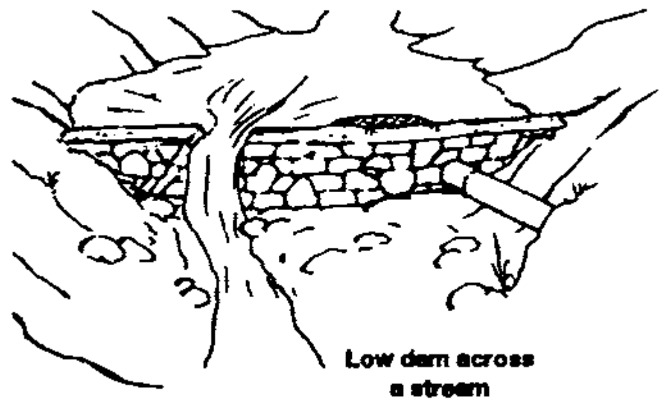
In most situations water emerging from springs is completely pure and safe for drinking, whereas by the time it flows into a stream or river it has often become contaminated. This is why it is preferable for a ram pump system supplying drinking water to be served directly by a spring.

Spring flows are often quite low and in many cases it is necessary to combine the flow from several springs to obtain enough drive flow for a ram pump system. When this is done, the drive head available to the pump(s) is the drop from the lowest of the springs to the pump(s).

Seasonal variations in spring flow must be allowed for. In most cases the users require the same amount of water at all times through the year, so the system must be designed to use the amount of flow available when the springs are at their lowest. If this is not done, there may be times during the year when there is not enough water delivered, or when there is too little water to run the pump(s) at all.

Stream intakes — dams and weirs

On streams and small rivers the feed pipe must draw water from a relatively deep pool to ensure that the end of the pipe remains below water level. To achieve this it is usual to build an obstruction across the stream to raise its level, and to run the feed pipe through it. The obstruction can either be a dam with a specific overflow point (a spillway), or a weir where the water flows over it across its whole length.


Low dam across a stream

The types of dam or weir normally constructed are relatively small and simple as the increase in level needs only to be around 300mm. In situations where the drive head must be increased significantly in order to make a system viable, larger dams may be necessary. In many parts of the world, simple, cheap designs of dam have been developed that suit the local conditions and use readily available materials.

Siting the intake The best site for a dam or weir is where the gradient of the stream is large, such as just above an area of rapids or a waterfall. This enables a high drive head to be obtained within a small distance and so helps to reduce the length of feed and drive pipes. In situations where the gradient of the stream is fairly constant, constraints on siting other components of the system may determine the location of the intake. On streams with a low gradient there are two ways of obtaining a suitable drive head. One is to build a small, cheap dam some distance upstream and run a long and therefore expensive feed pipe to the drive tank. The other is to build a much taller, more expensive and complex dam and run a short length of feed pipe to the drive tank. Siting the dam in a narrow section of stream can greatly reduce the amount of material required in construction, particularly where a taller dam is to be built.

When choosing a site, careful attention must be given to the foundation on which the dam structure will be built. Solid rock foundations ensure a firm base but require the use of cement or concrete to attach the structure securely. If building on loose material, such as soil or gravel, it is necessary to dig a trench across the stream bed to provide a firm anchor for the structure. The foundation should extend well into the banks on either side of the stream, especially where these are soft.

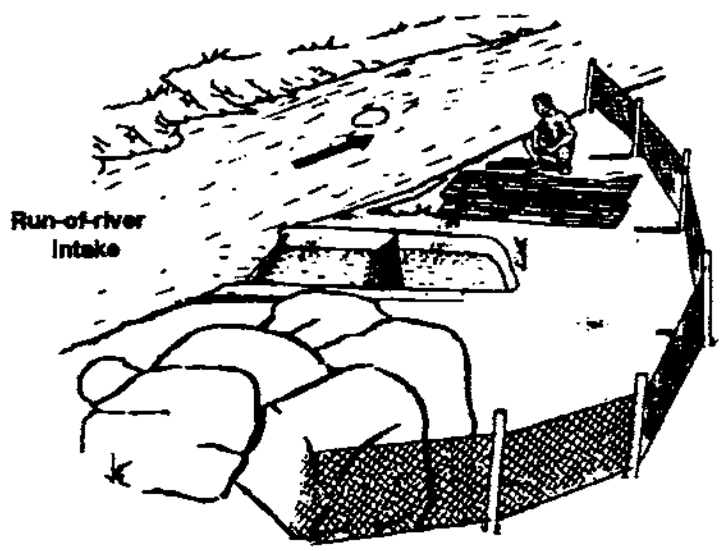
The size of expected floods and the type of debris carried by the floodwater also affects the type and site of the intake structure. A low, well-anchored dam or weir can be designed to allow floodwater to pass straight over it so that, apart from the additional build-up of silt and other debris behind it, the structure is little affected by floods. Larger dams should be built strongly enough to withstand flows much higher than the normal operating levels expected. A spillway will normally take excess water over the dam, but can rarely be large enough to cope with floodwater levels. In some cases a bypass can be built for floodwater but care should be taken to prevent the stream from establishing a new course along the bypass route after the flood.

Components of an intake A number of features should be built into any dam or weir intake:

- Feed pipe socket — a socket built into the structure to enable the end of the feed pipe to be disconnected in case repair or cleaning is required;
- Screen — a coarse wire mesh covering the inlet of the feed pipe to prevent larger debris such as leaves and sticks from entering the system with the drive water;
- De-silting pipe — a small bore steel pipe built into the base of the dam or weir to allow periodic or continuous removal of silt from behind the structure. Silt must be removed before it reaches the level of the feed pipe and starts to enter the system.

Run-of-river intakes

Where the flow of a watercourse is too great to allow construction of a suitable dam or weir, an intake can be built into one of the banks. Such a run-of-river intake diverts a portion of the water from the main flow of the watercourse and channels it into the feed pipe. The intake must be positioned at a site where there is fairly deep water near to the bank. It is also necessary to design and site it so as to prevent excessive silting.



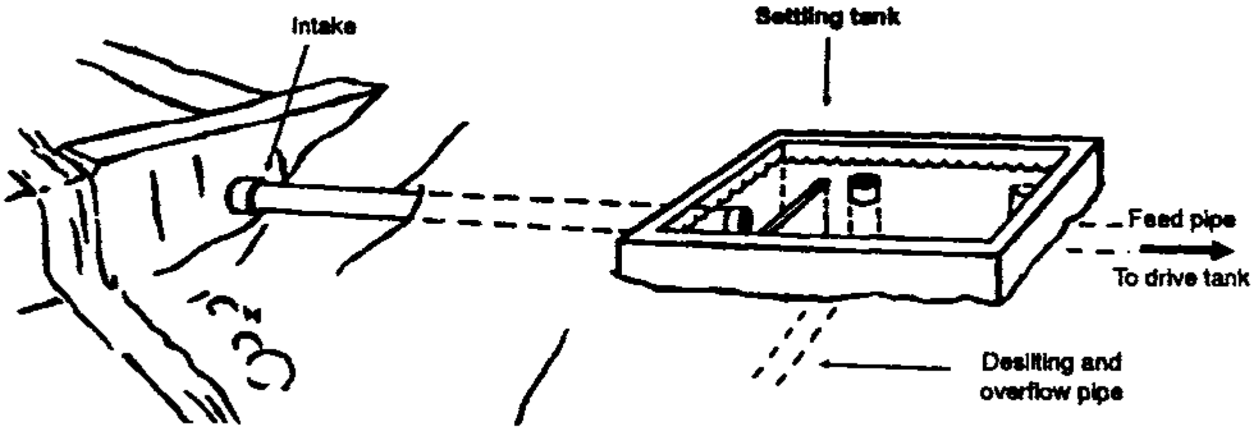
The spiralling flow of water on a bend tends to cause sedimentation on the inside of the curve, so intakes should never be sited in such a position. Siting an intake on the outside of a bend can be advantageous because the water tends to be deeper and silting reduced, but this position can leave it liable to damage by floating debris, especially during floods.

Section through river on a bend

Settling tank

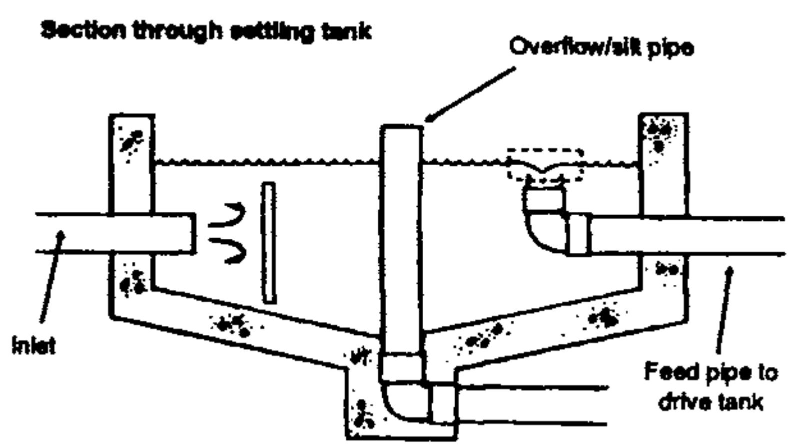
Water, particularly in streams and rivers, can carry a large amount of small pieces of soil and sand — often referred to as silt. The amount of debris carried by water depends upon its velocity and the type of ground over which it has travelled. Silt can cause damage by eroding pipework and pumps as it is forced along by moving water, or by settling and building up to create blockages. When it is planned to use water that has a large silt content for a ram pump system, it is wise to include in the system a means of removing the worst of this debris.

When the velocity of flowing water is reduced, the particles of silt being carried tend to fall slowly and settle out. In some systems, the large pool of water behind a dam or in a spring box can slow down the water sufficiently to allow silt to settle out. A large drive tank will also reduce the velocity of the water sufficiently to allow some settling to occur.



In situations where the source has a particularly high silt content and no large pool of water at the intake, a purpose-built settling tank should be included in the system. The settling tank should be designed to reduce the velocity of the water sufficiently to allow much of the silt to drop to the bottom of the tank rather than being carried with the water further into the feed pipe. The most convenient place to site a settling tank is near to the intake structure. If the system uses several springs, the settling tank can also be the most convenient place to combine the flows from each of the springs.

Settling tanks need regular maintenance. If they are well designed, a large amount of silt will collect very quickly and the tank will need cleaning. A removable overflow that serves as a silt pipe installed as shown can make cleaning fairly simple. Withdrawing the overflow pipe from the elbow in the base of the tank causes a large flow down the silt pipe, removing settled debris from the base of the tank.

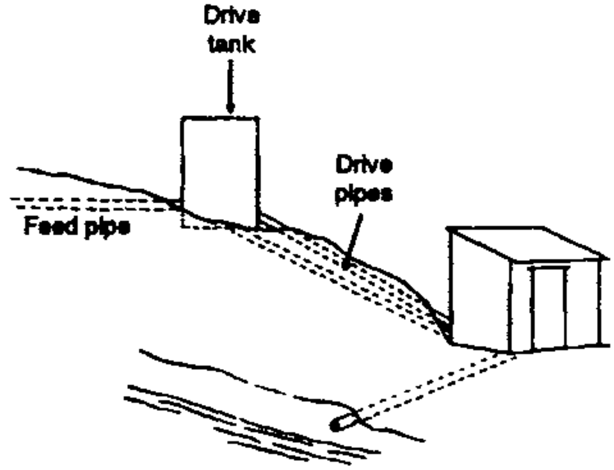


Drive system

The drive system is the part of a complete ram pump installation that receives water from the intake and feeds it to the pump(s).

It normally comprises three main items:

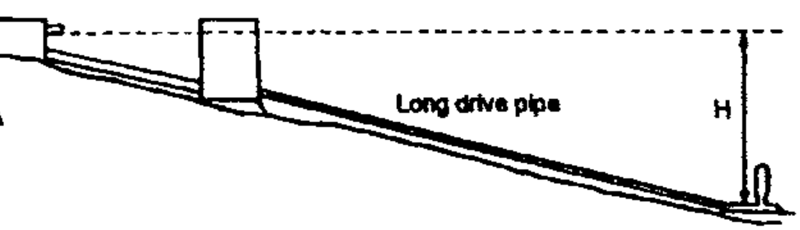
- a feed pipe that carries a constant flow of water from the intake to the drive tank;
- a drive tank that provides a store of water at the top of the drive pipe(s);
- the drive pipe(s) that supply water from the drive tank to the pump(s).



When first designing a drive system for a ram pump, all three items must be considered together because decisions involving any one of them will affect the design of the other two. In any particular situation there is a balance to be found between each of the items in the drive system. The balance involves the site layout and the relative costs of different types of pipe and of tank construction.

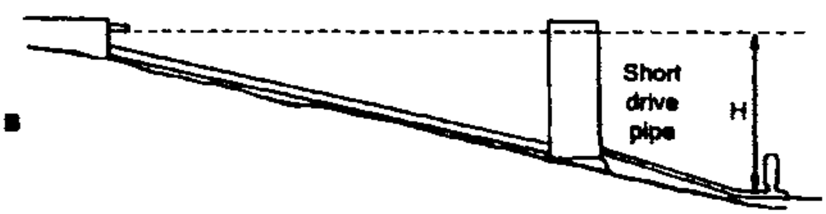
The system sketched below shows how the required drive head can be achieved in two different ways at the same site. Case A shows a small drive tank linked to the source by a short length of feed pipe. To reach the necessary drive head (H) a long drive pipe has to be installed to each pump.

Case A



Case B uses a very tall drive tank being fed from the intake by a long feed pipe and supplying the pump through a much shorter drive pipe.

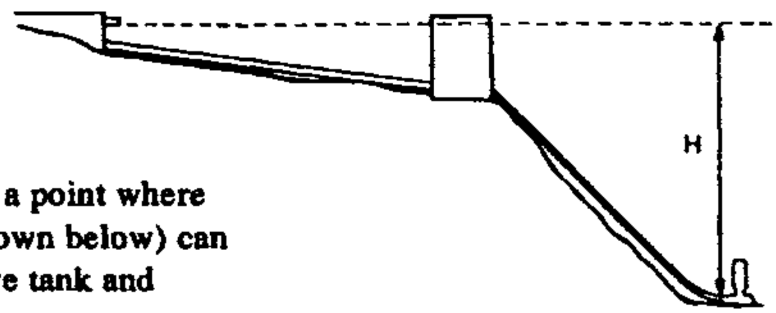
Case B



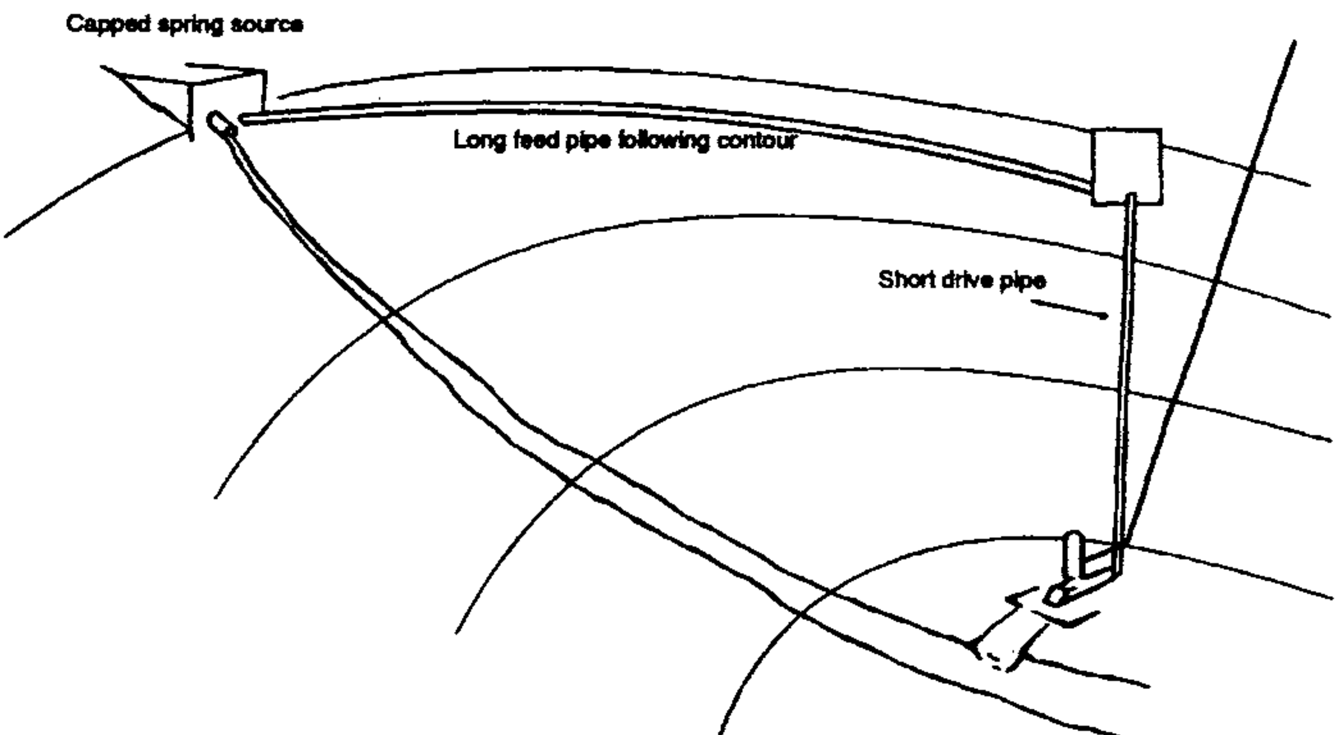
The final design choice in any situation will depend on the cost and the availability of materials and construction techniques.

Small drive tanks are generally cheap and simple to construct whereas tall ones require better foundations, reinforcing and more complicated construction techniques. On the other hand, galvanised steel drive pipe is usually far more expensive than low pressure plastic feed pipe and this cost constraint would often make Case B, using a long feed pipe with a short drive pipe, the more attractive option.

When designing a drive system it is best to look for an area below the proposed intake site which has a particularly steep slope. To obtain a certain drive head (H), the cheapest and most suitable site will be one that allows a small drive tank to be built at the top of a steep slope with a short drive pipe leading to the pump at the bottom as shown below. This will sometimes require a long feed pipe to carry water from the intake to the drive tank.



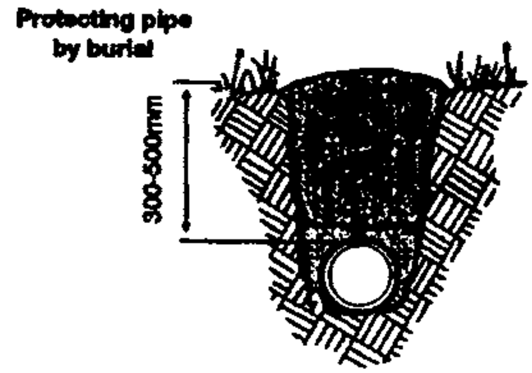
When using a spring or stream in a valley, the sides of the valley are often much steeper than the slope of the stream itself. Diverting the drive water in the feed pipe around the contour of the valley to a point where the side slopes more steeply (as shown below) can often enable the use of a small drive tank and relatively short drive pipe.



Feed pipe

The feed pipe carries a steady flow of water from the intake to the drive tank. It must supply sufficient water to feed all of the ram pumps running from the drive tank plus a small surplus that will overflow from the tank and ensure that a constant level is maintained inside. In small spring-based systems (or those using very small streams), all of the available source flow may be diverted into the feed pipe and the excess allowed to overflow from the drive tank. All feed pipes must be sized to carry enough water to supply all the pumps installed, plus an allowance for the overflow and any planned expansion of the system at a later date.

The pressure of the water in the feed pipe is generally very low, so cheap, low-pressure plastic pipe is usually adequate. If this type of pipe is used it should be buried or well covered in some way to protect it from damage by animals, farming, or strong sunlight.



The water flowing through the feed pipe will experience some friction against the pipe walls. This friction takes up some of the energy of the water and means that the water level in the drive tank under normal operating conditions will be lower than the level at the intake. This head loss caused by the friction of the flowing water must be taken into account when choosing the diameter of the feed pipe. In situations where there is plenty of drive head available a smaller and cheaper pipe size can be used. To achieve the required drive flow, the water in a small pipe will have to flow faster, with more friction and therefore a higher head loss, but when there is plenty of drive head available this will not matter. In situations where the drive head available is very limited, very little head loss in the pipe can be allowed and a larger diameter pipe through which the water flows relatively slowly should be used: the low velocity of the water in a large diameter pipe reduces the friction and therefore reduces the head loss.

Drive tank

The drive tank is an important component in the operation of a ram pump system. It performs the following tasks:

- it keeps a steady water level, ensuring a regular supply to the pumps and a constant drive head;
- it provides a large body of water with an open surface that reflects the pressure waves travelling up the drive pipe when the pump is running;
- it also prevents air from being sucked into the drive pipe.

Very occasionally systems can be sited on a steep slope (more than 30 per cent) directly beneath the water source and can be designed without a drive tank. This can reduce the cost and complexity of the system but extra care must be taken in the design of the water catchment structure, which must then also perform the functions of a drive tank.

One common mistake when designing drive tanks is to have the entrance to the drive pipe too close to the surface of the water. In such cases air is drawn into the drive pipe as the water accelerates down it. This can interrupt the operation of the pump and frequently leads to failure at the top of the drive pipe due to an effect called cavitation. It is recommended that the depth of water above the drive pipe inlet(s) be at least 500mm.

The volume of water in the drive tank, and in particular the cross-sectional area, must be large enough to maintain a steady level in the tank despite the intermittent outflow to the pump(s). As a rough guide, the cross-sectional area should be at least 20 times the total area of the drive pipes being supplied from it.

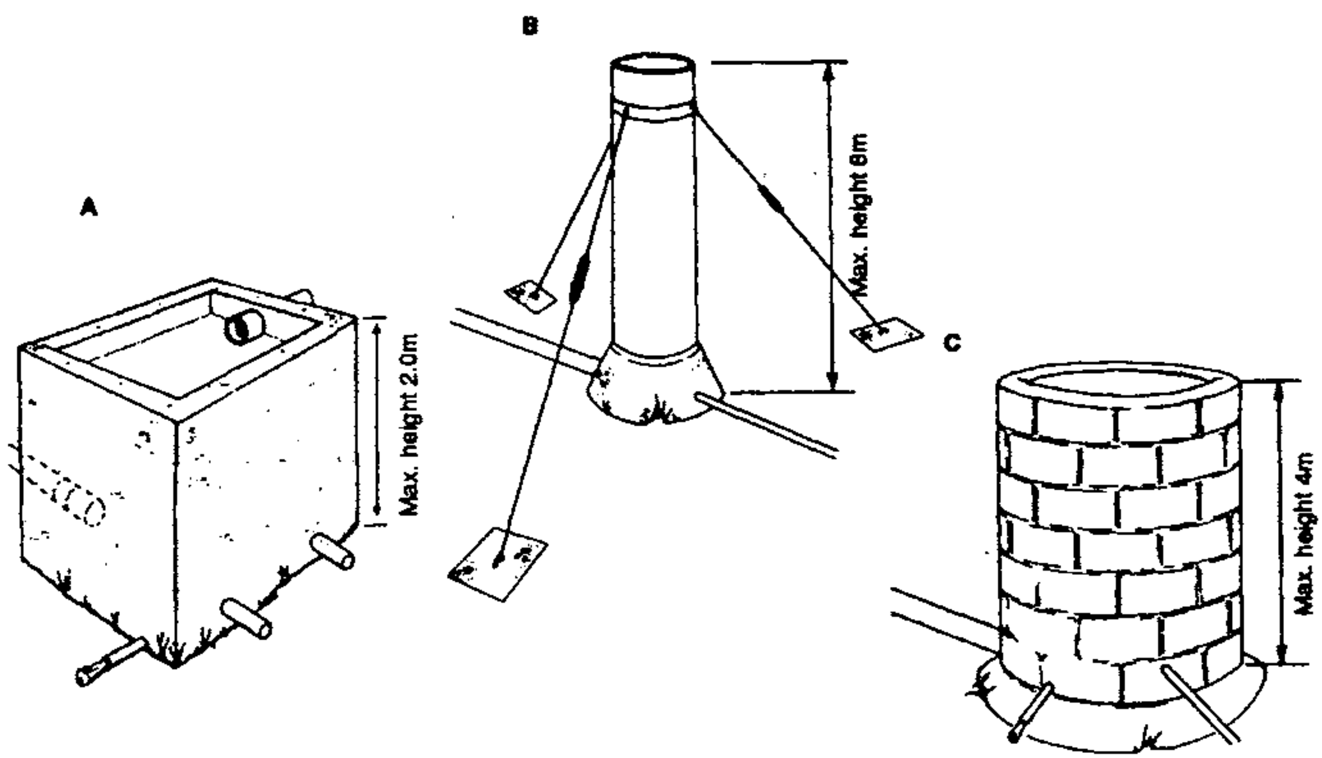
A number of types of drive tank can be considered. The choice between them depends both upon site conditions and the materials and skills available for their construction. From the authors' experience, three main types are useful, each with its own normal range of use.

A A square or rectangular tank made from cement block or fired brick and plastered inside and out. The height of these tanks is usually not greater than 2.0m.

B Large diameter PVC pipe set in a concrete base: restraining lines are recommended to ensure stability. These tanks are only appropriate for use in systems with small drive flows — particularly those systems that are spring- based. The height of these tanks is usually not greater than 6m.

C Circular tank made of cement blocks with re-bar (reinforcing bar) running vertically and horizontally within it. The height of these tanks is usually between 2 and 4m.

Care must be taken to site the drive tank conveniently for operational reasons and to ensure that it is stable. A large tank full of water is very heavy and the foundations must be strong enough to support this weight without movement. Where possible tanks should be built on a solid rock foundation.



Drive pipes

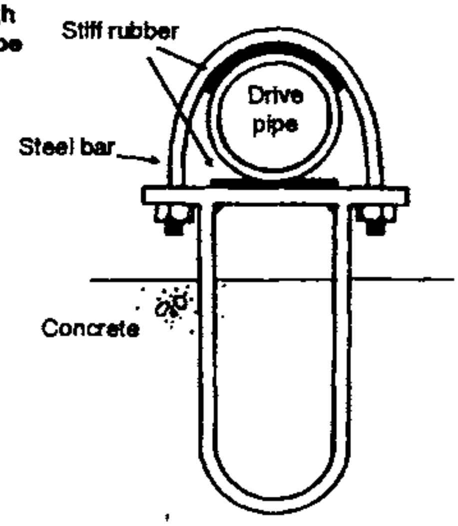
Most pumps are designed for a particular diameter of drive pipe and this recommended size should be used under normal conditions, i.e. in the normal range of drive flows and pipe lengths. Selection of drive pipe diameter is a compromise between having a large diameter to reduce friction losses and a smaller diameter to give adequate water velocity. Never install a pump with a drive pipe diameter larger than the diameter of the drive pipe connection to the pump. Research has shown that many traditional pump designs actually underutilise their drive pipe capacity and so, where drive pipe lengths and drive flows are towards the lower limit of their range, the next standard size of drive pipe below that recommended can be installed. For example, the DTU M8 and M6 steel pumps are normally driven via a drive pipe in internal diameter 2" (50mm): for drive flows (not delivery flows) of less than 70-80 litres/minutes, a 1½" (38mm) drive pipe can be used instead. Despite its higher friction, the smaller diameter pipe will give better overall efficiency (and output) for the available drive flow because the velocity of the drive water through it is higher. This is especially so when the delivery head is towards the top of the pump's range. It is generally true of all ram pumps that at low flows and high heads reducing the drive pipe diameter leads to an increase in pump efficiency.

There are limits to the length of a drive pipe in a ram pump system. These limits can be calculated by reference to several theories that take account of such factors as the energy available to the pump, the friction in the system and the movement of pressure waves within the pipe. A complete explanation of the theory involved would be too complex to give here, so only some basic guidelines are described.

A short drive pipe, that is, one with a length less than about twice the drive head, will give a very high frequency of operation. This can reduce efficiency and will shorten the life of the pump. The advantage of keeping the drive pipe as short as possible is that its cost is reduced. As a general guide, it is recommended that the length of the drive pipe be kept to between two and four times the drive head, but should never be less than 6m. For example, a system with a drive head of 5m should be designed to keep the length of the drive pipe between 10 and 20m. Systems with drive pipes longer than about five times the drive head may not be able to operate at the optimum efficiency or peak output levels quoted by the manufacturer. Ram pumps with very long drive pipes will operate satisfactorily in many situations but are inefficient and unnecessarily expensive.

The angle of the drive pipe is not a critical factor and will normally be dictated by the particular site conditions. The manufacturer's recommendations for the correct angle for a drive pipe can be ignored provided that the guidelines for pipe length given above are followed. The drive pipe can have gentle bends along its length but it must be very firmly secured near to these points.

Section through simple drive pipe anchor



Drive pipes are normally made from galvanised steel pipe, which is the usually the cheapest and most readily available material with adequate strength and life. Well sealed, threaded joints can be used in most situations, provided that the pipe is properly anchored and cannot vibrate. In systems with very high delivery heads (over 100m), threaded drive pipe connections are sometimes inadequate. In these cases flanges should be welded to the ends of the lengths of drive pipe so that they can be bolted together. The same flanged connections should be used when the threads on the pipe are of poor quality, or when large diameter drive pipe is being used.

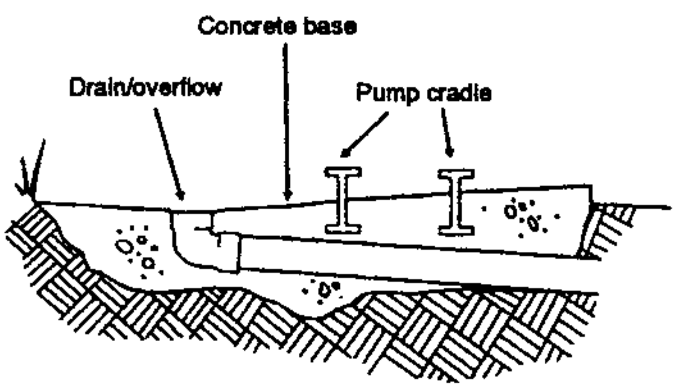
Pump

The pump itself should be considered as one component of the whole system. It needs to be carefully selected and sized in the same way as a tank or pipe, and its installation must be carried out carefully. The section on pump selection earlier in this pamphlet describes the process of choosing a ram pump or pumps to suit a particular set of conditions. After the particular requirements and constraints of an installation have been identified by the site survey, the pump(s) should be the first component to be selected. When the number and type of pump to be used is known, the sizing of tanks and pipes and the detailed design of the rest of the system can be carried out.

Pump base

It is essential that each pump be held rigidly to prevent it moving and vibrating with each pumping cycle. If a pump is not firmly secured, energy is lost as it moves and pipe joints will quickly break. The best way to secure a pump is to attach it firmly to a large concrete base. Some manufacturers recommend securing their pumps by setting bolts into the concrete but the difficulty of replacing these bolts when their threads rust or become damaged makes this unwise. A better method is to cast a steel frame, referred to as a pump cradle, into the concrete base.
There can be either a separate cradle for each pump or a single cradle to support them all. This can be made of angle-iron or other steel section welded together. The top of the cradle is left jutting out of the concrete and has holes pre-drilled through it so that the pump can be bolted down on it. If the securing bolts are damaged, they can be cut off and replaced without the need to recast the concrete base.

Section through pump base

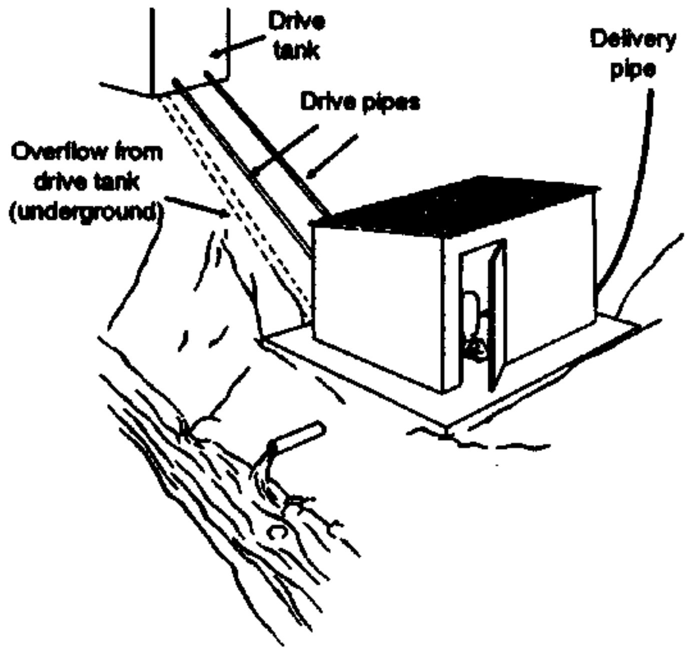


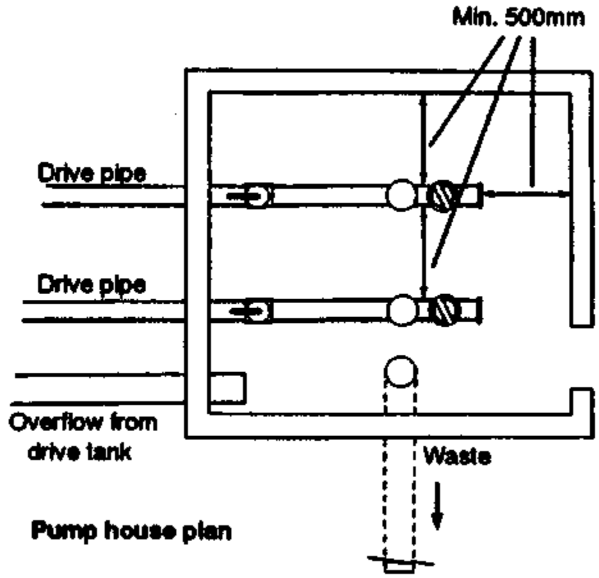
The shape of the cradle required will be different for each make of pump depending on the fixing methods incorporated by the manufacturer.

When designing a drive system, choose a suitable site for casting the pump base. The concrete base must be firmly anchored to prevent the continual vibration of the pump(s) damaging it, or separating the entire base from the material surrounding it. The site must also allow for drainage of the waste water from the pumps to prevent flooding around them.

Pump house

A structure built around the pumps is not essential to the operation of a ram pump system. However, a pump house is recommended to provide security and protection. Curious children can easily damage a ram pump left operating in the open and it also makes an easy target for thieves. The pump house must be made large enough to allow complete access to all parts of the pumps for installation and maintenance. As a general guideline, allow at least 500mm between any part of the pump and the walls of the pump house or another pump.





Pump house plan

Delivery manifold

The delivery manifold connects the outlet from one or more ram pump's air vessels to the delivery pipe that carries the water to the point of use. It also traps air and collects silt.

Delivery pipe

The delivery pipe is an important part of any ram pump system and in many cases its length leads to it being the most expensive item. This is true despite the fact that, unlike drive pipes, many pumps can share the same delivery pipe. The delivery pipe must be sized to suit the expected delivery flow and the cost constraints. The larger the diameter of the delivery pipe, the more expensive it will be, but the lower the friction losses within it. Relatively cheap delivery pipe with a small diameter will impose high friction on the delivery flow, causing a high head loss. The pumps will have to overcome this head loss as well as lifting the water to the point of delivery. Choosing a diameter for the delivery pipe is therefore a balance between having a small diameter pipe to reduce costs or having a large diameter pipe to reduce head loss and maximise delivery flow.

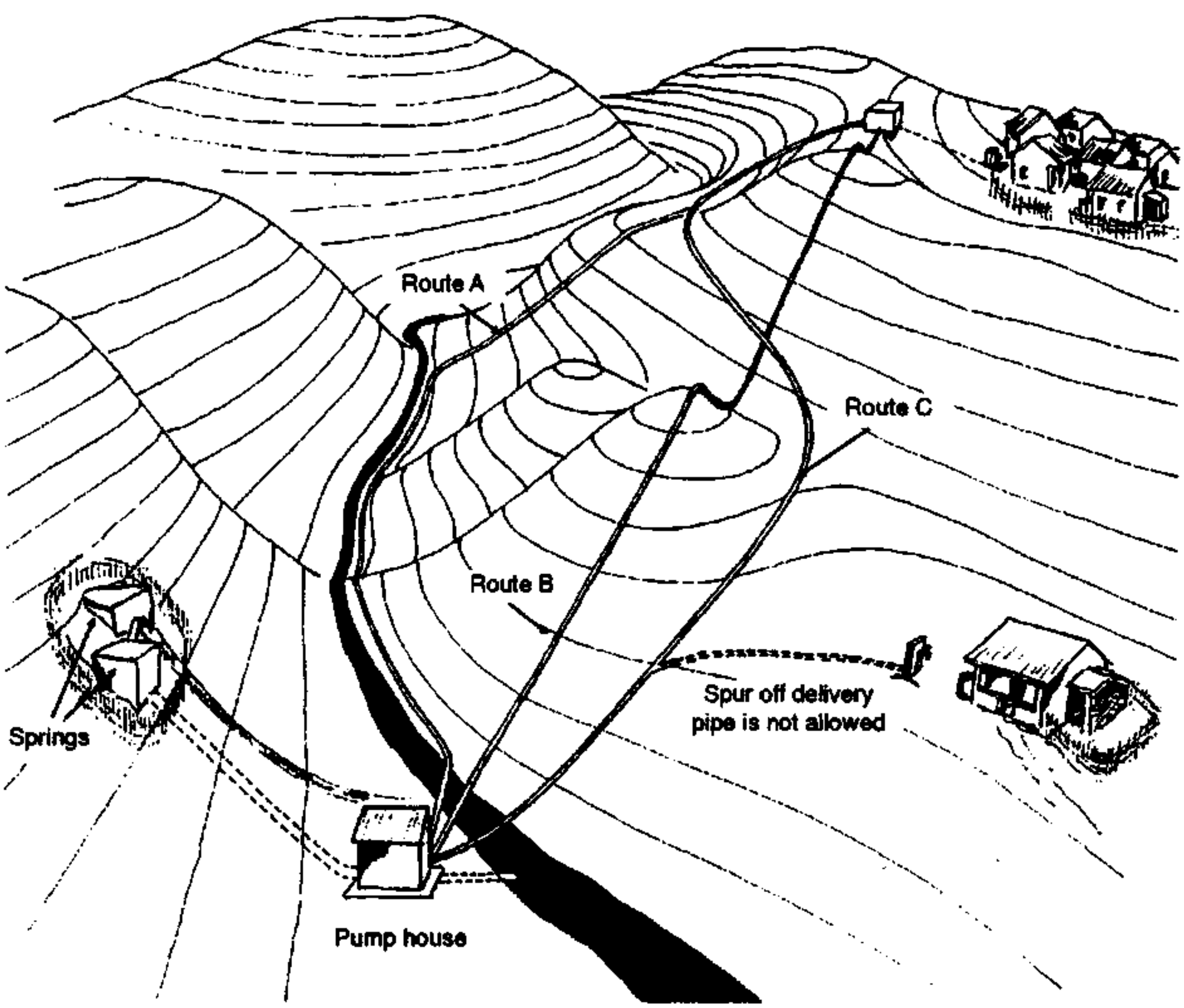
As a rule of thumb, the calculated head loss in the pipe when it is carrying the predicted delivery flow should be between 5 per cent and 10 per cent of the delivery head.

Where there is plenty of delivery water available and low cost is important, a small diameter delivery pipe can be used to reduce cost. If the expected delivery flow is only just sufficient for the users, a larger diameter pipe with a lower head loss should be chosen.

The route chosen for the delivery pipe should be as short as possible to reduce cost and head loss. However, the shortest route may not be technically feasible or necessarily the cheapest.

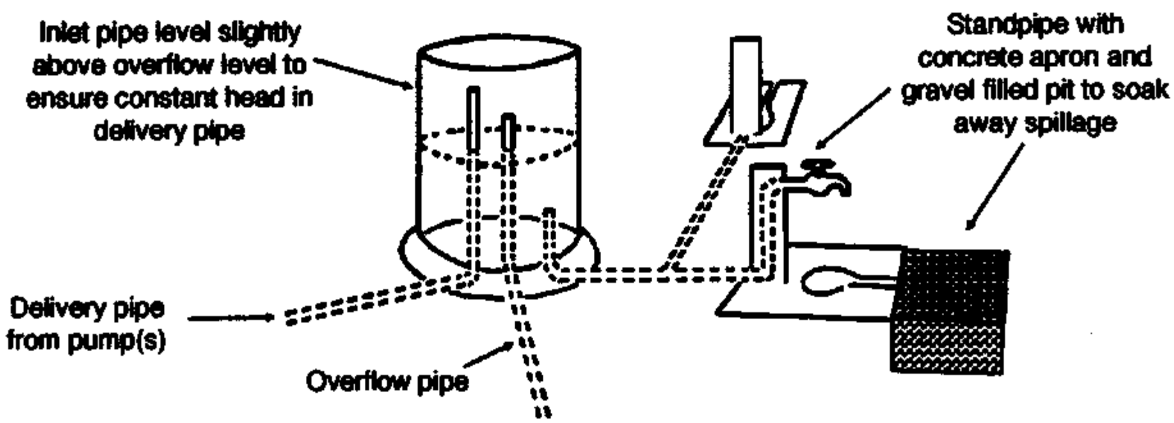
The sketch below shows a system with three possible delivery routes.

- Route A follows the lowest ground for much of its length: it is not recommended because a large section of the pipe is under high pressure and high pressure pipe tends to be more expensive;
- route B takes the most direct line but is not recommended because it has high points in which air would collect, causing air locks and reducing delivery flow;
- route C is the best route, rising quickly from the pump and then following a steady upward incline to the delivery tank. Only a short length of pipe is under high pressure and the line is fairly direct. A spur off the delivery pipe below the delivery tank should never be installed.



Distribution system

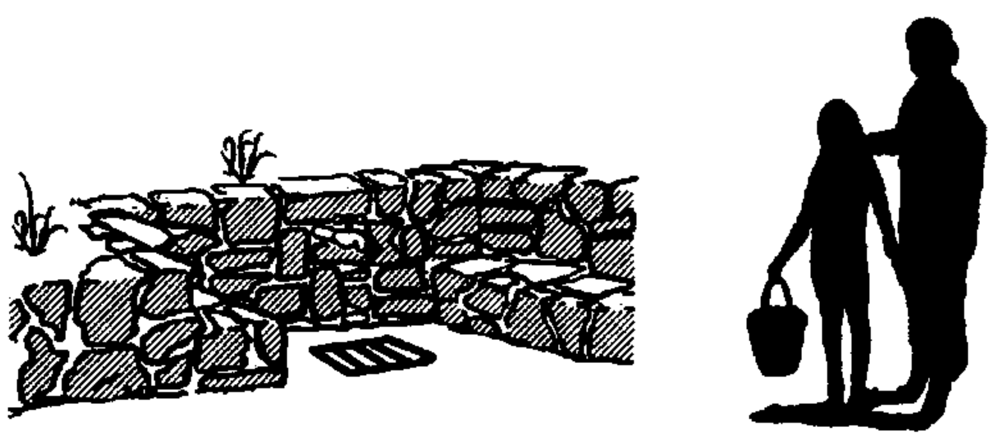
After installation and commissioning, most of the components of a ram pump system are forgotten by the users of the water supplied. The water is collected every day from stand pipes and it is the design of this distribution system that will be most relevant to them. It is particularly important to involve the whole community in the design of the distribution system and that the social organisation of water use is planned before construction.



Most ram pump systems for domestic use will require a storage tank at the end of the delivery pipe. A ram pump operates for 24 hours each day, so the water pumped overnight must be stored for use during the times of peak demand. If there is to be more than one storage tank, the water should be pumped to the highest one and then fed by gravity down to the others. Alternatively, if valves are fitted on the inlets to each tank (except the highest one) the flows can be controlled so that all subsidiary tanks fill at the same rate. The total capacity of the delivery tank or tanks in the distribution system must be large enough to hold all of the water pumped overnight. They should therefore be sized to hold at least 12 hours supply and preferably 16 hours supply. The storage requirement can be calculated using the rule:

$$\text{Storage required in litres} = \text{delivery flow in litres/minute} \times 60 \text{ minutes} \times 16 \text{ hours}$$
$$= q \times 960$$

Thus, for each litre per minute of delivery flow, provide about one cubic metre (= 1000 litres) of storage.



The delivery tank should either be conveniently situated for direct use or feed water into a distribution system. In many cases distribution can be directly through stand pipes connected to the delivery tank and fed by gravity. Larger systems serving bigger communities may require a number of distribution points each with a secondary storage tank and stand pipes. These tanks can either be assigned a priority — where the overflow from one tank feeds the next in a chain — or can be fed equally from the main delivery tank.

Single and multiple tank delivery systems

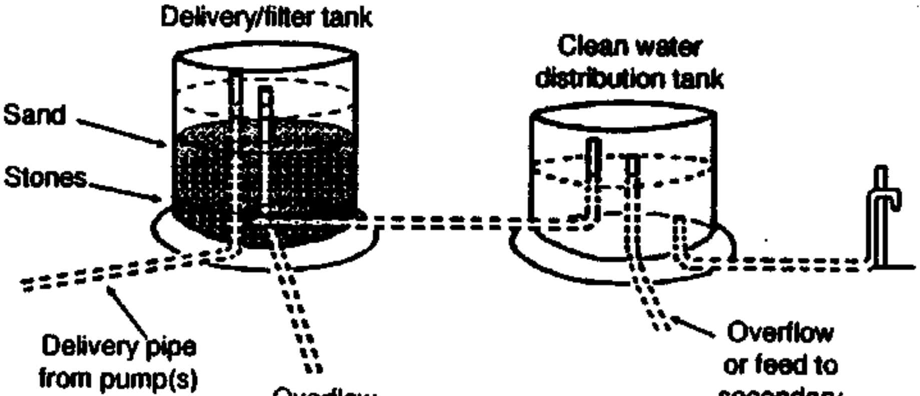
Single delivery tank

Priority chain delivery system

Equal priority multiple tank delivery system

Water drawn from springs and high mountain streams is often very clean and can be used without any form of treatment. Where the water to be used in a ram pump system is drawn from a dirty source, filtration of the water delivered may be necessary. Simple sand filters when carefully installed and maintained can be very effective in improving water quality. The details of filter design and construction can be found in many publications and are beyond the scope of this book. From a system design point of view, however, it is worth noting that the delivery tank can itself be a filter tank as shown below. The delivery/filter tank feeds clean water to a second storage tank from which water is drawn for use. The top of the link pipe in the distribution tank should be a few centimetres higher than the level of the sand in the filter tank: this ensures that the sand never dries out.

Simple method of filtering delivered water



MILESTONE 1: REPORT C1

“DRWH WATER QUALITY: A LITERATURE REVIEW”

Prepared by

PROF. PADMA VASUDEVAN AND DR. NAMRATA PATHAK

Centre for Rural Development & Technology

Indian Institute of Technology, Delhi

India

Sub programme C: Health Implications

Project sponsored by

EUROPEAN COMMISSION

TABLE OF CONTENTS (REPORT C1)

SECTION – I

DRINKING WATER: THE INDIAN SCENARIO

The Global Dimension

Water is life. This colourless, odourless and tasteless liquid is essential for all forms of growth and development - human, animal and plant. Also, water is a fundamental basic need for sustaining human economic activities. Not only does water support a wide range of activities, it also plays a central symbolic role in rituals through out the world and is considered a divine gift by many religions. Availability of water in the desired quantity and quality, at the right time and place, has been the key to the survival of all civilizations. No other natural resource has had such an overwhelming influence on human history. As the human population increases, as people express their desire for a better standard of living, and as economic activities continue to expand in scale and diversity, the demands on fresh water resources continue to grow.

While water is a renewable resource, its availability in space (at a specific location) and time (at different periods of the year) is limited, by climate, geographical and physical conditions, by affordable technological solutions which permit its exploitation, and by the efficiency with which water is conserved and used. Much of the world's fresh water is consumed by the agricultural, industrial and domestic sectors. Increasing water demands and the inadequacy of these sectors to effectively manage this resource, has lead to crises situations in many parts of the world - crises over the availability of adequate and appropriate quality water. The limits of sustainable use in each climatic region are determined by local climate, hydrological and hydro-geological conditions. In many parts of the world, the amount of water being consumed has exceeded the annual level of renewal, creating a non-sustainable situation. The International Drinking Water Supply and Sanitation Decade and other international declarations have clearly recognized that access to water is a fundamental right of people.

Fresh water lakes and rivers, which are the main sources of water consumed by people, contain an average of 90,000 cu km of water, or just 0.26 percent of total global fresh water reserves. This tiny fraction is distributed in a very uneven manner on earth, creating wide range of environments, from arid regions and deserts to humid areas which experience regular flooding. In many parts of the world, the rainfall pattern is highly skewed and is characterized by small periods of intense precipitation followed by long, dry periods. Great disparities may even be seen on the same continent. About 20% of the total global run-off flows in the Amazon river in South America, while the Atacama Desert has consistently received non annual rainfall.

Such variations become very important as human activities diversify geographically and in scale. In many water scarce parts of the world, human engineering initiatives have been geared towards balancing this spatial inequity. In South-Western USA, e.g. engineering interventions in the form of extensive dams have already exhausted most possibilities for enhancing fresh water availability. In many other parts of the world, future options are becoming extremely complex and uncertain as the levels of total fresh water consumption approach the limits imposed by the annual renewal of fresh water resources.

Advances in climatology and hydrology have contributed to improved, quantitative estimations of the processes which make up the global hydrological cycle. Though this knowledge has resulted in increased availability of water in some situations, an almost exponential growth in the level of utilization of this resource has balanced off the advantages so created. In this way, in spite of advances made on the scientific front, human survival and well being today are probably no less dependent on fresh water availability than in the early years of human civilization. Not with standing some impressive records in activities related to the UN Drinking Water and Sanitation Decade (WHO 1990), the provision of water at affordable cost and of acceptable quality is emerging as a major environmental challenge. In particular the all dependence of future food security on the availability of irrigation water, as well as growing awareness of water resources for conservation purposes has created widespread concern.

There are distinctions between `water scarcity', `water shortage' and `water stress'. Water scarcity is a relative concept intended to convey the imbalance between supply and demand under the prevailing legal, institutional, regulatory and where applicable, price arrangements. Water shortage is an absolute concept indicating low levels of water supply relative to minimum levels necessary for basic needs. Water stress signifies acute water shortages for prolonged periods.

In this respect, it is important to examine whether the emerging water scarcity in various parts of the world is absolute, needing drastic reductions in demand, or can be adequately addressed through new and holistic management strategies and restrained consumption patters. The need for a totally new perspective and the manner in which people use fresh water has been felt and the existing perceptions of engineers regarding water supplies has been questioned.

The need to adopt a systems approach to water management has been stressed. Along with the various ecological linkages governing the flow of fresh water in the hydrological cycle, the need to understand the use of water in its many diverse roles and its economic implications is also being recognized. The Earth Summit Agenda 21, specifically calls for local and national level actions.

Delhi and Dublin Principles

Delhi "some for all rather than more for some". Guiding principles:

- protection of environment and safeguarding of health through integrated management of water resources and liquid and solid waste;
- institutional reforms, integrated approach and full participation of women at all levels;
- community management and strengthening of local institutions in implementation;
- sound financial practices.

Dublin: Emphasis on sustainability and the need to consider water as an economic good. Guiding principles:

- fresh water is a finite and vulnerable resource essential to sustain life, development and the environment;
- water development and management should be based on a participatory approach involving users, planners and policy makers at all levels;
- women play a central part in the provision, management and safeguarding of water;
- water has an economic value in all its competing uses and should be recognized as an economic good.

Source: Global Consultation on Safe Water and Sanitation for the 1990s, Delhi and International Conference on Water and the Environment, Dublin, 1992.

Earth Summit, Agenda 21

At the lowest appropriate level, delegation of water resources management, generally to that level in accordance with national legislation, including decentralization of government services to local authorities, private enterprises and communities.

At the national level, integrated water resources planning and management establishment of independent regulation and monitoring of fresh water, based on national legislation and economic measures.

Source: United Nations (1992)

Water: The Indian Scenario

In a country where the first measurement of rainfall was made by Kautilya as early as AD 1200, it is surprising that estimates of the total availability of water in India are only quite recent. The total average annual run-off of all river systems in India is estimated to be 1,674 billion cubic metres (BCM).

The National Water Policy estimates that total precipitation in India is around 400 million hectare metres. In addition, ground water potential is about 42 million hectare metres (GOI 1987). The first estimates of ground water resources on a scientific basis was made in 1979 by the Central

Ground Water Board. Recent estimates based on a state-wise assessment have put the annual replenishable ground water resources of the country at 453 BCM. With a provision of about 15% (69.8 BCM) for drinking, industrial and other uses, the utilizable ground water resources for irrigation is computed as 383 BCM (GOI/CGWB 1995).

The amount of available aggregate annual utilizable water in India, surface and ground is about 1,100 BCM. Population growth is expected to result in a decline in the per capita availability of fresh water. In 1947, this was measured at 5,150 m^3. By the year 2000, it is likely to be 2,200 m^3.

Such aggregate figures, however, are quite misleading, since there is considerable spatial and temporal variation in rainfall. Some areas receive slight rainfall, whereas others experience monsoon conditions which often result in flooding, loss of life and increased poverty. To better understand such variations and their consequences on people's lives, it is necessary to examine specific situations at the village or community levels under different ecological situations.

Attention must, however, also be given to fast growing urban centres, where water requirements are expected to double from 25 BCM in 1990 to 52 BCM in 2025. The situation concerning industrial supplies is even more difficult to analyse. It has been indicated that industrial water demand would increase from 34 BCM in 1990 to 191 BCM by the year 2025. Agriculture, the largest consumer of water resources in India, will probably require 770 BCM by the year 2025 to support food demand. The total estimated demand of 1013 BCM by the year 2025 would be close to the current available annual utilizable water resource of India.

With predicted demands such as these, the supply of rural drinking water and requirements for ecosystems conservation are sure to face an uncertain future unless anticipatory policy measures are taken. It is evident that the politically and economically powerful urban-industrial sectors would obtain the water resources they need by organising long distance transfers from surrounding rural areas or even by inter-basin transfers. In such a scenario, alternative solutions of conservation and sustainable management of fresh resources will find little support. In view of this, the focus has to be on the requirements of rural drinking water and ecosystem conservation, while at the same time suggesting alternative approaches for meeting urban demands.

The National Policy

The government's concern since independence, has been raising the quality of life and the health of the people. Several initiatives were taken at policy formulation level leading to various programmes in this direction. Supply of safe drinking water and provision of sanitation are the most important contributing factors for improving the health of the people in any country. As per a World Health Organization (WHO) report 80% of the diseases are due to unhygienic conditions and unsafe drinking water. It is estimated that every year about 1.5 million children under five years die in India of water related diseases. The country loses over 200 million man days each year due to water and sanitation diseases. Age old cultural practices coupled with illiteracy and lack of awareness further complicate and exacerbate the problem. Supply of safe drinking water has therefore, been given very high priority in Indian Planning.

Providing drinking water in rural areas is the responsibility of the state governments and the funds were provided for the purpose in their budgets from the first five year plan. During 1954 National Water Supply and Sanitation Programme was introduced in the social welfare sector. The states built up gradually the Public Health Engineering Departments (PHEDs) to attend to the problems of water supply and sanitation. Under the programme 100% grants-in-aid to implement the different water supply schemes for the `problem villages' were provided by the government of India. In the mid 1960s it was realised that these schemes were implemented only in the easily accessible villages and in the process the hard core `problem villages' remained unattended. The government of India during the fourth five year plan took steps to provide assistance to the states to establish special investigation divisions for the problem villages.

In order to accelerate the pace of coverage of problem villages, the Government of India introduced the Accelerated Rural Water Supply Programme (ARWSP) in 1972-73. During 1974-75 the Minimum Needs Programme (MNP) was introduced because of which the ARWSP was withdrawn but it (ARWSP) was reintroduced in 1977-78 when the progress of supply of safe drinking water to the identified problem villages was not found to be satisfactory.

In the year 1977 the United Nations Water Conference separated the issue of drinking water and sanitation from other water issues to stress the seriousness and magnitude of the problem of drinking water. The conference recommended that each country should develop national plans and programmes for water supply and sanitation giving priority to the schemes of the population which require greatest attention. India was a signatory to the resolution seeking to achieve the target by 1991. The water decade programme was launched in India on 1st April, 1981 to achieve definite targets of coverage of entire population by 31st March, 1991.

In August, 1985 the subject of rural water supply and sanitation was transferred from Ministry of Urban Development to Department of Rural Development with the objective of securing implementation of the programme and their integration with other rural development programmes.

The National Drinking Water Mission was launched as one of the five societal mission in the year 1986. The mission was since named as Rajiv Gandhi National Drinking Water Mission (RGNDWM) in 1991. Government of India continues to give highest priority to rural drinking water sector through the activities of the mission and ARWSP. It also forms the part of the state funded MNP and point No. 7 of the twenty point programme, 1986.

It is claimed that the RGNDWM over the last decade has successfully covered the majority of habitations with hand pumps/stand posts. However, it has now been realised that the objective of supplying safe water would not be achieved to the extent and satisfaction expected unless the pollution aspects of water supply, as well as the issue of sanitation were addressed simultaneously. The focus has now shifted from water to water and sanitation. The mobilisation of large funds and efforts through RGNDWM in this direction have not yielded the desired impact on the health of the general population. Many reasons could be advanced to explain the not so satisfactory results of the efforts under water and sanitation programmes over the years.

During the 1970s, there was a marked departure from sustainable utilization of water resources. Food scarcities of the 1960s encouraged government policies towards increased irrigation. In this way, the users of drinking water and irrigation, which had until then been a singular entity, started to be separated. This shift affected the management of common water resources in basic ways. One of the most visible changes was the manner in which upper catchments were managed, leading to a degradation of water resources in tanks, lakes and rivers. It also led to ground water being extracted from greater depths, making the shallow hand dug wells, which until then had provided drinking water, redundants. The situation has been described as human induced water scarcity, normally mistaken as being the result of natural drought. What made the situation even worse was increased pollution of both surface and ground water resources which are of both man made and natural origin. In this perspective, unless pre-sumptive measures in terms of new regulatory and policy instruments are adopted, the water situation in India is certain to become chaotic.

National Water Supply and Demand

The changing socio-economic situation in India is leading towards higher levels of ground water exploitation. With the increasing availability of more sophisticated drilling and pumping technology, the search for ground water is bound to increase. The results of excessive ground water use is already showing. Small streams, are drying up due to insufficient catchments even during the monsoon season, and in both rural and urban areas people are drilling deeper and deeper borewells. In other situations, a significant amount of rain might fall, but it is not possible to store it for domestic needs. In the hills, deforestation and reduced ground cover results in very little rainwater percolating into the soil to feed the springs. Soil erosion further reduces the capacity of the ground to retain water. Cheerapunji in eastern India, for example, may receive 10.5 m of rainfall in the short monsoon period, but it too suffers from water scarcity!

India is heavily dependent on ground water sources. It is estimated that this source provides about 80 to 90 percent of domestic water supply in rural areas, 50% of the urban and industrial demand, and 50% of the irrigated area through over 17 million energized wells. In drought years, ground water represents the primary reliable source for irrigation. (World Bank/ GOI 1997a &

1997b). However, domestic water needs account for only about 5% of the total water extracted from the ground.

A dramatic increase in ground water extraction took place in India from 1951 to 1990. The number of dug wells increased from 3.86 million to 9.49 million, shallow tubewells from 3,000 to 4.75 million and public tubewells from 2,400 to 63,600. The number of electric and diesel pumps also increased during this period, from 21,000 to 8.22 million and from 65,700 to 4.36 million, respectively, electric pumps becoming more common as a result of rural electrification. In gross terms, however, the current level of ground water use is 32%, suggesting that there is still vast potential for its further development, but there are significant variations with a number of blocks in the country classified as `dark areas' or `over exploited' with more than 85 and 100% of ground water development, respectively (GOI/Central Ground Water Board, 1995).

With the heavy dependence of the country on ground water, the government's strategy has been based on using the dynamic component of ground water (i.e. the amount available in the zone of water level fluctuation), and temporary use of the static component (i.e. the amount available in the permeable portion of the aquifer) to cope with drought situations. The National Water Policy (1987) sets out the framework for the implementation of this strategy. Current legislation (common law) assigns property rights of surface (natural) water resources to the state, while rights to the extraction of ground water, which is the major source of drinking water in India, rest with those individuals who own the land above the aquifer. There is no limit on the quantity of ground water that a landowner can extract.

<u>National Water Policy (1987)</u>

- Water is a prime natural resource, a basic human need and a precious national asset.
- Periodic scientific assessment of the ground water potential, taking into consideration the quality of water and economic viability.
- Exploitation of ground water resources should be regulated so as not to exceed recharging possibilities and to ensure social equity with ground water recharge projects formulated and implemented for augmenting the available supplies.
- Integrated and co-ordinated development of surface water and ground water and their conjunctive uses planned right from the start of a project.
- Avoidance of over exploitation of ground water near the coast to prevent ingress of sea water into sweet aquifers.
- Drinking water needs of human beings and animals should be the first charge of any available water.

The water supply and sanitation sector, particularly in rural areas, has been given priority from the inception of the five year planning process in India. In total, during the five year planning periods 1951-56 to 1992-97 Rs. 336 billion or 3.3% of the total government budget has been allocated to this sector of which 60% (Rs. 202 billion) was for rural areas. Government investment in rural water supplies and sanitation was Rs. 143 billion upto 1996. From 1991 to 1995 total external support to the water supply and sanitation sector amounted to US $ 330 million or US $ 56.5 million per year which represents 2 per of total external disbursements in India. But it is also noted that the utilization rate of both multilateral and bilateral assistance in India is low. For example in 1992-1993 it was only 10% of commitments (World Bank/GOI 1997b). Estimates of private investments are not available, but they are likely to far exceed that of the government if irrigation and domestic expenditures in water extractions are included.

According to the Rajiv Gandhi National Drinking Water Mission (RGNDWM) a total of 520 million people have been provided access to public water supply since the launch of the first national water supply programme in 1954. During the period 1954-55 to 1994-95 it is estimated that 478 million rural people were covered with water supply. By 1994, 95% of the rural population had access to a `safe` source of water, with 52% fully covered with 40 litres per capita per day (lpcd) or more, and 48% partially covered with 10-40 lpcd. Only about 5 percent of the rural population were without access to safe water.

In terms of physical infrastructure, more than two million handpumps have been installed on drilled tube and borewells, 1,16,000 mini and regional piped schemes have been constructed supplying 1.5 million standposts and 4.3 million house connections. Moreover, handpumps account for 95% of the total number of publicly funded rural water supply schemes, serving almost 395 million people, or 75% of the rural population.

A 1994 Government of India survey examined the status of handpumps. It found that many required repair (more than 33%), or rehabilitation (22%), or were completely defunct (12%). In the case of piped water supply the situation was less serious with about 26% requiring repair or rehabilitation. Eighteen percent of all standposts were found to be without taps (World Bank/GOI, 1997b).

The RGNDWM Validation Survey has also reported significant problems with water quality. Approximately 82,000 habitations or about 4 million people are suffering from water quality problems as a result of excessive quantities of fluoride, iron, nitrate and arsenic or excessive salinity. The Ground Water Sub-group of the Water Resource Management Sector Study by the World Bank and Government of India reports that arsenic is a recognized problem in West Bengal (1,000 habitations or an approximate population of 5,00,000), fluoride levels are considered high in Andhra Pradesh, Gujarat, Haryana, Karnataka, Punjab, Rajasthan, Tamil Nadu and Uttar Pradesh (28,000 habitations or an approximate population of 14 million), high iron levels have been found in the north-east and eastern parts of the country (58,000 habitations or an approximate population of 29 million), and high salinity is prevalent in Gujarat, Haryana, Karnataka, Punjab, Rajasthan and Tamil Nadu (World Bank/GOI, 1997b).

With an area of 3,268,100 km^2, India has 33 meteorological sub-divisions. Almost one-third of the country, 99 districts in 13 states, covering 108 million hectares have been classified as drought prone. As of March 1994, out of the 7024 blocks, Mandals, Talukas and watersheds in the country, 537 blocks and Mandals (102 Mandals in Andhra Pradesh, 32 in Haryana, 9 in Karnataka, 3 in Madhya Pradesh, 73 in Punjab, 68 in Rajasthan, 97 in Tamil Nadu, 65 in Uttar Pradesh, 2 in West Bengal), 45 Talukas in Gujarat and 35 watersheds in Maharashtra were classified as `dark` or critical where the projected net extraction in five years would be in excess of 85% of the ground water resources utilizable for irrigation. Another 600 blocks, Mandals, Talukas and watersheds are classified as `grey` or `semi-critical` with projected extractions in the 65-85% range.

In response to the emerging problems of ground water, the RGNDWM has as far back as 1987 identified strategies for the short and long term for meeting drinking water needs and micro-watershed management such as the conservation of water and recharging of ground water aquifers and a model bill has been proposed by the Central Government.

Technology Mission: Some approaches for the conservation of water and recharging of ground water aquifers

- collection of hydro geological and related data in problem areas;
- reconnaissance to verify the available data and to update micro level data base;
- **chemical analysis of water for evaluating its suitability for drinking;**
- construction of suitable structures for ground water exploration and periodical monitoring of their performance;
- **designing and construction of suitable structures for rain water harvesting;**
- **artificial recharge of aquifers (wherever feasible);**
- development and adoption of measures for reduction of evaporation losses from surface water bodies;
- conservation of water through adoption of appropriate irrigation practices;
- identification of micro-watersheds in problem areas;
- assessment of hydro geological parameters of aquifers;
- estimation of recharge to ground water regime;
- assessment of total water potential in basins and micro-watersheds;
- scientific management of water resources using computerized mathematical modeling.

Key components of the Ground Water Model Bill

- Control and/or regulation of the extraction of ground water in any area deemed necessary and notified by a Ground Water Authority;
- Need to obtain a permit to extract and use ground water in the notified areas;
- Registration of existing and new users in the notified areas;
- Monitoring and enforcement of the controls and regulations by the Ground Water Authority.

In 1992, the constitution act (73rd amendment) gave responsibility for drinking water and sanitation to the Panchayati Raj Institutions. The underlying rationale is that the public health engineering departments and water boards are centralized, monopolistic, overstaffed, and lacked accountability to users. The Gram Panchayats as the local level tier are now expected to be responsible for choice of technology, recovering costs and operations, and maintenance of rural water supply and sanitation. The assets would be owned by the community. This process is, however in a very early stage in most states, but Gram Panchayats are now almost entirely implementing development programmes that are handed down to them by the state and central governments. However, because the governments continue to control the grants to the Panchayats, they continue to exercise control on the day-to-day functioning of the panchayats, and the state governments still continue to act as the providers of minimum coverage of free water supply in rural areas.

The broad picture of the demand and availability of fresh water has typically suggested certain generalized solutions such as the need for resource management rather than controlled resource extraction and improved environmental management in critical zones. Alternative mechanisms for water allocation in such a complex situation have been studied by Meinzen-Dick and Mendoze (1996). Specific solutions have pointed to the promotion of water markets, reforming the tariff structure of electricity, prohibiting certain crops in water scarcity areas, creating legal and institutional frameworks, and re-orienting investments in the sector.

The ability of some suggested changes has often not taken into account the regional and ecological differences that prevail in the nature and assessment of the fresh water situation, including social and cultural factors. Policies and plans developed at the national level, and calculations of per capita fresh water needs based on national data have little meaning in a country of this size. The water issues in India must be analysed in a dynamic context, both over time and for specific locations.

Drinking Water: Current Problem and Perspective

The lives of women and children as well as the environment have been seriously threatened by water shortages in the country.

- As a result of excessive extraction of ground water, drinking water is not available during the critical summer months.
- About 5 percent of the rural population does not have access to regular safe drinking water and many more are threatened by less and less access to safe drinking water in the not so distant future. Water shortages in cities and villages have led to large volumes of water being collected and transported over great distances by tankers and pipelines.
- High levels of fluoride, arsenic and iron, lead to major environmental health problems and in the case of iron, people simply do not like to drink the water because of its smell / taste.
- Ingress of sea water into coastal aquifers as a result of over extraction of ground water has made water supplies more saline, unsuitable for drinking and irrigation.
- Pollution of ground and surface waters from agro-chemicals (fertilizers and pesticides) and from industry poses a major environmental health hazard, with potentially significant costs to the country.
- The World Bank has estimated that the total cost of environmental damage in India amounts to US $ 9.7 billion annually, or 4.5 percent of the gross domestic product. Of this, 59 percent results from the health impacts of water pollution.

- It has been recently estimated that by 2017 India will be `water stressed' per capita availability will decline to 1600 cu m. Cities generate 2000 crore litres of sewage but treat only 10% of it. Poor drinking water and sanitation infrastructure will lead to high levels of water related diseases and death. It is estimated that 60% of irrigation water is wasted by seepage through unlined field channels and due to over application.

Evidence suggests that not only is there an emerging water crisis at the global scale, but that the crisis is already happening in many parts of India. Ground water is being over exploited, surface water is utilized inefficiently, as is water used for irrigation and urban water supply and water pollution is escalating at exponential rates, not least because of poor sanitation. The poor in rural and urban areas, particularly women and children, continue to be hard hit by these emerging problems. There is a fear that unless urgent measures are taken, present and future generations of children will have to bear immense health and economic costs. The government has recognized that there is a problem with availability of quality water.

India's National Water Policy recognizes the importance of providing safe drinking water to its people. It states "Drinking water needs of human beings and animals should be the first charge on available resources". For children, specifically, this right is also enshrined in article 24 of the convention on the rights of the child (CRD) ratified by the Government of India. It has been recommended that water be treated as not just as an `economic good' but as an `economic resource' which is essential for growth and development. But many aspects of the National Water Policy, legislation and regulations and the rights of children under the CRC have not been implementable in the Indian context for various reasons.

Water for irrigation is available as a `free' resource apart from its extraction costs, and while industry may be paying for water through a metering system, water is not treated as an `economic resource' whose price reflects its demand and supply in its competing uses. Ground water is considered a `free good', despite the fact that it comes from common pool aquifers. Subsidized water is often cornered by rich farmers who also cause long term aquifer damage due to excessive withdrawal by taking advantage of energy or water subsides intended for small farmers. The greatest pressure and the most serious impact is on availability of water for domestic uses which faces competition from irrigation and industry. The selling of water through water markets for irrigation has developed in parts of the country. There is, however, no effective regulatory mechanism to ensure ecological sustainability. Sustainable domestic water supply cannot be assured without at the same time addressing the inter-linked issues of water for irrigation, industry and ecosystem sustainability.

Market and state regulatory mechanisms which allocate water resources to competing uses agriculture, industry, domestic and ecosystem sustainability respond to different signals. The price of agricultural products, the major consumer of ground water, is a key determinant of the crops grown and the cropping pattern followed along with the soil, climate and water situation. However, partly because legal rights have been conferred to water which lies below an individual's land, and there has been no pricing of water, the cropping pattern does not take account of the cost of over-extraction. the issues in the case of industry are water tariffs based on usage, water pollution and recycling. Without an effective pricing and regulatory mechanism, the cost of water pollution may not be factored into the price of manufactured goods.

The fresh water problem in India, therefore, as suggested by existing analyses and policies, is that adequate quality fresh water is not available at an affordable cost in the right place at the right time for basic needs and ecosystem sustainability.

The problem is conceptualised further in terms of control and management issues. The fresh water resource from surface and ground water is physically available to each of the levels household/community, district, state, national and international. At the lowest level, a household, village or community has access to a well or surface water within its own boundaries, which is not shared with other levels. Water may also be obtained from an aquifer or surface source which is shared with other levels, from the household to the village/community, district, state, national and international levels. In these cases, if a household or community is using water from a shared aquifer, then the rate of extraction at each level will affect the level of availability at the other levels. Thus, concerns over water access may range from those of villagers in an isolated area, to people many kilometers .

The interlocking nature of concerns at different levels results in legal, institutional and economic issues which must be taken into account. While a household/community has control over a water resource wholly within its land, its ability to manage the resource varies as it is shared at successively higher levels. This has implications, for instance, for community management of water resources.

Conceptually, this also suggests that when designing actions, the shared nature of the resource must be taken into account. Although this would suggest that ground water should be regarded as a public good, the existing system of `water rights' and the grandfathering of the rights prevents the design of policies and programme along these lines alone. The fact that these rights has been conferred, and millions of ground water extraction structures created, that there is heavy reliance on ground water is a political look in India, all mean that the economics of water resource management in Indian will be quite complex. The conceptual framework itself suggests that actions will be required at different levels, household, village/community, district, state, national and international.

Thus the problem has to be addressed at all these levels. The integration of the issues of fresh water supply and demand and the water balance, technology, institutions, and the legal and socio-economic aspects at the local level have to be meshed into policies for water resource management.

Domestic Roof Water Harvesting (DRWH): How Relevant?

From the above discussions it is clear that all possible approaches must be tried to mitigate the immediate problem of drinking water, maximising the control of the households with regard to their own water security. Domestic roof water harvesting seems to be an appropriate option under these conditions provided it is cost effective. Infact traditionally this was the major option to the population in water scarce areas where the people had also learnt to control their demands, conserve water to the extent possible and manage to fulfil drinking and other domestic water needs, essentially by water harvesting. Given the ubiquitous contamination of not only surface but also ground water by pathogens and chemical contaminants induced by population growth, intensive agriculture and industrialization, DRWH has become more relevant than ever, even in areas which enjoy high rainfall. Infact these regions could be the best models for testing the feasibility of water security at affordable costs by DRWH. Fortunately the government is also supportive of propagation of DRWH. Water harvesting has been recognised as one of the appropriate technologies useful in different parts of the country (Ref. `Directory of Rural Technology', Volume 3, 1989. Drinking Water. Council for Advancement of People's Action and Rural Technology, New Delhi). A number of NGO's have shown interest in testing and propagating DRWH. In vies of this, the Indian partners of the current project on DRWH have taken up the study with special focus on Water Quality (WQ). In the subsequent chapters, the role of DRWH in international / national context is reviewed with focus on the WQ aspect.

References

1. *`Fresh Water for India's Children and Nature', Chapter 1 & 2, by WWF and UNICEF, 1995, by Ashok Nigam, Biksham Gujja, Jayanta Bandyopadhyay and Rupert Talbot.
2. `Dying Wisdom', 1997. A publication by Centre of Science and Environment, New Delhi. A. Aggarwal and S. Narain
3. `Water and Sanitation and Baseline Survey'. A consolidated Report, January, 1998. On behalf of Rajiv Gandhi National Drinking Water Mission, New Delhi and Indian Institute of Mass Communication.

P.S.* Much of the above chapter has been adapted from reference number 1.

SECTION II
ROLE OF WATER HARVESTING IN INDIA

Introduction

Water harvesting is the deliberate collection and storage of rain water that runs off a natural or man made catchment surface. Catchments include roof tops, compounds rocky surfaces or hill slopes or artificially prepared impervious/semi-impervious land surfaces. Storage, on the other hand, may be done into tanks, lined pits, small dams or in the sandy beds of seasonal rivers. The `stores' are filled during rainfalls. Water users are thus left with a fixed volume of water until the next rain comes. The amount of water harvested (collected and stored) depends on the frequency and intensity of rainfall, the catchment characteristics and water demand and how quickly and how much runoff occurs (or conversely, how easy it is for the water to infiltrate through the surface to recharge the aquifer).

Water harvesting has been of particular importance in the arid and semi-arid regions and remote isolated habitations and in difficult terrains, where it may often provide the only feasible solution for an improved water supply. Hence in India, traditionally water harvesting technique had sustained in certain areas . For example, the type of water harvesting systems in different kinds of terrain's have been documented by Agarwal (Dying Wisdom, 1997).

Unfortunately traditional water harvesting techniques have been severely eroded. Modern attempts to restore them must reckon with the causes of their decline. Also since in the modern context, the problem of water scarcity (in terms of availability of uncontaminated water source) has become more wide spread and varied, the technology must be examined in the different eco-regions of India. The government of India has focussed on regions which suffer water scarcity, shortages or stress at some time during the year. The following have been identified as the main parameters which should be considered:

◊ Precipitation and topography
◊ Hydrological characteristics
◊ Geo-hydrological characteristics
◊ Level of industrialization
◊ Intensity of irrigation
◊ Level of urbanization

While combinations of these parameters can generate a wide range of situations, some broad generalization may be made. For example, India can be divided into several eco-regions which represent a broad range of situations where water resources are a major concern. These are:

- Arid and semi-arid regions in the west
- Rain shadow of the Western Ghats
- Drought prone regions
- Coastal regions
- Mountains and highlands
- Plains of the Ganges river
- Deserts
- Urban and metropolitan areas
- Other regions with severe chemical pollution

Realising importance of water related issues, various state governments as well as CAPART (Council for Advancement of People's Action and Rural Technology) and RGNDWM (Rajiv Gandhi National Drinking water Mission) have supported different projects on watershed management and RWH. Specifically DRWH has also been supported.S.E.R.C.(Structural Engneering Research Centre) has played a leading role in developing ferrocement technology and propagating this for building appropriate storage tanks. They have developed and commercialised a variety of techniques such as (I) Skeletal cage technique ,(II) SERC segmental casting technique and (III) Sac mould tecnique.

Dr.P. C. Sharma of SERC has also conducted several training programmes in South East Asia and India. However most of the Water Harvesting systems created by then are in the hilly areas of North and North eastern states.

However this technology has not been tested to a significant extent. A few leading NGO's and researchers have in the last 2-3 years, put up WH units on an experimental basis.(See Section iii). Keeping this in view in the current project, special attention has been paid to Kerala state which has good amount of biennial rain so that precipitation will not be a restricting factor. Rainfall pattern in Kerala is examined in the background of rainfall in India.

Rainfall Pattern in India

India (Fig. 2.1) has very good rainfall averaging around 1050 mm though it fluctuates widely over the country. Regions can be broadly classified according to rainfall figures: Desert (0-100 mm), Semi-desert (100-250), Arid (250-500 mm) and Semi-arid (500-750 mm) and medium-high rain fall areas (1000-3000 mm).

The coastal areas of the states receive high rainfall, decreasing over the interiors. The entire west coast covering the coastal areas of Maharashtra, Karnataka and Kerala receives annual rainfall of the order of 2500 mm. The rainfall increases all along the western ghats to 4000 mm. The rainfall decreases rapidly on the lee side of the western ghats over the plateau areas on the eastern side to about 500 mm. As we move further eastwards, the annual rainfall increases again to about 1000 mm along the eastern coast of Tamil Nadu and Andhra Pradesh. The states of Orissa, MP and coastal area of south Gujarat receive annual rainfall in the order of 1500 mm whereas the interior portion of Gujarat receives about 750 mm further decreasing to 400 mm over the extreme west. India's desert and arid regions are characterized by low and highly variable annual rainfall, strong variations in rainfall through space and time, high temperature and evaporation levels.

Under the current project, DRWH systems will be tried in state of Kerala (Fig. 2.1) as representative of humid tropics receiving a high annual rainfall.Therefore the variation in rainfall within the state of Kerala is discussed in detail. Representative sites will be selected based on both, rainfall and other considerations, for DRWH implementation in this state.

Rainfall over Kerala

Kerala which lies at the extreme southern parts of the peninsula is one of the smallest states in India. Through out the year Kerala receives rainfall, though the major rainy season is the South West monsoon period from June to September. The annual and seasonal distribution of rainfall is described briefly.

Climatology

Kerala occupies the portion of the subcontinent bounded by Lat. 8^o-13^oN and long. 75^o-77^oE. By physical features Kerala is divided into 3 natural divisions: (i) the low lands consisting of coastal areas, (ii) the middle land and (iii) the high land or the forest area of the Western Ghats on the eastern side. The extreme southern parts of the Ghats run along the eastern boarder of Kerala around 560 km in length.

The climate of the state is typical tropical monsoon with seasonally excessive rainfall and hot summer. There are basically 4 seasons (i) March to May the summer or pre-monsoon season, (ii) June to September the south west monsoon season (iii) October to December north east monsoon and (iv) January-February the winter season.

Rainfall

The annual rainfall in the state varies from 3800 mm in the north to 1800 mm in the extreme south. The annual average rainfall for Kerala is 3070 mm and the departure from this in different years is shown.(Table 2.1).

Seasonal Rainfall: South West Monsoon

The major rainfall season for Kerala is the south west monsoon period from June to September. The normal date of onset of south west monsoon is 1st June. During this period the average rainfall expected is 213o mm (Table 2.2) which constitutes 70% of the annual rainfall. This also varies from north to south, the variation being 85% in the north to 54% in south. The district wise normal rainfall during south west monsoon period is given in the tables (2.21-2.24). The captions in the tables are derived from the names of the district.

It can be seen that lowest rainfall during south west monsoon is being received in Thiruvananthapuram (TRV) district while the highest rainfall is in Wayanad (WYD) district. The annual highest average rainfall is in Kozhikode (KZK) district. The heavy falls during south west monsoon is due to the monsoon depressions which form over Bay of Bengal and Arabian Sea.

Next to the south west monsoon, the other principal rainy season is the northeast monsoon period which starts from October and ends with December. During this period Kerala receives 16% of its annual rainfall, i.e., 500 mm and there is a reversal in the order of rainfall activity from north to south. When the southern district receives around 600 mm of rain, only 350 mm of rain is received in the northern districts during this period.

During the summer period, i.e. March, April and May, Kerala receives 40 cm of rain which is 13% of the annual rainfall. This is mostly due to thuderstorm activity which is purely a local phenomenon.In winter season, i.e., January-February, only 1% of the annual rainfall is received. This amounts to only 3 cm of rain.

The state receives an annual rainfall of 307 cm which is much above the average rainfall (110 cm) for the entire country 86% of the total rain is being received during the two monsoon seasons, i.e., June to December. It may be noted that this rain water is the main source for the next 6 months, i.e., January-May for the different kinds of use in various activities. Any failure in the southwest monsoon or northeast monsoon will result in scarcity of water. This will also affect the availability of drinking water, electricity production and agriculture (see deviations in annual rainfall, Table 2.2). All efforts should therefore be made to plan and manage the use of water with utmost care so that even when the monsoon fails, water scarcity is not felt. Collection of rain water during the rainy season both for drinking and other purposes would hence be most useful.

In terms of water security, it may be that among the 35 Meteorological Sub-divisions in India, Kerala receives the maximum annual rainfall. Considering the area and population, around thirteen thousand liters of water is available per head per day out of which perhaps only one or two percent is sufficient for meeting the daily needs of a person. Thus water security in terms of quantity especially in the high rainfall areas of Kerala, is very good. This state is hence highly suitable for testing DRWH in terms of economic viability and water quality visa-vis other alternatives for at priding water.

Table 2.1: All India - Annual rainfall (cm) departure

1976	1977	1978	1979	1980	1981	1982	1983	1984	1985
0	1	+10	-10	-3	+4	-25	-9	-14	-16
1986	1987	1988	1989	1990	1991	1992	1993	1994	1995
-26	-24	-13	-13	-14	+1	+9	-8	+11	-7

Table 2.2: India State by State rainfall data

2.21: Southwest monsoon district wise rainfall (cm)

TRV	KLM	PTA	ALP	KTM	IDK	EKM	TSR	PLK	MLP	KZK	WYD	KNR	KSG
96	140	178	179	202	232	222	223	162	209	276	292	279	296

2.22: Northeast monsoon district wise rainfall (cm)

TRV	KLM	PTA	ALP	KTM	IDK	EKM	TSR	PLK	MLP	KZK	WYD	KNR	KSG
53	62	68	62	57	59	53	47	41	48	48	35	35	32

2.23: Summer (Pre-monsoon) district wise rainfall (cm)

TRV	KLM	PTA	ALP	KTM	IDK	EKM	TSR	PLK	MLP	KZK	WYD	KNR	KSG
39	48	61	50	49	45	49	41	27	33	41	31	32	29

2.24: Normal annual rainfall district wise (cm)

TRV	KLM	PTA	ALP	KTM	IDK	EKM	TSR	PLK	MLP	KZK	WYD	KNR	KSG
192	256	313	296	313	338	327	313	233	291	367	360	347	358

References

1. Proceedings “National Seminar on Rural Water Supply & Sanitation” edited by K. Pushpangadan. Sponsored by RGNDWM and organised by CDS, June 20-22, 1996. “Rainfall over Kerala”,K.K.Kunju Kunju Kutty, pp 163.

2. “Handbook on rainwater harvesting”. Rajiv Gandhi National Drinking Water Mission, 1998 (In press)

SECTION III

`DRWH' AND WATER QUALITY: A DESK REVIEW

Background

Effectively a modern DRWH system has to take into consideration 4 aspects:

i. Technology – Maximising efficiency of collection and storage at minimal cost.
ii. Water quantity/water security.
iii. Water quality to meet the current standards for drinking water.
iv. Attitude of people, policy makers, planners/administrators.

Under the EC project, all the above are being studied each aspect being led by one of the partners. A study on WQM is lead by I.I.T., Delhi, with support from the University of Warwick.

Water quality is a very important issue. According to WHO, 80% of diseases are caused due to contaminated water. The major contaminants may be classified into biological and non-biological. The diseases related to some of these are listed in the table 3.1 and 3.2

The water quality standards recommended for drinking water by WHO and other similar bodies are stringent. The maximum permissible limits of various physico-chemical parameters defining water quality (WQ) are given in table 3.3-3.6 as prescribed by the Indian RGNDWM and WHO. Each country while generally following international standards may provide for slight deviations depending upon the agro-climate, availability of water resources, socio-cultural practices, water conservation rates etc.

In the developing countries biological contaminants arising from water pollution by faecal waste and excreta of various animals are prevalent and is a major cause for concern. On the other hand a variety of metal ions which are highly toxic originate from improper handling and discharging of industrial effluents. Contamination by salts, fluoride and arsenic of geo-chemical origin also occur due to changing pattern of water use, resulting from over exploitation of water resources. Pesticides and other agro-chemicals originate from adoption of intensive agriculture practices for high yielding varieties of crops. Fortunately in `RWH', by the very nature of water collection process, many of the contaminants are not expected to be present in the water. For example ,if water is collected directly from roof and stored above ground, bacterial contamination should be minimal or absent unless the roof is accessible to human and animals. Bird droppings can be an issue and have to be guarded against by not having perches such as trees, in the vicinity. Also storage under anaerobic condition is expected to check the proliferation of bacteria due to lack of air and nutrients, even if it has been introduced from the roof. Problems of bacterial contamination may arise in ground water storage if there are cracks in the lining and contaminated water leaks into it.

As for chemical contaminants, these may arise from a variety of materials with which the rain water comes into contact starting from the atmosphere. Rain water could dissolve gases and wash off chemicals from contacting dust particles and roof materials. It could dissolve the metals or derived chemicals. Elution of chemicals from the walls is also possible from the storage tank.

The Global Scenario

A review of DRWH literature reveals that out of the large number of papers published, only a few have reported on `WQ'. In the traditional system, generally WQM is not done systematically WQ seems to be accepted as long as the water doesn't cause any diseases and the looks and taste are satisfactory. In the modern context however, the physical parameters are measured in terms of pH, turbidity, colour, odour and TDS. In addition tests are made to quantify various biological and chemical contaminants. The available literature related to WQM from different continents and sub-continents is summarised below.

Africa

- J.E. Gould[3.1] has discussed bacteriological analysis (total coliform, faecal coliform and faecal streptococci) from roof tank water. Accepted water qualtiy standards of Botswana is also tabulated. Generally high quality of properly stored rainwater is seen.

  Periodic chlorination is the most economic solution as suggested by the author. However the factors which will determine whether a water source is used or not are more likely to be related to taste, colour and odour, rather than necessarily directly to quality as stated in the paper.
- Gould and McPherson[3.2] have described bacteriological analysis of water samples from 13 roof tanks and 8 ground catchment tanks in Botswana. The results show that rainwater collected from corrugated iron roofs and stored in covered tanks is of high quality compared with traditional water sources. Water from roof catchment systems in Botswana presents a serious health hazard.
- Mayo and Mashauri[3.3] have given the bacteriological (total and faecal coliform and faecal streptococci), chemical (pH and total hardness) and physical (turbidity & colour) analyses of water samples from rainwater cistern system at the University of Dar es Salaam in Tanzania between October, 1988 and December, 1989. The results showed that 86% of samples were free from faecal coliform. However, faecal streptococci were obtained in 53% of the samples and 45% of the samples tested for total coliforms were positive. About 54% of the consumers raised objections over the taste of water. The pH range was found out to be 9.3-11.7 which is above standard limits.
- Otieno[3.4], Kenya has established from a study that except for the initial rainfall, the quality of rainwater is quite high, comparing favourably with river waters. (He has tabulated comparison of rainwater from roof catchment with river water and WHO standards). Guidelines for prevention of cisterns have been described and the need for maintaining quality of the stored water is emphasized.
- Bambrah and Haq[3.5] have discussed the suitability of using untreated rainwater for human comsumption in Kenya. They have reviewed existing literature on rainwater quality in their country. Guidelines for drinking water quality and various physical and chemical treatments have been described for disinfecting the stored water.

Australia

- Ghafouri and Phillips[3.6] have tabulated water quality guidelines by NSW recycling coordination committee. WQ of stormwater from urban catchments has been given. They have mentioned that roof water is seen as one of the possible sources of collecting rain water.
- Most extensive study on quality aspects of water stored in domestic rainwater tanks has been given by Fuller et al[3.7]. For South Australia. Water samples from three different areas (Vineyard and Orchard areas: 7 cities), industrial areas: 4 cities, and residential areas: 2 cities) were collected which reflected conditions in water stored in domestic rain

water tanks through South Australia. Galvanised iron tanks within the range of 10,000 to 25,000 liter with closed tops were selected. Tanks which had catchments of unpainted galvanised iron were chosen. Also householders were aksed to answer a series of question regarding use and maintenance of their tanks. Microbiogiological parameters, heavy metals (Pb, Zinc, Cd), pesticides and other physico-chemical tests (temperature, pH, suspended solids, total dissolved solids and salinity). Results of the study are summarised:

i) Coliform bacteria: coliform bacteria were present in 12 of 41 tanks, up to 500 coliforms/100 ml were recorded.

ii) E. coli: E. coli was detected in 6 tanks 15% of 41 tanks levels up to 220 E. coli/100 ml were recorded.

iii) Plate counts gave an indication of the general level of microbiological contamination of water. Plate counts in most rainwater tanks were in excess of 1000/ml.

iv) Heavy metals:
   - Cadmium: One of the tanks reported relatively high cadmium concentration (0.018 mg/l). This could be a sampling error or contamination caused by an isolated event.
   - Lead: Concentrations of lead in rainwater from tanks in Port Pirie were significantly higher (0.061 & 0.072 mg/l) than other sites. This could be a result of dust from surroundings country sides washed from roof tops with each rainfall.
   - Zinc: Zinc concentrations were found to be excess of 15 mg/l

v) Pesticides were not detected in the majority of samples.

vi) Suspended solids: Concentrations were negligible in all samples.

vii) pH: range of pH values was 6.1 to 9.2 low. pH values can accelerate corrosion problems in domestic appliances while high pH is an indication of undesirable biological activity in the tank.

viii) T.D.S.: Only samples taken from 2 rainwater tanks had T.D.S. concentrations in excess of 100 mg/ml (caused by sea spray).

Europe

- Per Jacobsen[3.8] has tabulated concentrations of lead (0.1 mg/l) and Zinc (0.1-1.00 mg/l) exceeding the standards for drinking water in Denmark. Lead, Zinc, Cadmium and copper were estimated.
- Wilhelm Meemken[3.9] has tabulated quality of rain water collected from roofs in Germany. Chemical parameters included (Fe, Mn, Cu, Pb, Zn, pH, Ca, Mg, Na, K, NH_4^+, SO_4^{2-}, Cl^-, NO_3^-, NO_2^- and electrical conductivity. The table showed the rainwater collected from roofs could be used for flushing toilets, washing cloths and watering plants without special treatment.

Asia

- Appan[3.10] has described roof water collection systems in some southeast Asian countries (Thailand, Indonesia, The Philippines, Malaysia and Singapore).

  Thailand: Physico-chemical parameters (pH, colour, turbidity, iron, manganese, lead and cadmium) and bacteriological analysis (T. coli, F. coli) has been tabulated. In terms of physico-chemical parameters, more than 83% of the samples were satisfactory except for about 40% of the samples exceeding the allowable limits of lead. In terms of E. coli, more than about 76% of the samples had values exceeding the WHO guideline standards.

In another series of bacteriological tests conducted in three locations in Khon Kaen, 709 water samples were collected from tiled roofs and gutters, containers located in homes, jars and the point of consumption. In terms of the three coli groups, only 10% to 67% of the samples were within the WHO guideline values. Samples other than those collected from the container showed that, due to an FC/FS ratio of less than one, 79% to 82% of the contamination could have emanated from animal droppings. In the same study, only manganese (in 2-20% of the samples) and zinc (in 4-26% of samples) did not meet the guideline levels.

Indonesia: Although RWCS seem to have been practiced with extensive experimentation using different types of materials, there is practically no available information on the quality of collected water. It has been observed that some measures such as fish rearing within the tanks have been practised so as to keep the water clean.

The Philippines: In a study spread over a period of one year in three villages in the Province of Capiz involving 25 ferrocement tanks, it was shown that not less than 24% of the samples had T. coli exceedings the WHO guidelines values.

Malaysia: The quality of rainwater and roof runoff has been monitored and 72 samples were collected from two types of roofs in West Malaysia. The range of turbidity, lead and F. coli values far exceeded the WHO guidelines values. The pH value of rainwater also has a tendency to lie towards the lower range of the guideline values.

Singapore: Roof water was monitored from a high rise building in the Nanyang Technological University for six years from May 1989. The values appear to be acceptable in all the physico-chemical parameters except pH which is quite low. T. coli and F. coli values also exceed the guideline values. Earlier field investigations have also shown that, during January 1974 to July 1983, the range of pH in 11 monitoring stations distributed throughout Singapore was 4.8 to 5.5.

- Appan[3.11], in a case study on rainwater catchment systems has summarised data on quantity and quality of water for non-potable uses. Physico-chemical parameters (pH, colour, turbidity, salinity, TSS, TCC, TDS, oil and grease) of raw rainwater and treated water are tabulated. Rain water is effective for non-potable uses like flushing, cleaning, gardening etc.
- Xijing et al[3.12]. from China have analysed and assessed WQ of the catchment and storage rainwater physico-chemical testing result have been discussed. pH, Cu, Pb, Se, Zn, K, nitrites, total alkalinity have higher values in the rainwater contained in concrete water cellars with cement or grey tile catchment surfaces. The indices of As, Fe, Ca^{2+}, Mg^{2+}, total hardness have higher values in the rainwater contained in soil water cellars. Total coliform become fewer in rainwater contained in concrete cellars with tile surfaces. The author emphasized that the problem could be solved through changing the building materials of catchment surfaces and water cellars, improving hygienic conditions and taking some effective disinfectant measures.
- Kita and Kitamura[3.13] from Japan have described fluctuation in the quality of rainwater stored in container during storage. The results are:
  - pH & COD remains constant
  - NH_4 first increases then decreases and finally remains O mg/l
  - Color and turbidity remain constant
  - Results of coliform group of bacteria were positive throughout the periods.

- Feasibility of using roof rainwater catchment systems in West Bank (Palestine) as supplementary water source have been investigated by Sharekh[3.14] water quality issues and discussed. Physico-chemical and bacteriological test results tabulated have been (pH, Ec, TDH, Ca, Mg, Na, K, Cl, CO_3^-, HCO_3^-). It was found, rainwater stored in cisterns was used for drinking and domestic purposes. Level of total coliform contamination were found 27%>0 coliform. Concentration of major constituents well within the prescribed limits.
- Metals (Fe, Zn, Pb) concentration greatly exceeded the standards in a study of rainwater collected from roof tops in Japan (Tachi-kawa).
  In Chiyuda city (Japan), quality of 1st flash of rainwater after a 11 day dry spell have been investigated physico-chemical (Fe, Cd, Cr, Cu, Mn, Ni, Pb, Zn) parameters tabulated initial rainfall upto 1.5 mm shows that the voluem of pollutants had accumulated on the roof for a 11 day dry spell.
  Nariyuki Fukano[3.15] (Japan) has discussed, prevention of mosquito breeding in rainwater storage tanks. As for rainwater storage tanks installed in large structures, mosquito breeding can be prevented by installing insect proof nets or other insect proof devices in vent routes. In home use tanks, water is always consumed and replenished, so rainwater does not stay in storage tanks for a long time. Therefore, even if insect eggs are mixed into tank water or a misquito enters into the tank and spawn eggs, there is virtually no likelihood of mosquito proliferation. Mosquitoes seem to breed easily in rainwater drainage pits that drain rainwater from roofs and the ground surface into sewers because many leaves tend to accumulate producing a condition conducive to mosquito breeding. To change conventional rainwater drainage pits to ones made to permeable material is a good solution to avoid mosquito problems, even if some cost is involved because the rainwater in permeable rainwater drainage pits infiltrates into the grouns so mosquito larvae cannot breed there.
- Water shortage in various regions in Japan is taken care by utilizing rainwater as an alternative as reported by Kitamura et al[3.16]. Although rainwater quality is acceptable, variations take place during storage.
  - Physico-chemical parameters studied for rainwater and ground water
  - No bacteriological studies were reported.
  - Stored rainwater pH were almost constant except for the last 3 weeks duration.
  - Turbidity varied during first two weeks
  - Ammonia nitrogen varied decreasing storage, which was probably caused by the bacterial activities. Bird droppings and insect carcasses were stored together with collected rainwater. It was a sanitary problem.
- When the alternative water source which had normal fluoride levels such as rainwater is adopted, the water borne disease/fluoride endemics can be controlled or possibly eradicated as reported by Bo-ling[3.17] in China. Investigation demonstrated the highest fluoride contents were in the well water. 24-40 mg/l while the lowest was in the rainwater: 0.78 mg/l.
- Physico-chemical quality along with bacteriological quality has been tanlated for 3 selected sites in Sri Lanka by Heijen and Mansur[3.18]. Color/turbidity are a bit higher than expected. Dirt on the roof and the non-application of a first flush device or a simple filter were the likely causes. E. coli count was consistant low, though 21% of samples shown 100 colonies/100 ml of total coliform.
- Hussain et al[3.19]. in the paper described RWCS, storage and purification practices in 3 locations. Also it described materials used in gutters, rainfall, method of collection,

maintenance and water use. Periodic cleaning of tanks and proper closure of the month of the storage container was desired. Water quality control was not often practiced. Few villagers tried to maintain quality of water due to fear of water borne disease. Biological, chemical, filtration and sedimentation were some of the methods for water quality control.

- National Water Supply and Drainage Board, Sri Lanka[3.20] has tabulated all chemical and bacteriological analysis (total coliform, E. coli and faecal striptococci) report. No chemical pollution has been found, bacterial contamination was there and the board recommended boiling of stored rainwater before consumption.

India

- Sh. P.K. Sivanandan[3.21] has reported chemical analysis of water sample from open wells in Adimalathura area, Kerala (Jan.-Feb., 99). 11 samples from all around villages were collected and chemical testing was done. Parameters studied were pH, EC, D.O., chloride, total hardness, Ca hardness, Mg, total alkalinity, bicarbonates and carbonates). Results indicated that chemical quality of water had potable status.
- Mitraniketan[3.22] (Kerala) in a project report tabulated rainwater analysis. Chemical parameters included pH, alklanity, chloride, iron, nitrate, nitrates, sulphate, total solids and hardness. Bacterial examination was also done. In a span from 10.6.98 to 26.7.98, the results revealed that the stored water had potable status.
- Ruchi[3.23] (Solan, Himachal Pradesh) stated that water quality was tested. Initially and was suitable for drinking purpose.

Conclusions

The literature on WQ generally has reported on pH , EC, turbidity, total hardness etc. However correlation of these with details on technology of water (type of roofing/guttering) and storage are missing. The methodology for valuation of WQ is also missing. Besides quality of water one of the other concern is insect breeding in DRWH. Mosquito breeding would be a health hazard. Hardly any data related to this is available. To collect more authentic details of WQ, a survey format has been designed by IIT Delhi and will be administered in different parts of the country. Based on limited data is seen that generally metal ions and other chemical contaminants are reported. The survey is designed to get data for correlating WQ with DRWH design parameters based on which optional designs could emerge.

In Indian context, list of key organiszations/institutions who are working in DRWH area have been collected (Appendix 3). Only a few of these however have done WQM. In the next phase of the work, it is proposed to cover WQ of all organisations who are working in Kerala and a few from other states for comparison.

References

Africa

3.1 J.E. Gould. Rain water catchment possibilities for Botswana. April, 84, pp 10-12. BTC (Botswana Technology Centre).

3.2 J.E. Gould and H.J. McPherson. Bacteriological quality of rain water in roof and ground catchments system in Botswana. Water International 1987, 12, pp. 135-8.

3.3 A.W. Mayo & D.A. Mashauri. Rainwater harvesting for domestic use in Tanzania. A case study. Universityof Dar es salaam staff houses. Water International 16 (1991), pp 2-8.

3.4 F.O. Otieno. Quality issues in rainwater collection. Raindrop, June, 94.

3.5 Dr. G.K. Bambrah and Ms. S. Haq. Quality issues in rain water harvesting in Kenya. Proceedings of 8th International Conference on RWCS, April 25-29, 1997, pp 547-553.

Australia

3.6 R.A. Ghafouri & Dr. B.C. Philips. Urban storm water re-use opportunities and constraints. Proceedings of 8th Int. Conference on RWCS, April 25-29, 1997, pp 554-565.

3.7 Dr. C.O. Fuller, Mr. R.P. Walters and Mr. T.J. Martin. Domestic rainwater tanks working party, March, 81. Quality aspects of water stored in domestic rainwater tanks (a preliminary study).

Europe

3.8 Per Jacobsen. Metals in Rainwater in Denmark. Tokyo International Rainwater Utilization Conference, Sumida City, Summer, 94, page 9.

3.9 Wilhelm Meemken. Quality examination of rainwater collected from roofs and stored in tanks. Tokyo International Rainwater Utilization Conference, Sumida City, Summer, 94, page 9-10.

Asia

3.10 A. Appan, Roof water collection systems in some southeast Asian countries: Status and water quality levels. The Journal Royal Society of Health. October, 97, Vol. 117 No. 5, pages 319-323.

3.11 A. Appan, The utilization of rainfall in airports for non-potable uses. Proceedings of the 6th Int. Conference on RWCS. Nairobi, Kenya, 1-6 August, 1993.

3.12 Yang Xijing, Zhu Hong and Zhang Xingyou. The water quality problems of rainwater utilization in the arid regions of Northwest China. 7th International RWCS, Conference, June 21-25, 1995, Beijing, China, page 1-14.

3.13 Ichiro Kita and Kunihiko Kitamura. Fluctuation of the quality of container stored rainwater during storage. 7th International RWCS, Conference, June 21-25, 1995, Beijing, China, page 27-32.

3.14 M.S. Abu Sharekh. Rain water roof catchment systems for domestic water supply in south of West Bank. 7th International RWCS, Conference, June 21-25, 1995, Beijing, China, page 65-90.

3.15 Tokyo International Rain Water Utilization Conference, Sumida city, Summer, 94, page 9-11.

- Metals in rainwater in Tachikawa Saiwai-Cho Housing Complex
- Quality of first flash of rainwater (Aug.-Sept., 86).
- Prevention of mosquito breeding.

3.16 Kunihiko Kitamura, Ichiro Kita and Isao Minami. The effects of storage on rain water quality. Proceedings of the 8th International Conference on RWCS, April 25-29, 1997, Tehran, Iran, page 590-595.

3.17 Bo-Ling, KDF-55 processing medium to disinfect eistern water. International Symposium and 2nd Chinese National Conference on Rainwater Utilization, Sept. 8-12, 1998, page 79-80.

3.18 Han Heignen and U. Mansoor. Symposium on rainwater harvesting for water security, Feb. 28, 1998. “Rainwater harvesting in the community water supply and sanitation project”.

3.19 MD Daulat Hussain, MD. Ahiduzzaman, Thomas Rozario. Rain water storage, purification and distribution in selected rural areas in Bangladesh. Proceedings of the 8th International Conference on Rainwater Catchment Systems, April 25-29, 1997, Tehran, Iran, page 648-653.

3.20 National Water Supply and Drainage Board, Sri Lanka. Chemical and bacteriological analysis report (15.10.98, 14.12.98 and 16.12.98).

India

3.21 A report by Sh. P.K. Sivanandan (Kerala governemtn) on water quality of open wells in Adimalathura, Kerala. (Jan-Feb.'99)

3.22 All India Coordinated project report, sponsored by CAPART, Science and Society Division, Mitraniketan, Kerala.

3.23 Ruchi, Solan (Himachal Pradesh) in a letter dated 11.3.99 to I.I.T., Delhi, described water quality issues carried out by them in stored rain water tanks.

Table 3.1: Water based disease transmission and preventive strategies

Classification	Transmission	Examples	Preventive strategies
Water-borne (water-borne diseases can also be water-washed	Disease is transmitted by ingestion	• Diarrhoeas (e.g. cholera) • Enteric fevers (e.g. typhoid) • Hepatitis A	• Improve quality of drinking water • Prevent casual use of other unimproved sources • Improve sanitation
Water-washed (water scarce)	Transmission is reduced with an increase in water quantity: • Infections of the intestinal tract • Skin or eye infections • Infections caused by lice or mites	• Diarrhoeas (e.g. amoebic dysentery) • Trachoma • Scabies	• Increase water quantity • Improve accessibility and reliability of domestic water supply • Improve hygiene • Improve sanitation
Water-based	The pathogen spends part of its life cycle in an animal which is water-based. The pathogen is transmitted by ingestion or by penetration of the skin.	• Guinea worm • Schistosomiasis	• Decrease need for contact with infected water • Control vector host populations • Improve quality of the water (for some types) • Improve sanitation (for some types)
Insect-vector	Spread by insects that breed or bite near water	• Malaria • River blindness	• Improve surface-water management • Destroy insects' breeding sites • Decrease need to visit breeding sites of insects • Use mosquito netting • Use insecticides

Ref.: Waterlines. "Technical Brief No. 52: Water: Quality or quantity", Vol. 15, No. 4, April 1997

Table 3.2: Pathogenic micro-organisms in water

Bacteria	Disease
Salmonella typhi	Typhoid fever
Salmonella enteritidis	Gastroenteritis
Shigella dysenteriae	Dysentery
Vibrio cholerae	Cholera
Escherichia coli	Gastroenteritis
Leptospira icterohaemorrhzgicae	Leptospirosis (Weil's disease)
Mycobacterium tuberculosis	Tuberculosis
Legionella pneumophila	Legionellosis (Legionnaives disease)
Viruses	
Hepatitis A virus	Infectious hepatitis
Polio virus	Infantile paralysis, poliomyelitis
Enteroviruses	Gastroenteritis
Protozoa	
Giarodia lamblia	Giardiosis
Entamoeba histolytica	Amoebioses

Reference: Otto F. Joklik, Potabilization of Rainwater (Austria). 7th International Rainwater Catchment System Conference, June 21-25, 1995, China, page 33-47.

Table 3.3 : WHO Guidelines for Drinking Water (1984)

Parameter	Guideline value*
Colour (TCU)	15
Turbidity (NTU)	5
PH	6.5-8.5
Hardness (as $CaCO_3$)	500
Iron	0.3
Manganese	0.3
Sulphate	400
Chloride	250
Total Dissolved Solids	1000
Nitrate	10
Arsenic	0.05
Cadmium	0.005
Chromium	0.05
Cyanide	0.1
Fluoride	1.5
Lead	0.05
Mercury	0.001
Selenium	0.01
Zinc	5
F. coli/100 ml	0
T. coli/100 ml	0

* Note: All units, except pH, in mg/l unless stated otherwise

Table 3.4: WHO Guidelines for Drinking Water Constituents
Microbiological and Biological Quality

Organism	**Unit**	**Guideline value**	**Remarks**
I. MICROBIOLOGICAL QUALITY			
A. Piped water supplies			
A1 Treated water entering the distribution system			
Faecal coliforms	Number/100 ml	0	Turbidity 1 NTU: for disinfection with chlorine pH preferably 8.0: free chlorine residual 0.2-0.5 mg/litre following 30 minutes contact
A2 Untreated water entering the distribution system			
Faecal coliforms	Number/100 ml	0	
Coliform organisms	Number/100 ml	0	In 98% of samples examined throughout the year in the case of large supplies when sufficient samples are examined
Coliform organisms	Number/100 ml	3	In an occasional sample, but not in consecutive samples
A3 Water in the distribution system			
Faecal coliforms	Number/100 ml	0	
Coliform organisms	Number/100 ml	0	In 95% samples examined throughout the year in the case of large supplies when sufficient samples are examined.
Coliform organisms	Number/100 ml	3	In an occasional sample, but not in consecutive samples
B. Unpiped water supplies			
Faecal coliforms	Number/100 ml	0	
Coliform organisms	Number/100 ml	0	Should not occur repeatedly if occurrence is frequent and if sanitary protection cannot be improved an alternative source must be found if possible.
C. Bottled drinking water			
Faecal coliforms	Number/100 ml	0	Source should be free from faecal contamination
Coliform organisms	Number/100 ml	0	
D. Emergency water supplies			
Faecal coliforms failure	Number/100 ml	0	Advise public to boil water in case of to meet guideline values
Coliform organisms	Number/100 ml	0	
Enteric viruses			
II. BIOLOGICAL QUALITY			
Protozoa (pathogenic)		Do	
Helminths (pathogenic)			Do
Free living organisms (algae others)		Do	

Table 3.5: Tentative Limits for Toxic Substances in Drinking Water

Substance	Upper limit of concentration (Mg/l)	Methods of estimation
Arsenic (as As)	0.05	• Polarographic estimation • Atomic absorption spectrophotometric method • Use of Gutzeit generator
Cadmium (as Cd)	0.01	Dithizone method
Cyanide (as CN)	0.05	Can be estimated by means of a number of methods of which the following are generally in use and are equally satisfactory. • Titration with silver nitrate in dilute ammoniacal solution using diphenyl carbazide as an absorption indicator • Colorimetric method: Conversion of cyanide to either cyanogen chloride or cyanogen bromide & coupling with a suitable aromatic amino compound, such as dimodone, pyrozikine or sulfanilie acid • Colorimetric method: Yellow ammonium sulfide converts cyanide to thiocyanate in slightly alkaline solution: the thiocyanate reacts quantitatively with ferrie iron to form coloured ferric thiocyanate
Lead (as Pb)	0.1	• Polarographic estimation • Atomic absorption spectrophotometric method • Calorimetric methods
Mercury (as Hg)	0.001	• Neutron activation analysis • Atomic absorption
Selenium (as Se)	0.01	Colorimetric method using gum arabic solution, hydroxylamine hydrochloride sulphur dioxide and concentrated hydrobromic acid

Table 3.6: Specifications for Drinking Water (RGNDWM)

S.No.	Characteristics	Maximum permissible limits	Adverse effects beyond permissible limits	Alternative extended limits if no toxicity confirmed
1.	Colour (Hazen units)	10	Consumer acceptance decreases	50
2.	Odor	Unobjectionable	-	-
3.	Taste	Agreeable	-	-
4.	Turbidity (NTU)	10	Consumer acceptance decreases	25
5.	T.D.S. (mg/1)	500	• Palatability decreases • May cause g-i irritations	3000 (WHO limits: 4500)
6.	PH value	6.5 to 8.5	Mucous membrane affect	9.2
7.	Total hardness as $CaCO_3$ (mg/1)	300	Encrustation and adverse effect on domestic use	600
8.	Calcium as Ca (mg/1)	75	Do	200
9.	Magnesium as Mg (mg/1)	30	Do	100
10.	Copper as Cu (mg/1)	0.05	• Astrigent taste • Discoloration & corrosion of metalc parts	1.5
11.	Iron as Fe (mg/1)	0.3	• Taste/appearamce affect • Promotes iron bacteria	1.0
12.	Manganese as Mn (mg/1)	0.1	Taste/appearance affected	0.5
13.	Chlorides as Cl (mg/1)	250	• Taste/palatability reduces • Corrosion increases	1000
14.	Sulphates as SO (mg/1)	150	Gastro-intestinal irritations when Mg or Na present	400 (provided Mg does not exceed 30)
15.	Nitrate as NO (mg/1)	45	Methnaemoglobinemia takes place	No relaxation
16.	Fluoride as F (mg/1)	0.6-1.2	• Low fluoride are linked with dental care • Above 1.5 fluorosis	1.5
17.	Phenolic compounds as C H OH (mg/1)	0.001	Objectionable taste and odour	0.002
18.	Mercury as Hg (mg/1)	0.001	Toxicity increases	No relaxation
19.	Cadmium as Cd (mg/1)	0.01	Do	Do
20.	Selenium as Se (mg/1)	0.01	Do	Do
21.	Arsenic as As (mg/1)	0.05	Do	Do
22.	Cyanide as CN (mg/1)	0.05	Water becomes toxic	Do
23.	Lead as Pb (mg/1)	0.1	Do	Do
24.	Zinc as Zn (mg/1)	5	• Astrigent taste • Opalescence	15
25.	Anionic detergents as MBAS (mg/1)	0.2	Frothing in water	1
26.	Chromium as Cr^{6+} (mg/1)	0.05	Carcinogenic	No relexation
27.	Polynuclear aromatic hydro-carbons as PAH (μ g/l)	-	Do	-
28.	Mineral oil (mg/1)	0.01	Undesirable taste and odour	0.03
29.	Residual free chlorine (mg/1)	0.2 (minimum)	-	0.5 For protection against viral infectn

<u>Source</u>: Indian Standard Specifications for Drinking Water, IS: 10500, 1983

Appendix - 3

Key Institutions/Organisations Working on `DRWH' in India

1) Uttarakhand Jan Jagriti Sansthan
Khadi, P.O. Jajal, Distt. Tehri Garhwal, U.P.-249175, India

2) Dr. SunderLal, Director
Social Centre for Rural Initiative & Advancement
Khori-123101, Distt. Rewari, Haryana, India
Ph: 01274-86625, 86652

3) Rural Centre for Human Interests (RUCHI)
Technology Complex, Bandh, P.O. Bhaguri Via Patla
Distt. Solan, Himachal Pradesh-173233, India
Ph: 91-1792-82454, 83732; Telefax: 91-1792-72649, 82516

4) Society for Integral Development Action
Koovally, Distt. Kottayam, Kerala-686518, India

5) Comprehensive Area Development Service
5/2, R.B.C. Road, P.O. Naihati, Distt. 24 Parganas-North, West Bengal-743165, India
Ph: 581-3073, 3341

6) Sh. Ramani, Dy. Manager (MM)
ONGC
Southern Regional Business Centre, CMDA Building, 9th Floor (E)
8, Gandhi Irwin Road, Egmore, Chennai-600008, India

7) Dr. Y.M. Kool/B.R. Singh/S.K. Pande
Studies on some effective rainwater harvesting systems
Plateu, Madhya Pradesh, India

8) Water & Land Management Institute
P.B.-538, Ravishankar Nagar, Bhopal-462016, M.P., India

9) Sh. R.N. Saran, Dean
College of Agriculture
Indore-452001, M.P., India

10) Mr. Rakesh Agarwal
`SAHAYOG' (Sahayog Society for Participatory Rural Development)
Prem Kuti, Pokharkhali, Almora, U.P., India Ph: 91-5962-22389

11) Sh. N. Kamalamurra
Gandhi Gram Rural Institute (Deemed University)
Gandhi Gram-624302, Tamil Nadu, India (1992)
(Technology for Rural Development, Series-4, Rain Water Harvesting)

12) Elements of Rainwater Harvesting
Centre of Science for Villages
Magan Sangrahalaya, Wardha-442002, India (1992)

13) Chennai Metropolitan Water Supply & Sewerage Board
No. 1, Pumping Station Road
Chintradripet, Chennai-600002, India

14) Sh. Shiv Kumar, Agriculture Engineer
JRD Tata Institute
M.S. Swaminathan Research Foundation
III Cross, Taramani, Chennai-600113, India

15) Dr.(Mrs.) Nirmala Raghunath
Scientist `C'
Environment Studies Division
CWRDM, Kunnamangalam
Calicut-673571, Kerala

16) Mrs. Jesintha Prosper, Manager
Centre for Appropriate Technology
5, Chidambaranathan Colony
Ramavarmapuram, Nagar Coil-629001, Tamil Nadu, India

17) Mrs. Sarawam, Field Coordinator
Women Empowerment Programme (WEP)
KSSS Community Health Development Programme
Thirumalai Ashram Social Centre
Chunkankadai-629807, Tamil Nadu, India

18) Sh. Vinay Mahajan/Sh. Charul Bharwada
Garage, 65, Brahman Mitra Mandal Society
Ellisbridge, Ahmedabad-380006, India

Sandarbh Studies & Interventions, Ahmedabad
Drying Water Traditions: Traditional Water Systems- Their use, status and decline in Kutch, Gujarat

19) MITRANIKETAN
Velland, Thiruvananthapuram-695543, Kerala, India
Ph: 0472-882045, 882680, Fax: 0472-882015

20) Dr. P.C. Sharma, Head
Material Science Division, Structural Engineering Research Centre (SERC)
(A constituent unit of CSIR)
Sector-19, Central Govt., Enclave, Kamla Nehru Nagar, P.O. No. 10,
Ghaziabad-201001 (U.P.), India
Ph: 0575-721884 (O) 0575-793082, 793102 ®

21) Centre for Science and Environment (CSE)
41, Tughlakabad Institutional Area, New Delhi-110062
Ph: 6983394, 6986399; Fax: 91-11-6985879, 6980870

22) Mr. Mihir Bhatt, Director,
Disaster Mitigation Institute
411, Sakar Five, Near Natraj Cinema, Ashram Road, Ahmedabad-388009, India
Tel: 79-6586234/6583607; Fax: 79-6582962

SECTION – IV

WATER QUALITY MONITORING TECHNIQUES

Introduction

In the earlier chapter, parameters to be tested for WQM in DRWH were listed. Elaborate testing requires appropriate instrumentation and availability of a variety of chemicals. Further requisite skills may be called for. All this contributes to high cost of water testing. It may not be feasible to conduct such elaborate tests especially at the field levels, which may be located in remote locations. In view of this, a number of rapid testing methods have been evolved and standardized against the more elaborate testing procedures. Also in many countries, a set of tests are made available as a package in a compact field kit along with manual of instructions which can be followed easily by field level personnel. While less accurate than elaborate laboratory tests, these rapid tests are adequate to monitor whether the value of a given parameter exceeds acceptable limits. If rapid tests indicate unacceptable values for WQ, further tests and in depth studies can be recommended for immediate corrective action.

It may however be noted that basic principles involved in the rapid and elaborate tests may be the same. Hence the general principles and procedures for each of the WQ parameters are discussed below to help the user conduct the tests with an understanding of the processes involved. This will also help the user in taking proper precautions while conducting the tests.

A. TESTING OF CHEMICAL CONTAMINANTS

4.1) **Turbidity (Suspended solids)**

As the rain water falls on the roof, it may pick up fine dust particles. These will get washed down into the storage tank in the first flush. With suitable filters and appropriate mechanisms for rejecting the first flush, the water should be without suspensions. However in case the fine particles are of colloidal dimensions they will still persist and the water could thus appear turbid.

Turbidity can be measured by a Nepherometer.

Principle: This method is based on a comparison of the intensity of light scattered under the defined conditions with the intensity of light scattered by a standard reference suspension under the same conditions.

4.2) **Total Dissolved Solids (TDS)**

Water, a polar solvent has the capacity to dissolve ionic components from materials it contacts. In DRWH, as water contacts the roofs, gutters and inner walls of the storage structure, it may dissolve metallic salts and polar organic compounds if present in the environment.

Total dissolved solids can be measured by evaporating the water from an aliquot of water sample and weighing the residue.

Procedure: Measured volume of well mixed sample is filtered through glass fiber filter. It is washed with three successive 10 ml volumes of distilled water, complete drainage between washing is allowed and suction is continued for about 3 min after filtration is complete. Filtrate is then transferred to a weighed evaporating dish and is evaporated to dryness on a steam bath (if filtrate volume exceeds dish capacity successive portions are added to the same dish after evaporation). It is dried for at least 1 hour in an over at 180 ± 2^oC and cooled in a desiccator and weighed until a constant weight is obtained.

$$\text{Total dissolved solids in mg/1} = \frac{(A - B) \times 1000}{\text{Sample volume (ml)}}$$

Where,

A = weight of dried residue + dish (mg)

B = weight of dish (mg)

Ref. Richards (1954)

4.3) <u>Electrical Conductivity (EC)Measurements</u>

Electrical conductivity is commonly used for indicating the total concentration of ionic (salt) soluble constituents. It is closely related to the sum of the cations or anions as determined chemically and usually correlates closely with the total dissolved solids. It is a rapid and reasonably precise determination that does not alter or consume any of the sample. The apparatus for measuring electrical conductivity consists of an electrical resistance bridge and conductivity cell having electrodes coated with platinum black.

<u>Reagents</u>

Standard potassium chloride solutions: 0.7456 gm of dry reagent grade potassium chloride is dissolved in freshly prepared double distilled water and made to one liter. At 25^oC it gives an electrical conductivity (EC) of 1411.8×10^{-6} mhos/cm. The conductivity bridge is calibrated and cell constant is determined with the help of this solution.

<u>Procedure</u>: The measurement of EC (expressed in mmhos/cm) is adjusted for a known temperature (usually 25^oC) of the solution by setting up the knob provided for this purpose. The higher the salt concentration, the higher is the EC

Currently digital conductivity metres are also available. Just by dipping this in the water sample EC can be measured. The cost of an Indian meter of this kind is ~ Rs. 4,000. Many of the rapid testing kits have an EC meter incorporated which is calibrated to directly give the TDS.

4.4) pH

pH (the negative log 10 of hydrogen ion concentration in a solution) is a solution) is a useful parameter to indicate the presence of acidic or alkaline substances in water. The pH of water without contaminants is around 7 (neutral). If acids are present pH goes below 7 and with alkali it rises above 7. pH can be measured by colorimetric methods using various indicators. The indicators are chemical compounds which undergo change in color rapidly at a certain pH due to changes in their chemical structure. The indicators may be used in a solution and the colour change is seen using a colorimeter. For rapid tests the indicator may be mounted on paper strips.

For accurate measurement of pH, use of electrometric method employing hydrogen ion sensitive electrodes, in a pH meter is preferred.

<u>Apparatus</u>: There are number of makes and models of pH meters. Portable pH meters operated by battery are also available. The accuracy of pH measurement can vary from 0.01 to 0.1 depending on the make. Some pH meters employ two electrodes, an indicator glass

electrode, and a calomel reference electrode, while other may have a combined glass and reference electrodes. Most pH meters also have a temperature compensation systems to avoid the differences arising due to the different temperatures.

Procedure: The pH meter is calibrated using suitable buffers whose pH is known. The water sample is taken in a beaker and the glass electrode is dipped into it. Equilibrium between electrode and sample is established by stirring the sample to ensure, homogeneity and pH meter reading is read after dipping the electrode into sample for one minute.

4.5) **Estimation of Calcium and Magnesium Ions (Total Hardness)**

Hard water contains calcium and magnesium ions and estimation of these are important for determining the hardness of water. In the context of DRWH, water samples may contain these ions eluted from walls of storage tank if made of cement or lined with compounds containing calcium like lime or mortar.

Titration with EDTA (Ethylenediamine Tetra Acetate) is a common method for estimation of these ions.

Reagents

A. Sodium hydroxide (0.1 N)

B. Standard calcium solution : Weighed 1.0 gm of anhydrous calcium carbonate into a 500 ml erlenmeyer flask. A funnel was placed in the flask neck and 1+1 HCl is added until all $CaCO_3$ gets dissolved. The volume is made upto 1 liter.

C. Erichrome black T indicator – 0.5 gm of Erichrome black T in 100 g triethanolamine.2 drops of indicator is added to per 50 ml solution to be titrated.

D. EDTA (Versenate) solution (0.01 M) – 3.723 gm of disodium dihydrogen ethylene-diamine-tetra-acetate is dissolved in water and diluted to 1 liter. The solution is standardized ($CaCl_2$), using titration procedure given below using indicator (Erichrome black).

E. Buffer solution : 16.9 g ammonium chloride (NH_4Cl) is added in 143 ml conc. ammonium hydroxide. 1.25 g of magnesium salt of EDTA is then added and the volume is made to 250 ml with distilled water.

Procedure:

Titration of sample:

25 ml sample is diluted to 50 ml with distilled water. 1-2 drops of buffer solution are added (to give a pH of 10.0). 1-2 drops of indicator solution are added and the solution is titrated with standard EDTA solution, with continuous stirring till the end point is reached. (blue color)

Calculation

$$\text{Hardness as mg } CaCO_3/l = \frac{A \times B \times 100}{\text{ml sample}}$$

Where,

A = ml titration for sample

B = mg $CaCO_3$ equivalent to 1.00 ml EDTA titrant

A x B x 400.8

Calcium as mg Ca/l = ----------------------
ml sample

A = ml titrant for sample
B = mg $CaCO_3$ equivalent to 1 ml EDTA titrant at the calcium indicator end point

Magnesium as mg/l
Total hardness (as mg $CaCO_3$) -Calcium hardness (as mg $CaCO_3$/l) x 0.244

4.6) **Chloride**

Determination of chloride is important where water is brackish. Normally in DRWH, the water is not expected to have chloride. However since chlorine is also used for water treatment, the method of estimation of chloride is included herein. One of the procedures involves titration of the chloride containing sample with silver nitrate using chromate as indicator. The method is discussed below.

Reagents

a) Potassium chromate indicator solution: 50 gm K_2CrO_4 is dissolved in little distilled water. $AgNO_3$ solution was added until a definite red precipitate is formed. It is allowed to stand for 12 hrs. Then it is filtered and diluted to 1 liter with distilled water.
b) Standard silver nitrate titrate (0.0141 N): 2.395 gm $AgNO_3$ in distilled water is diluted to 1000 ml. It is standardized against 0.0141 N NaCl (1.0 ml = 500 gm Cl). It is stored in a brown bottle.
c) Standard sodium chloride (0.0141 N): 824.0 mg NaCl (dried at 140^oC) is dissolved in distilled water and diluted to 1000 ml (1.00 ml = 500 μg Cl).
d) Special reagents for removal of interference:
    1. Aluminium hydroxide suspension – 125 gm aluminium potassium sulphate of aluminium ammonium sulphate, AIK $(SO_4)_2$. $12H_2O$ or $AlNH_4(SO_4)_2$. $12H_2O$ in 1 liter distilled water. It was warmed to 60^oC and 55 ml concentrated ammonium hydroxide (NH_4OH) are added slowly with stirring. It is allowed to stand for 1 hour and transferred to a large bottle. The precipitate is washed by successive additions, with thorough mixing and decanting with distilled water, until it is freed from chloride. When freshly prepared, the suspension occupies a volume of approximately 11.
    2. Phenolphthalein indicator solution
    3. Sodium hydroxide, NaOH, 1 N.
    4. Sulphuric acid, H_2SO_4, 1 N.
    5. Hydrogen peroxide, H_2O_2, 30%.

Procedure

a) Sample preparation: Suitable portion of the sample is diluted to 100 ml. [If found coloured, 3 ml $Al(OH)_3$ is added to the sample, and the suspension formed is allowed to settle and filtered. If sulphide, sulphite or thisulphate is present, 1 ml H_2O_2 is added and stirred for 1 min.

b) Titration: Samples are directly titrated in the pH range 7 to 10. The pH of sample is adjusted to 7 to 10 with H_2SO_4 or NaOH, if it was not in this range. 1.0 ml $K_2Cr_2O_4$ indicator solution was added and titrated with $AgNO_3$ titrant to a pinkish yellow end point.

$AgNO_3$ titrant is standardized and reagent blank value was obtained by titration. Calculation is outlined below.

Calculation

$$\text{mg } Cl^-/l = \frac{(A - B) \times N \times 35450}{\text{ml sample}}$$

where,

A = ml titration for a sample
B = ml titration for blank, and
N = normality of $AgNO_3$
mg NaCl/l = (mg Cl^-/ L)x 1.65

4.7) **Nitrate**

Nitrates from decomposition of organic residues may reach ground water. Rain may normally be free of nitrate unless there is a high level of oxides of nitrogen in the atmosphere and rainwater dissolves it. The spectrophotometric method for measurement of NO^-_3 is discussed below.

Principle: Measurement of UV absorption at 220 nm enables rapid determination of NO^-_3. The NO^-_3 calibration curve follows beers law up to 11 mg/ml.

Reagent

Nitrate free water
Stock nitrate solution of KNO_3 (100 ppm) standard nitrate solution is diluted (50 ml stock solution made to 500 ml with water (10 ppm).
HCl soln (IN)

Procedure: Prepare NO^-_3 calibration standards in the range of 6-7 ppm by diluting the standard solution. Add 1 ml of HCl solution and mix thoroughly. Read absorbance against distilled water set at zero absorbance at 220 nm.

4.8) **Fluoride**

Normally fluoride is not expected to be present in rain water. Occurrence of F^- however depending on the level of industrialization and contamination has been reported under certain conditions. The method of measurement is discussed below:

Flouride (F^-) SPADNS method (spectrophotometric)

The reaction rate between flouride and ions is influenced greatly by the acidity of the reaction by increasing the proportion of acid in the reagent the reaction can be made practically instantaneous.
Reagent – Standard flouride solution of NaF (1000 ppm)
SPADNS solution dissolved 958 mg SPADNS sodium 2- (parasulfolthenylzao)- 1,8 dehydromy 3, 6 naphthalene disulfonate, the distilled H_2O and dilute to 500 ml

Zirconyl acid reagent dissolve 133 mg zirconyl chloride octohadrate, zrocl in about 25 ml distilled water. Add 350 ml Conc. HCl and dilute to 500 ml with distilled water.

Phenanthroline Method

Principle: Iron is brought into solution, reduced to the ferrous state by boiling with acid and hydroxylamine, and treated with 1,10-phenanthroline at pH 3.2 to 3.3. Three molecules of phenanthroline chelate each atom of ferrous iron to form an orange red complex. The colored solution obeys beer's law; its intensity is independent of pH from 3 to 9. A pH between 2.9 and 3.5 insures rapid color development in the presence of an excess of phemanthroline. Color standards are stable for at least 6 months.

Reagents

a) Hydrochloric acid, HCl, conc, containing less than 0.00005% iron.
b) Hydroxylamine solution: Dissolve 10 g NH_2OH HCl in 100 ml distilled water
c) Ammonium acetate buffer solution: Dissolve 250 g $NH_4C_2H_3O_2$ in 150 ml distilled. Add 700 ml conc (glacial) acetic acid. Because even a good grade of $NH_4C_2H_3O_2$ contains a significant amount of iron, prepare new reference new reference standards with each buffer preparation.
d) Sodium acetate solution: Dissolve 200 g $NaC_2H_3O_2$ $3H_2O$ in 800 ml distilled water.
e) Phenanthroline solution: Dissolve 100 mg 1,10-phenanthroline monohydrate, $C_{12}H_8N_2H_2O$, in 100 ml distilled water by stirring and heating to 80 C. Do not boil. Discard the solution if it darkness. Heating is unnecessary if 2 drops conc HCl are added to the distilled water. (Note: one milliliter of this reagent is sufficient for no more than 100 μg Fe.

Procedure

Preparation of calibration curves for samples analyzed in accordance with is below:
Range 0 to 100 μg Fe/100 ml final solution pipette 2.0, 4.0, 6.0, 8.0 and 10.0 ml standard iron solution [3 gl] into 100 ml volumetric flasks. Add 1.0 ml NH_2OH HCl solution and 1 ml sodium acetate solution to each flask. Dilute each to about 75 ml with distilled water, add 10 ml phenanthroline solution, dilute to volume, mix thoroughly and let stand for 10 min. Measure absorbance in a 5 cm cell at 510 nm against a reference blank prepared by treating distilled water with the specified amounts of all reagents except the standard iron solution.

Acid zirconyl-SPADNS reagent: Mix equal volumes of SPADNS solution and zirconyl-acid reagent.
Reference solution: Add 10 ml SPADNS solution to 100 ml distilled water. Dilute 7 ml conc HCl to 10 ml and add to the diluted SPADNS solution. The resulting solution, used for setting the instrument reference point (zero).

Procedure

Preparation of standard curve: Prepare fluoride standards in the range of 0 to 1.40 mg/l by diluting appropriate quantities of standard fluoride solution to 50 ml with distilled water. Pipet 5.00 ml each of SPADNS solution and zirconyl-acid reagent, or 10.00 ml mixed acid-zirconyl-SPADNS reagent, to each standard and mix well. Set photometer to zero absorbance with the reference solution and obtain absorbance readings of standards immediately. Plot a curve of the fluoride absorbance relationship.

Stannous Chloride Method

Principal: Molybdophosphoric acid is formed and reduced by stannous chloride to intensely colored molybdenum blue.

Reagents

1. Ammonium molybdate reagent I: Dissolve 25 g $(NH_4)_6Mo_7O_{24}$ $4H_2O$ in 175 ml distilled water. Cautiously add 280 ml conc H_2SO_4 to 400 ml distilled water. Cool, add molybdate solution, and dilute to 1l.
2. Stannous chloride reagent I: Dissolve 2.5 g fresh $SnCk_2 2H_2O$ in 100 ml glycerol. Heat in a water bath and stir with a glass rod to hasten dissolution. This reagent is stable and requires neither preservatives nor special storage.
3. Standard phosphate solution

Add with thorough mixing after each addition, 4.0 ml molybdate reagent 1 and 0.5 ml (10 drops) stannous chloride reagent 1. After 10 min, but before 12 min, using the same specific interval for all determinations, measure color photometrically at 690 nm and compare with a calibration curve, using a distilled water blank.

4.9) **Estimation of Metal Ions**

Metal ions could be eluted from the roof material. Generally toxic metal ions are not expected. The estimation of these can be done by titrimetry, atomic absorption spectrophotmetry etc. The methods are available in standard reference books.

Reference

1. Standard Methods for the examination of water and waste water (1995). 16^{th} Edn. APHA, AWWA/WPCF.

B. **TESTING FOR BACTERIAL CONTAMINANTS**

If rain water gets infected by bacteria and conditions are conducive to their growth it will become unpotable. To ensure that the drinking water satisfies public health requirements, it is necessary that samples should be examined at regular intervals for indicator micro-organisms. Pathogenic organisms are difficult to monitor as the methodology involved is time consuming and highly involved.

Any indicator bacteria selected should be closely associated with source of pathogen. It should be able to provide an accurate estimate of the number of pathogens present at the levels which pose a health risk. It should have same survival characteristics of the most resistant pathogen and be measurable by simple methods with accuracy. It should occur in greater numbers than the intestinal pathogens concerned and should be more resistant to disinfectants and natural processes than the pathogens.

The recommended bacterial indicator is coliform group of organisms. Coliform group of organisms are not pathogenic, they are aerobic, facultatively anaerobic, gram negative, non-sporulating, rod shaped bacteria, that ferment lactose with gas formation and aldehyde/acid with in 48 hours at 35^{o}C inculation. Coliform are capable of growth in presence of bile salts or other surface active agents which are cytochrome oxidase negative. Coliform belongs to family Enterbacteriaceae which includes genera *Escherichia, Klebsiella, Salmonella, Shigella, Aerobacter, Scrratia, Citrobacter* and *Proteus*.

Coliform group (total coliform) are almost always present in large number in faeces of man and other warm blooded animals. They can be detected even after dilution. Detection of thermotolerant faecal coliform particularly presence *Escherichia coli* one of the members of the coliform group is considered specific faecal pollution. However unlike many others which are not thermotolerant *E. coli* groups grow well at 44.5^{o}C, i.e. they can ferment lactose and also mannitol at elevated temperatures with the production of acid and gas and form tryptophan. Confirmed *E. coli* indicates positive reaction to methyl red test (undergoing change in colour due to pH change) failure to produce acetyl methyl carbinol (V.P. test) and failure to utilize citrate as sole source of carbon. The four biochemicals tests popularly known as IMViC test give confirmation of E. coli (Table 1).

Table 4.1: Characteristic based on IMViC pattern

Organism	Indole	Methyl Red	Voges Proskauer	Citrate
Escherichia coli	+	+	-	-
Shigella sp	+ or -	+	-	-
Citrobacter freundii	-	+	-	+
Citrobacter diversus	+	+	-	+
Klebsiella,Enterobacter & Aerobacter group	+ or -	-	+	+

The presence of faecal coliform cannot definitely identify human contamination of water systems and this has led to the introduction of supplementary indicator organisms. Another important component of human microflora is faecal streptococci. A group that includes all streptococci found in the intestine of warm blooded animals. The group `D' streptococci known as Enterococci includes *Streptococcus bovis*, Streptococcus equines and *Streptococcus* *fecalis*. Faecal Streptococcus and sulfile reducing *Clostridia* are useful in determining the origin of faecal pollution and also assessing the efficiency of water treatment process.

Ratio of faecal coliforms (FC) to faecal Streptococcus (FS) (FC-FS ratio).

Human population has a much higher ratio of FC-FS usually in excess of 4.0 whereas other warm blooded animals has FC-FS ratio less than 0.7. This kind of ratio can indicate as to whether contaminant in a water sample is of human origin or not.

Sample Collection, Storage and Transport

Samples should not be taken from leaking taps as there is possibility of external contamination. Water should run for 1 minute to ensure stagnant water is flushed from the pipes before sample is collected. It is desirable to flame the mouth of the tap before sampling.

Containers for sample should be clean, sterile, glass or autoclavable plastic bottles containing 0.1 ml of a 1.8% solution of sodium thiosulphate per 100 ml of sample bottle capacity to neutralize residual chlorine if chlorination has been done. Samples should be kept cool at 4-10^{o}C and transported to the laboratory within 6 hours of collection but never >24 hours.

Methodology for Coliform Detection

1) IMViC reaction
2) Multiple tube method or most probable number method (MPN)
3) Membrane filtration (MF) technique
4) Simple field test for detection of faecal pollution in drinking water (H_2S strip test).

1. **IMViC reaction**
1. Indole Test
   a. Reagents:
   1) Medium – Tryptophane broth
   2) Test reagent – Dissolves 5 g para-dimethyl-aminobenzaldehyde in 75 ml isoamyl (or normal amyl) alcohol. ACS grade and add 25 ml conc HCl. The reagent should be yellow. The amyl alcohol solution should have a pH value of less than 6.0.

   b. Procedure : Inoculate 5 ml portions of medium from a pure culture and incubate at 35$\pm$0.5 C for 24$\pm$2 hr. Add 0.2 to 0.3 ml test reagent and shake. Let stand for about 10 min and observe results.
   A dark red color in the amyl alcohol surface layer constitutes a positive indole test, the original color of the reagent, a negative test. An orange color probably indicates the presence of skatole.

2. Methyl Red Test
   a. Reagents:
   1) Medium: buffered glucose broth
   2) Indicator solution: Dissolve 0.1 g methyl red in 300 ml 95% ethyl alcohol and dilute to 500 ml with distilled water.

   b. Procedure: Inoculate 10 ml portions of medium from a pure culture. Incubate at 35^{o}C for 5 days. To 5 ml of the culture add 5 drops methyl red indicator solution.

Record a distinct red color as methyl red positive and a distinct yellow color as methyl red negative. Record a mixed shade as questionable and possibly indicative of incomplete culture purification.

3. Voges-Proskauer Test
   a. Reagents:
   1) Naphthol solution: Dissolve 5 g purified ∝-naphthol (melting point 92.5^oC or higher) in 100 ml absolute ethyl alcohol. When stored at 5 to 10^oC, this solution is stable for 2 week.
   3) Potassium hydroxide, 7N: Dissolve 40 kg KOH in 100 ml distilled water.
   b. Procedure: Separate 5 ml of the culture inoculated for methyl red test after 48 hr or inoculate 5 ml of salt peptone glucose medium from a pure culture and incubate for 48 hr at 35$\pm$0.5^oC. To 5 ml of culture add 3 ml naphthol solution and 1 ml KOH solution, and shake vigorously. Development of a pink to crimson color within 15 min to 1 hr constitutes a positive test.
4. Sodium Citrate Test
   a. Alternate media: Use either Koser's citrate broth or Simmons' citrate agar.
   b. Procedure:
   1) Lightly inoculate liquid medium with a straight needle, never with a pipette. Incubate at 35$\pm$0.5^oC for 72 to 96 hr. Record visible growth as positive, no growth as negative.
   2) Inoculate agar medium with straight needle, using both a stab and a streak. Incubate 48 hr at 35$\pm$0.5^oC. Record growth on the medium with (usually) a blue color as a positive reaction, record absence of growth as negative.

2. Multiple tube method or Most Probable Number Method (MPN) for standard total coliform (Table 4.2)

(1) Measured volumes of the samples of one or more dilutions are inoculated into a series of bottles or tubes containing suitable differential medium containing lactose. The tube should have inverted Durham tubes. After incubation at 35 or 37^oC for 48 hours tubes are examined for acid and gas production. Positive reaction is only presumptive as in addition to coliform other lactose fermenters also grow and produce acid and/or gas.

- This should be followed by confirmatory tests using selective differential media

- MPN method is a statistical estimate of most probable number of coliform in a given volume of sample.
- Sub-cultures made from positive tubes can be used for differentiation of coliforms.

1. Presumptive test
   Use Mac Conkey Broth in the presumptive test
   a. Procedure: Take 10 ml of Mac Conkey broth in fermentation tubes, add inverted durham tubes, plug and autoclave.
   1) Inoculate a series of fermentation tubes ("primary" fermentation tubes) with appropriate graduated quantities (multiples and submultiples of 1 ml) of sample. If 100 ml sample portions are used prewarm bottles at 35^oC. After adding sample mix thoroughly.
   2) Incubate inoculated fermentation tubes at 35$\pm$0.5^oC. After 24$\pm$2 hr shake each tube gently and examine it and if no gas has formed or been trapped in the inverted

vial, re-incubate and reexamine at the end of 48 ± 3 hr. Record presence or absence of gas formation regardless of amount at each examination of the tubes.

b. Interpretation: Formation of gas in any amount in the inner fermentation tubes or vials within 48+3 hr constitutes a positive presumptive test.

2. <u>Confirmed Test</u>

Use brilliant green lactose bile broth fermentation tubes for the confirmed test.

a. Procedure: Submit all primary fermentation tubes showing any amount of gas within 24 hr of incubation to the confirmed test. If active fermentation appears in the primary fermentation tube earlier than 24 hr preferably transfer to the confirmatory medium. If additional primary fermentation tubes show gas production at the end of 48 hr incubation, submit these to the confirmed test.

b. Procedure with brilliant green lactose bile broth: Gently shake or rotate primary fermentation tube showing gas and with a sterile metal loop, transfer one loopful of culture to a fermentation tube containing brilliant green lactose bile broth.

Incubate the inoculated brilliant green lactose bile broth tube for 48 ± 3 hr at 35 ± 0.5^{o}C. Formation of gas in any amount in the inverted vial of the brilliant green lactose bile broth fermentation tube at any time within 48 ± 3 hr constitutes a positive confirmed test.

3. <u>Completed Test</u>

Use the completed test on positive confirmed tubes to establish definitively the presence of coliform bacteria and to provide quality control data. Double confirmation into brilliant green lactose bile broth for total coliforms and EC broth for fecal coliforms may be used. Consider positive EC broth results as a positive completed test response.

a. Procedure:

1) Streak one or more eosin methylene blue plates from each tube of brilliant green lactose bile broth showing gas, as soon as possible after the appearance of gas. Incubate plates (inverted) at 35 ± 0.5^{o}C for 24 ± 2 hr.

2) The colonies developing on eosin methylene blue agar are called typical (nucleated with or without metallic sheen); atypical (opaque, unnucleated, mucoid, pink after 24 hr incubation), or negative (all others). Pick two or more colonies considered most likely to consists of organisms of the coliform group and transfer growth from each isolate to a lauryl tryptose broth fermentation tube and to a nutrient agar slant.

Incubate secondary broth tubes at 35 ± 0.5^{o}C for 24 ± 2 hr, if gas is not produced within 24 ± 2 hr reincubate and examine again at 48 ± 3 hr. Microscopically examine gram-stained preparations from those 24 hr agar slant cultures corresponding to the secondary tubes that show gas.

b. Interpretation: Formation of gas in the secondary tube of lauryl tryptose broth within 48 ± 3 hr and demonstration of gram negative, nonspore-forming, rod-shaped bacteria in the agar culture constitute a satisfactory completed test, demonstrating the presence of a member of the coliform group.

The sequence of tests for coliform for deciding on bacterial contamination in water are summarised in the flow chart.

Detection of coliform in water

Presumptive test

Inoculation of Mac Conkey Broth

Gas produced = Presumptive evidence of Coliform present

No gas = Coliform not present Examination stops here

Confirmed Test

Brilliant green lactose bile broth (BGLB) this medium inhibits the growth of Lactose fermenters other than coliform, thus gas formation in the BGLB medium constitutes a confirmed test, i.e. Coliforms present

or

Eosin-methylene blue agar (EMB) (coli-aerogenes organism produce characteristic colonies; Escherichia, small colonies, dark, almost black centres with green metallic sheen; Enterobacter; large pinkish mucoid colonies, dark centres, rarely show metallic sheen; presence of typical colonies constitutes a confirmed test, i.e., coliforms present)

Completed test

The most typical colonies are selected from EMB plate or BGLB and are inoculated into

Lactose broth (coliforms produce gas)

Agar slant (Gram stain prepared from growth, coliforms are Gram-ve, non sporulating bacilli)

Table 4.2: For most probable number (MPN) index and 95% confidence limits for various combinations of positive results when five tubes are used per dilution (10 ml, 1.0 ml, 0.1 ml)

Combination of Positives	MPN Index/100 ml	95% Confidence Limits	
		Lower	Upper

0-0-0	<0	-	-
0-0-1	2	<0.5	7
0-1-0	2	<0.5	7
0-2-0	4	<0.5	11
1-0-0	2	<0.5	7
1-0-1	4	<0.5	11
1-1-0	4	<0.5	11
1-1-1	6	<0.5	15
1-2-0	6	<0.5	15
2-0-0	5	1	13
2-0-1	7	2	17
2-1-0	7	2	17
2-1-1	9	3	21
2-2-0	9	1	21
2-3-0	12	2	28
3-0-0	8	2	19
3-0-1	11	4	25
3-1-0	11	4	25
3-1-1	14	5	34
3-2-0	14	3	34
3-2-1	17	5	46
4-0-0	13	5	31
4-0-1	17	7	46
4-1-0	17	9	46
4-1-1	21	7	63
4-1-2	26	9	78
4-2-0	22	9	67
4-2-1	26	11	78
4-3-0	27	12	80
4-3-1	33	7	93
4-4-0	34	11	93
5-0-0	23	15	70
5-0-1	31	11	89
5-0-2	43	16	110
5-1-0	33	21	93
5-1-1	46	17	120
5-1-2	63	23	150
5-2-0	49	28	130
4-2-1	70	25	170
5-2-2	94	31	220
5-3-0	79	37	190
5-3-1	110	44	250
5-3-2	140	35	340
5-3-3	180	43	500
5-4-0	130	57	300
5-4-1	170	90	490
5-4-2	220	120	700
5-4-3	280	68	850
5-4-4	350	120	1,000
5-5-0	240	68	750
5-5-1	350	120	1,000
5-5-2	540	180	1,400
5-5-3	920	300	3,200
5-5-4	1,600	640	5,800
5-5-5	≥2,400	-	-

2. **Membrane Filtration Techniques (MF)**
   Coliform organisms in water is determined by filtering known volume of the sample or appropriate dilution of it through porous cellulose acetate membrane, which is then incubated face upwards on suitable selective media in the absorption pads in the petri-dishes at 35-37^oC for 24-48 hours. Colonies that develop can be counted with the help of magnifying lens. Counts are expressed per 100 ml basis. It is usual to use 2 membranes for each sample, in order to incubate at 35-37^oC for total coliform and the other at 44.5^oC for faecal coliform.

Laboratory apparatus

Sample bottles, dilution bottles, pipettes and cylinders, containers for the culture medium, culture dishes, filteration units, filter membrances, absorbent, incubator, microscope

Procedure

Selection of sample is governed by expected bacterial density which in finished water samples will be limited only by the degree of turbidity.

Filteration of sample

Using sterile forceps, place a sterile filter over porous plate of receptacle, grid side up. Place matched funnel unit over receptacle and lock it in place. Filter sample under partial vacuum. With filter still in place, rinse funnel by filtering three 20-30 ml portions of sterile dilution water. Unlock and remove funnel, immediately remove filter with sterile forceps and place it on sterile pad or agar with a rolling motion to avoid entrapment of air. Use sterile filtration units at the beginning of each filtration series as a minimum precaution to avoid accidental contamination. A filtration series is considered to be interrupted when an interval of 30 minutes or longer elapses between sample filtrations. After such interruption, treat any further sample filtration as a new filtration series and sterilize all membrane filter holders in use. Decontaminate the equipment between filtrations by sterilization.

Enrichment technique

Place a sterile absorbent pad in the upper half of a sterile culture dish and pipette enough enrichment medium (1.8 to 2.0 ml lauryl tryplase broth) to saturate pad. Carefully remove any surplus liquid. Aseptically place filter through which the sample has been passed on pad. Incubate filter without inverting dish for 1.5 to 2 hr. at 35 ± 0.5^o C in an atmosphere of at least 90% relative humidity. Remove enrichment culture from incubator, lift filter from enrichment pad and roll out the agar surface. If the liquid medium is used, prepare final culture by removing enrichment culture from incubator & dish halves. Place a fresh sterile pad in bottom half of dish and saturate it with 1.8 to 2.0 ml. of final M-Endo medium. Transfer filter to new pad. Discard used pad with either agar on the liquid medium invert dish and incubate for 20-22 hr at 35 ± 0.5^oC.

Counting

Typtical coliform colony has a pink to dark red colour with a metallic surface sheen. The sheen area may vary in size from a small pinhead to complete coverage of colony. Count the colonies with colony counter

Calculation of coliform density:

$$\text{Total coliform colonies/100 ml} = \frac{\text{Coliform colonies counted x 100}}{\text{ml sample filtered}}$$

3. Simple field test for detection of faecal pollution in drinking water (H_2S strips)

The test is based on detection of hydrogen sulfide producing organisms in samples in water. It has been observed that coliform in drinking water is consistently associated with organisms that produce H_2S (*Salmonella, Proteus, Citrobacter* and same strains of *Klebsiella*)

Methodology: The method uses concentrated medium containing:

i)	Bacto peptone	20 g
ii)	Dipotassium hydrogen phosphate	1.5 gm
iii)	Ferric ammonium citrate	0.75 gm
iv)	Sodium thiosulphate	1.0 gm
v)	Teepol	1.0 ml
vi)	H_2O	50 ml

One ml of concentrated medium were absorbed on to 80 cm^2 folded tissue paper, which was placed in McCartney bottles sterilized and dried at 50^oC. The water samples to be screened for faecal pollution were placed in bottles up to pre-calibrated mark (20 ml) and allowed to stand at ambient temperatures (30-37^oC). Faecal pollution is indicated if the contents of the bottle turned black due to formation of iron sulfide in 12-18 hours.

H_2S producing organisms	Release H_2S + Fe	FeS
Coliforms in H_2O sample	During growth in medium that contain Ferric ammonium citrate	Black colour in the medium

The new test medium has good correlation with MPN test. The method is reliable and simple to perform.

Note: Recently UNICEF held a workshop on the efficacy of H_2S strip test and modifited methods to make it more rapid and accurate.

Tests for the faecal streptococcus group

Multiple tube technique

1. Presumptive test procedure
a. Take a series of test tubes containing 10 ml. Azide dextrose broth. Use 10 ml single strength broth for inocula of 1 ml or less and 10 ml double strength broth for 10 ml inocula
b. Incubate inoculated tubes at 35$\pm$0.5^oC. Examine each tube for turbidity at the end of 24$\pm$2 hrs. If no turbidity is present, reincubate and read against at the end of 48$\pm$3 hrs.

Confirmed test

Subject all azide dextrose broth tubes showing turbidity streak a portion of growth from each positive azide dextrose broth on a petridish containing PSE agar. Incubate the inverted dish at $35 \pm 0.5^{o}C$ for 24 ± 2 hr. Brownish black colonies with brown holes confirms the presence of *Faecal stretococci*.

Computing & recording of MPN as given in table 2.

C. TEST KITS AVAILABILITY AND DETAILS IN INDIA

For rapid testing kits have been developed by a number of Indian institutions and are being marketed. Also other countries have developed kits. Recently the Central Pollution Control Board (CPCB), Delhi and the Sriram Institute for Industrial Research, Delhi have conducted a comparative evaluation of various water testing kit packages available in India. Some manufacturers names and address are listed below . References on major reports concerning WQ monitoring are also listed.

Details of selected Water Quality Kits: Availability in India

1.	Developed by	:	Defence Laboratory, Jodhpur
	Method of Analysis	:	Qualitative, visual colour comparison
	Parameters that can be analysed	:	Fluoride, nitrate, iron, nitrite, residual chlorine, chlorides, total dissolved solids, faecal coliform
	Power Supply	:	220V AC mains or 12 V/DC battery
	Weight	:	10 kg
	Size	:	42 cm x 28 cm x 25 cm
	Components	:	Single unit with built in incubator
2.	Developed by	:	Industrial Toxicology Research Centre, Lucknow
	Method of Analysis	:	Quantitative, colorimetric & titrimetric
	Parameters that can be analysed	:	pH, fluoride, nitrate, iron, nitrate residual chlorine, chlorides, alkalinity, hardness and faecal coliform
	Power supply	:	AC mains, 20/110 volts for 1 KVA generator
	Weight	:	16 kg
	Size	:	50 cm x 35 cm x 28 cm
	Components	:	Incubator and calorimetric unit with suitcase holding chemicals (2 units)
3.	Developed by	:	National Environmental Engineering Research Institute, Nagpur
a)	Name of the kit	:	Rapid Aqua Tester
	Method of analysis	:	Colour comparison
	Parameters that can be analysed	:	pH, fluoride, iron, residual chlorine
	Power supply	:	No power required
	Weight	:	300 g
	Size	:	9.7 cm x 9.7 cm x 3.1 cm
	Components	:	Single unit
b)	Name of the kit	:	Rapid Bacteriological Tester
	Method of analysis	:	Bacteriological
	Parameters that can be analysed	:	Faecal coliform (12 hours)
	Power supply	:	230 volts AC
	Weight	:	10 kg
	Size	:	39 cm x 34 cm x 30 cm
	Components	:	Single unit
c)	Name of the kit	:	Aquazer

	Method of analysis	:	Visual colour comparison, titrimetric tablet count method, bacteriological examination
	Parameters that can be analysed	:	pH, fluoride, iron, residual chlorine, chlorides, hardness, nitrite, nitrate and faecal coliform (12 hours)
	Power supply	:	230 volts AC
	Weight	:	10 kg
	Components	:	Single unit
d)	Name of the kit	:	Titrimetric Water Analyser
	Method of analysis	:	Titrimetric tablet count method
	Parameters that can be analysed	:	Acidity, alkalinity, hardness and chlorides
	Power supply	:	No power required
	Components	:	Single unit
e)	Name of the kit	:	Mini Digital Colorimeter
	Method of analysis	:	Quantitative
	Parameters that can be analysed	:	Fluoride, iron, residual chlorine, nitrate and chromate
	Power supply	:	Single 9 volt battery and 2 1.5 volt cells
	Weight	:	400 g
	Components	:	Single unit
4.	Developed by	:	Elico Water Quality Analyser
	Method of analysis	:	Through probes
	Parameters that can be analysed	:	pH, temperature, dissolved oxygen, conductivity
	Power supply	:	Two 9 volt battery
	Weight	:	6 kg
	Size	:	Medium size VIP suitcase
	Components	:	Single unit
5.	Developed by	:	CSIO, Chandigarh (Central Scientific Instruments Organisation)
	Parameters that can be analysed	:	Turbidity, conductivity, salinity and pH
	Operated by	:	Main supply and battery
6.	Developed by	:	AIIH & PH (Calcutta) All Indian Institute of Hygiene & Public Health
	Parameters that can be analysed	:	Turbidity, pH, hardness, chloride, iron, nitrate, fluoride, residual chlorine and arsenic Bacteriological test kit with incubator is provided

INTERNATIONAL AGENCIES

A) Millipore Kits
   - i) Faecal coliform field kit
   - ii) Portable water analysis kit

B) HACH kit (USA)
   - i) Colorimetric
   - ii) Titrimetric

iii) Acid mini drainage test kit

C) Acid mine drainage test kit

REFERENCES ON WQ-KITS/MONITORING

1. Refresher course on `Water quality monitoring and surveillance' Oct. 9-19, 1995, sponsored by Central Public Health Environment Engineering Organisation, Ministry of Urban Development, Government of India, New Delhi, conducted by `Centre for Environmental Science and Technology, Karnataka.
2. Draft base paper on Water Quality Monitoring and Kits by Industrial Toxicology Research Centre (Council of Scientific and Industrial Research), P.O. 80, M.G. Marg, Lucknow-226001.
3. `Manual for Water Testing Kit' developed by Central Pollution Control Board, Parivesh Bhawan, East Arjun Nagar, Delhi-110032, January, 1997.
4. Booklet, JLN Centre for Science Education, IISc, Bangalore, (Kit, Video, JLNCSE, Bangalore). Ref. NCSTC Communications, Vol. 10, No. 8, Nov., 98 (118th Issue).
5. `Prevention and Control of Fluorosis'. Water Quality and Defluoridation Techniques, Vol. II. Rajiv Gandhi National Drinking Water Mission, Ministry of Rural Development, 9th Floor, B-1 Block, Paryavaran Bhawan, C.G.O. Complex, New Delhi-110003, (1993) (pp. 9-30) (Drinking water – physical & chemical standards p. 59).
6. Studies have been conducted by Central Pollution Control Board (CPCB), India.

SECTION – V

WATER TREATMENT IN DRWH

A Desk Review

In limited cases where water treatment has been reported for `DRWH', boiling, chlorination and use of traditional plants (e.g. *Morenga olifera* seeds, *Semacarnus anacardium* seeds, *Eletaria cardamum* seeds, *Vetiveria zizanoides* etc.) and alum are referred to for treatments of water.It is noted that often only the physical status (color, odour and tests) of water is considered before consuming it, and as long as there is no adverse effect effect, the water is taken to be acceptable.

The available data from different regions of the world are summarised below:

Africa

- In a report[5.1] by UNEP describing treatment of water using traditional methods of treatment of water using indigenous plants and natural products is described. Certain soils, such as the clay known as `clarifying earth' in the Gezira and Northern province in Sudan, produce floc in turbid water and induce sedimentation. A similar flocculation of suspended solids can be achieved by adding certain pounded or crushed plants to water. (Common practice in Sudan and the coastal region in East Africa.
- Bambrah and Haq[5.2] discussed the following physical and chemical methods used for treating stored rain water in Kenya.
  - i) Physical Disinfection
  - ii) Boiling
  - iii) ii)UV irradiation: protects the water against new contamination and can serve as a control and monitoring mechanism. (UV light disinfection is rarely applied in developing countries)
  - iv) Chemical Disinfection: Chemicals used include
  - v) Chlorine and chlorine compounds
  - vi) Iodine doses in suitable form
  - vii) Ozone

viii) Potassium permanganate (oxidant)

ix) Hydrogen peroxide (oxidant)

- Gould[5.3] described chlorination as a suitable treatment for rain water in Botswana and observed that periodic chlorination (2 mg chlorine per litre water requires ½ hour contact to be effective and application approximately every 3 months) will be adequate to combat bacteriological contamination.
- Otieno[5.4] discussed quality issues in rain water in Kenya and recommended that rainwater to be used for drinking may be disinfected by chlorination. For example, household bleach containing the active ingredient sodium hypochlorite may be used, at the rate of 1 ml/25 litres of water. Where resources and technology permit, filters could be used along with ultravoilet (UV) light to disinfect drinking water.

U.S.A.

- Fujioka et al.[5.5] described a solar powered UV-system to disinfected cistern water at (Honolulu/Hawaii). Water in the cistern tanks contain high concentrations of total bacteria as well as faecal indicator bacteria well in excess of drinking water standards. The paper describes a system which is a small, solar powered UV unit which uses gravity flow to process 1.5 litres of water per minute. This unit was shown to be effective in inactivating up to 99.9% of faecal indicator bacteria (faecal coliform, E. coli, Enterococci) and upto 99.999% of total heterotrophic bacteria present in cistern water samples.
- Fujioka et al.[5.6] also recommends a simple to do home owners test for bacteria in cistern water to indicate level of contamination. The hydrogen sulfide (H_2S) test described is a simple, reliable test that home owners can use to analyse cistern water in their own homes without the use of special equipment. This is a semi-quantitative test. The home owners are provided with a 10 ml test container and 100 ml test container to analyse each water sample. The smaller (10 ml) container is filled upto the line indicating 10 ml volume and the larger (100 ml) container to a line indicating 100 ml volume. Both containers have the appropriate dried test paper which includes all the necessary reagents. After filling the two containers with water, containers are recapped and held at ambient temperatures (23-30^{o}C). If blackening of the test paper

is observed after 24-48 hours it is taken as a positive test result for H_2S and hence the bacteria. A cistern sample which is positive in 10 ml and 100 ml sample indicates that the water is heavily contaminated with bacteria and should be disinfected. On the other hand, if the test is negative for H_2S bacteria in the 10 ml sample but positive in the 100 ml sample, a lower level of contamination is indicated.

Europe

- Joklik[5.7] described potabilization of water using ultra violet plant in Austria. A rainwater potabilization plant with a design capacity of 2000 litres of safe potable water/hour is described. The plant uses uv short wave UV-C radiation of 253.7 nm for rainwater disinfection without the use of chemicals. The rainwater is collected from the roof into a rainwater cistern and then directed to a combined filtration unit consisting of a pre-filter, a coarse filter and a fine filter for the removal of mechanical impurities and then passed to an activated carbon filter for the removal of dangerous chemicals and for taste and smell improvement. The filtered and activated carbon treated water is then proceeded to a photo reactor with a coaxilally arranged UV-C sensor and all the process data are recorded on an UV-C data logger. The disinfected water is finally passed onto a hardness increase filter and a micro dosage unit for the addition of essential minerals, vitamins and trace elements to obtain a healthy drinking water of excellent quality. Also the author has tabulated pathogenic micro-organisms in water.

Asia

- Appan[5.8] described utilization of rainfall in airports in Singapore for non-potable uses. The raw water is treated before usage. Rain water from impounding reservoir is pre-chlorinated and then dosed with alum and lime. The coagulated water is then flocculated in an upward flow sedimentation tank and the settled water is filtered in rapid sand filters.
- Appan[5.9] also reported on the water quality control in some southeast Asian countries and has listed some treatment methodologies. In some cases quality control is arbitrary and limited to few cases to diverting of first flushes and rearing of fish

within the container. Boiling, despite its limitations, is the easiest and surest way to achieve disinfection. Alternatively, simple methods of adding any halogen compounds could be practiced. Chlorine in the form of household bleach has been successfully applied to collected roof water, retaining residual levels of 0.2 mg/l.

The cheapest UV system could still be prohibitive in many developing countries. It is also recommended that, as most of the countries in the region have abundant amount of sunlight, research should be undertaken to study the possibility of using solar radiations as T. coli, F. coli & F. steeptococci have effectively been removed when exposed to sunlight in Honolulu and Thailand.

- Bo Ling[5.10], (China) discussed a novel disinfection process, termed KDF-55.

  Water in cistern tanks , contains high concentrations of total bacteria as well as faecal indicator bacteria, well in excess of drinking water standards. One simple to operate reliable and low costs disinfectiong method which can be used in the cistern water without the need for electrical power source and without changing taste of water is a kind of processing medium which is made of a very purified alloy of Cu-Zn which is called as KDF-55.

- Different water treatment and water protection methodologies in Iran are reported by Hussain et al.[5.11], and are summarised below:

  i) Biological Control: A special kind of fish known locally as Loi, Sing or Magur (cat fish) was allowed to grow in the stored water. These species ate the larva of mosquitoes and other insects. On the other hand, the fishes discharged their own excreta into the water which caused deterioration in the quality of drinking water

  ii) Chemical Treatment: Lime or alum is used to purify water. A few households in the area developed a technique to maintain the quality of stored water by treating it with about 25 g/200 litre of burnt snail shell. Some households claimed that water treated with burnt ash of snail improved taste of drinking water.

  iii) Filteration: Several types of filter elements such as a piece of cloth or mosquito netting were used to filter water. In hilly and forest areas a sand filter unit was used to filter rainwater before it was consumed. Pond water was also filtered sometimes to remove suspended particles before drinking. Sedimentation or boiling were also practiced before river or canal water was consumed.

India

- Ramani[5.12] (Madras) described various treatments of stored rainwater as follows:
  - i) A piece of Alum is tied to a rope is swirled through the water for 5-7 minutes. In 2-3 hours the flocculated mass with a mild film formation on the top of the water level along with the other foreign materials is allowed to settle. (if ferric alum is used, the duration can be reduced and better results are obtained).
  - ii) To prevent bacterial growth ,a mild dozing of chlorine powder/liq. Chlorine is given as preservative agent and the storage tank is kept closed (anaerobic). For using residual chlorine 2-3 drops of Ortho-Tolidine is used in a small quantity of water. (if color of H_2O changes to yellow, presence of residual chlorine is indicated. The water sample is then taken to be bacteria free and safe for drinking).
  - iii) $KmnO_4$ may also be used for disinfection.
- In a report[5.13] From RGNDWM, Avinashilingam Institute (Coimbatore) described rain water purification methods/treatment at household level. These include;
  - i) <u>Physical Methods</u>: Filteration through cloth, Boiling and Boiling and filtering
  - ii) <u>Chemical Methods</u>: Chlorination, Alum and Lime treatment.
- In the rainwater harvesting system designed by P.C. Sharma at SERC, Ghaziabad designed a pre filter is inbuilt at the mouth of the storage tank. Water is chlorinated as needed.

A number of compilations are available on water treatment in general . These can also be applied to DRWH. In the Indian context Vasudevan and Saxena[5.14] have listed available technologies for water treatment in the directory on Drinking Water of Rural Technologies, Volume 3 on drinking water, 1989, compiled by the authors. They have described low cost methodologies appropriate for the rural areas where there are financial and infrastructural constraints. It includes domestic filter unit, charcol water filter, filter Aid FA-5 for mobile filtration, slow and filtration plant, water filter candles, package water treatment plant, traditional .water purifying seeds, chlorine tablets for distribution and pot chlorination respectively. and Rural Energy Journal has discussed `Solar water disinfection' as treatment procedure. The details are reproduced in appendix (5A-1 – 5A-10).

References

Africa

5.1 Rain stormwater harvesting in rural areas, The UNEP. Pub: Tycoby International Pub. Ltd. Dublin, 1983, Page: 39.

5.2 Dr. G.K. Bambrah and Ms. S. Haq. Quality issues in rainwater harvesting in Kenya. Proceedings of 8th International Conference on RWCS, April 25-29th, 1997, page 552.

5.3 J.E., Gould. Rainwater catchment possibilities for Botswana, Botswana Technology Centre. April, 84, page 10-12.

5.4 F.O. Otieno, Quality issues in rainwater collection. Raindrop, June, 94.

USA

5.5 Roger Fujioka, Geeta Rijal and Bo Long, 7th International RWCS Conference, A solar powered UV-system to disinfect cistern water (Honolulu/Hawaii). June 21-25, 1995, Beijing, China. Proceedings, Vol. 2, page 48-53.

5.6 Geeta Rijal and Roger Fujioka, A Home owners test for bacteria in Cistern waters. 7th International RWCS Conference, June 21-25, 1995, Beijing, China. Proceedings, Vol. 2, page 58-64. (Honolulu/Hawaii).

Europe

5.7 Otto F. Joklik, Potabilization of Rainwater (Austria). 7th International Rainwater Catchment System Conference, June 21-25, 1995, China, page 33-47.

Asia

5.8 A. Appan, The utilization of rainfall in airports for non-potable uses. Proceedings of the 6th Int. Conference on RWCS. Nairobi, Kenya, 1-6 August, 1993.

5.9 A. Appan, Roof water collection systems in some southeast Asian countries: Status and water quality levels. The Journal Royal Society of Health. October, 97, Vol. 117 No. 5, pages 319-323.

5.10 Bo-Ling, KDF-55 processing medium to disinfect eistern water. International Symposium and 2nd Chinese National Conference on Rainwater Utilization, Sept. 8-12, 1998, page 79-80.

5.11 MD Daulat Hussain, MD. Ahiduzzaman, Thomas Rozario, Proceedings of the 8th International Conference on Rainwater Catchment Systems, April 25-29, 1997, Tehran, Iran, page 648-653.

India

5.12 R. Ramani, March, 99 (1050, 41st Street, T.N.H.B. Colony, Korattur, Chennai-600080), page 15, 17, 22-23.

5.13 Effective use and conservation of water resources by rural households. A micro level action study sponsored by RGNDWM, conducted by Avinashilingam Institute for Home Science and Higher Education for Women Deemed University, Coimbatore (1993) p. 63.

5.14 Directory of Rural Technology, Volume 3, 1989. Drinking Water. Council for Advancement of People’s Action and Rural Technology, New Delhi, page 46-54.

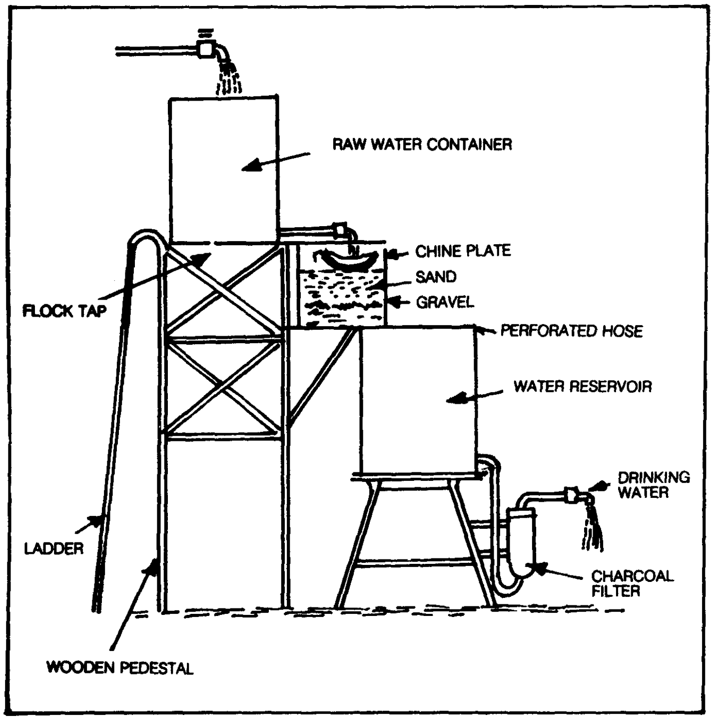
Appendix 5A-1

Drinking Water	Water Purification, Storage and Supply	DOMESTIC FILTER UNIT

Code PS 2001

CLASSIFICATION

Lab. stage	•
Prototype field tested	
Manufactured	



DOMESTIC FILTER UNIT

SCOPE

It is a simple device for purifying and disinfecting raw water for drinking purposes.

SALIENT FEATURES

The equipment provides a simple way of performing disinfection, sedimentation and filtration. It consists of three tanks namely a raw water tank, a filter and a clean water reservoir.

The unit is operated by adding half a teaspoonful of calcium hypochlorite to about 100 l of raw water in the tank, it is stirred gently for a few minutes (five minutes or so), then one spoonful of aluminium sulphate or two spoonfuls of potassium alum is added and stirred for five minutes until sediments start forming. At this point one spoonful of calcium carbonate is added and the water in the tank is stirred and the flow is drained through cock which is at the bottom.

Then the tap is opened and water flows on the swash plate in the second container until the tank is full, the outlet tap allows water to flow through a charcoal filter contained in a cylindrical vessel of about 75 cm length and 25 cm diameter (it is filled with charcoal between two layers of fibrous material such as palm fibre). Filtered water through this is collected in a clean water reservoir.

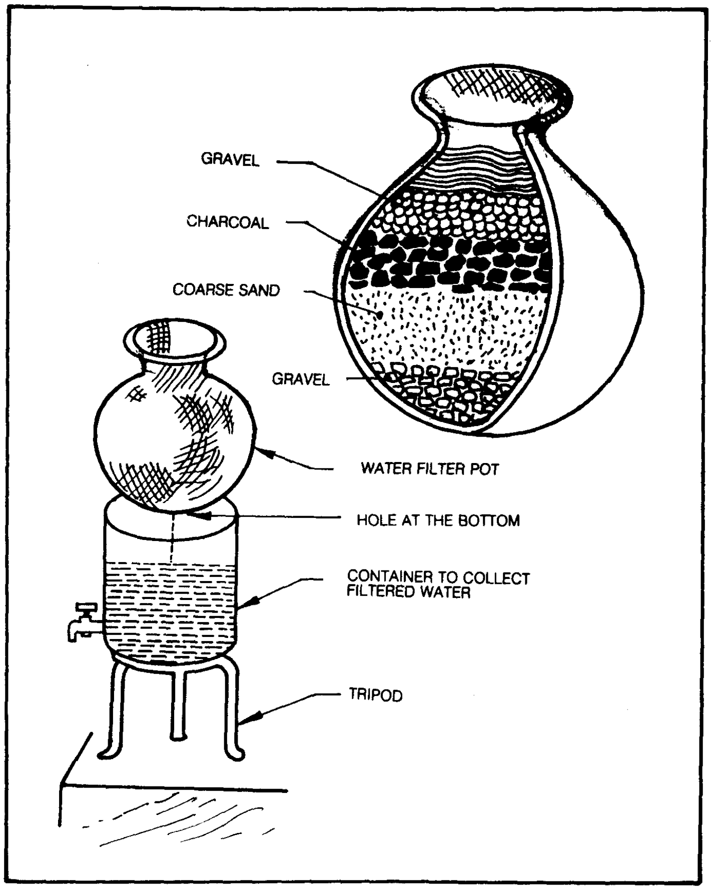
The unit can be easily fabricated in the village and would cost approximately Rs. 250/- only.

CONTACT AGENCY

Director
Appropriate Technology Development Association
Gandhi Bhawan
Lucknow-226 001

Appendix 5A-2

Drinking Water | **Water Purification, Storage and Supply** | **CHARCOAL WATER FILTER**



CHARCOAL WATER FILTER

Code PS 2002

CLASSIFICATION

Lab. stage	
Prototype field tested	
Manufactured/In use	●

SCOPE

It is an effective technique for the removal of solids, suspended materials and other harmful bacteria. Suitable for adoption at the family level.

SALIENT FEATURES

It consists of a clay pot fitted with a tap at the bottom. Layers of washed gravel, coarse sand, charcoal and another thin layer of gravel again are piled respectively upto 2/3rd of pot. Lowermost layer of gravel removes dust and dirt and the uppermost layer of gravel prevents the charcoal pieces from floating and keeps them stationary. It is advisable to add a few drops of chlorine solution (5-25 concentration) to a jug of filtered water as a disinfectant.

SPECIFICATIONS

Diametre of clay pot	42 cms
Height of clay pot	100 cm
Thickness of lowermost gravel layer	25 cm
Thickness of coarse sand layer	25 cm
Thickness of charcoal layer	15 cm
Thickness of uppermost layer	5 to 10 cm

PRECAUTIONS IN USE

The filter requires cleaning or renewal of the beds when these become clogged. To avoid frequent washing larger clay pots (200 l) can be used.

FIELD APPLICATION

It is being used in rural areas of Maharashtra.

CONTACT AGENCY

Director
National Environmental Engineering Research Institute,
Nehru Marg, Nagpur-400 020
Maharashtra.

Appendix 5A-3

Drinking Water | **Water Purification, Storage and Supply** | **FILTER AID FA-5 FOR MOBILE FILTRATION**

Code PS 2003

CLASSIFICATION

Lab. stage	
Prototype field tested	●
Manufactured	

SCOPE

Pressure filtration through precoat filters is very useful for water filtration on a small scale.

SALIENT FEATURES

The filter aid FA-5 is made from wood charcoal powder of very fine grit (50-100 μm). Raw water having turbidity upto 125 formazin units can be filtered through it without any chemical treatment. With a raw water turbidity of 20-40 FU, upto two cubic meter of filtered water can be obtained from 0.46 m^2 septum area while using 1075 $\pm$ 10 g of FA-5 per square meter septum area. The filter aid could be reused particularly at turbidity levels of less than 60 FU, 3-4 times.

The estimated cost of FA-5 is about Rs. 5/- per kg. and cost of filtration is about 50 paise per cubic meter.

CONTACT AGENCY

Director
National Environmental Engineering Research Institute
Nehru Marg, Nagpur-440 020
Maharashtra

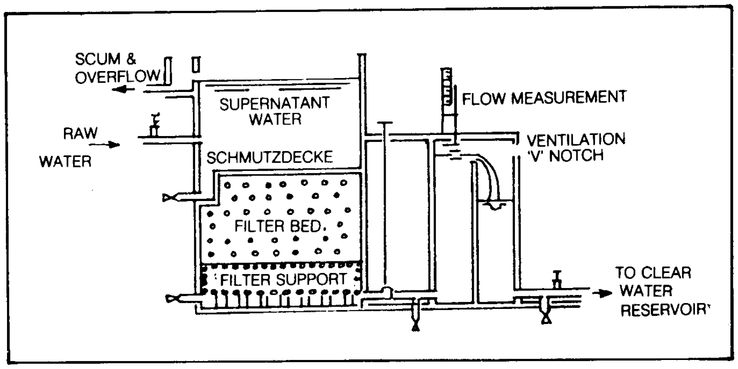
Appendix 5A-4

Drinking Water

Water Purification, Storage and Supply

SLOW SAND FILTRATION PLANT

Code PS 2004



SLOW SAND FILTRATION PLANT

CLASSIFICATION

Lab. stage	
Prototype field tested	
Manufactured/In use	●

SCOPE

It is a simple and economical treatment plant, which does not require any chemicals. It has an edge over rapid gravity and high rate type filters.

SALIENT FEATURES

Slow sand filtration is a simple method for purifying polluted surface waters, especially for rural and small community water supplies. It provides a single step treatment for surface waters of low turbidity (20 NTU). It also improves simultaneously the physical, chemical and bacteriological quality of raw water. For surface waters of high turbidity suitable pretreatment steps such as an infiltration system, sedimentation tanks or lagoons, horizontal flow roughing filtration etc. are required.

The filter unit consists of large size open watertight tanks 2.5–3 m deep (built in masonary/RCC) and filled with fine grade sand (0.25–0.30 mm E.S.) overlaying layers of gravel. Raw water is distributed gently over the top of the filter bed without addition of any chemical, and clean filtered water is collected by a system of under drains constructed at the bottom of the filter.

The process of purification of water by slow and filtration is as follows:

Water passes slowly through the filter bed at a rate of 0.1–0.2 m/hour. Its quality gets improved considerably during this passage due to removal of impurities and reduction in the number of micro-organisms (bacteria, viruses, cysts etc.) Soon after the start of the process a biological film or filter skin forms on the surface of the sandbed, which contains a wide variety of biologically active micro-organisms. These break down organic matter, including bacteria and viruses in water and convert it into water, carbon di-oxide and harmless inorganic salts. At the same time, a great deal of suspended inorganic matter is retained by straining. The continuous straining process gradually increase the resistances in the filter skin.

SPECIFICATIONS

Initial depth of filter bed	0.8–1.21m
Number of filter beds	minimum two
Minimum depth of sand before resanding	0.5 m
Depth of underdrains	0.2-0.4 m
Sepcification of filter	effective diameter 0.15–0.35 mm, uniformity coefficient 5
Specification for sand support Coarse sand gravel	 1-1.4-100 mm deep 4-5.6–100 mm deep 16-23–100 mm deep
Rate of filtration	0.1–0.3 m/hour
Period of operation	24 hours capacity 8000 m^3/day

PRECAUTIONS IN USE

After about 1–3 months the resistance in the filter skin becomes high enough to decrease the capacity of the plant for filtration. Hence cleaning of the filterbed by scraping off the top 2 cm of sand including filter skin, must be taken up periodically.

FIELD APPLICATION

These plants have been constructed in villages of Punjab, Haryana, Uttar Pradesh, Andhra Pradesh and Tamil Nadu. Demonstration is available at National Environmental Engineering Research Institute, Nagpur.

CONTACT AGENCY

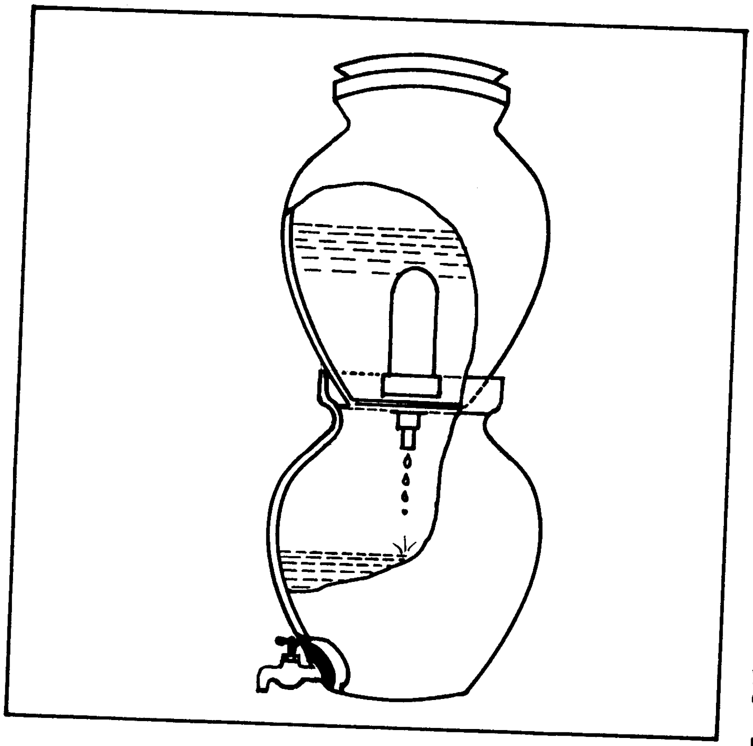
Director,
National Environmental Engineering Research Institute,
Nehru Marg, Nagpur-440 020.

Appendix 5A-5

Drinking Water

Water Purification, Storage and Supply

WATER FILTER CANDLES



Code PS 2005

CLASSIFICATION

Lab. stage

Prototype field tested

Manufactured •

SCOPE

This is an inexpensive way of getting potable water in the house. The filter candle can be fitted in any type of container and can remove all suspended impurities including harmful bacteria.

SALIENT FEATURES

The water filter candles are manufactured from non plastic materials like quartz and felspar, which are ground to the required fineness in a ball mill and mixed with china clay, plastic clay and some organic combustible materials in required proportions. A casting slop is prepared by adding the required quantity of electrolytes. Candles are made by the usual casting process with plaster of paris moulds. These are finished and baked at a suitable temperature. Candles are then given a special chemical treatment so that they could give bacteria free water. The life of a candle is generally 2–3 years and can be installed in any domestic container.

PRECAUTIONS IN USE

The candle should be handled carefully as it is easily breakable. After long use the surface should be cleared off the mud coating, with the help of a knife or blade periodically.

CONTACT AGENCY

(i) Director
Regional Research Laboratory
Council of Scientific & Industrial Research
Jorhat-785 006.

(ii) Director
Central Glass and Ceramic Research Institute
Jadhavpur University
Calcutta-700 032.

Appendix 5A-6

Drinking Water

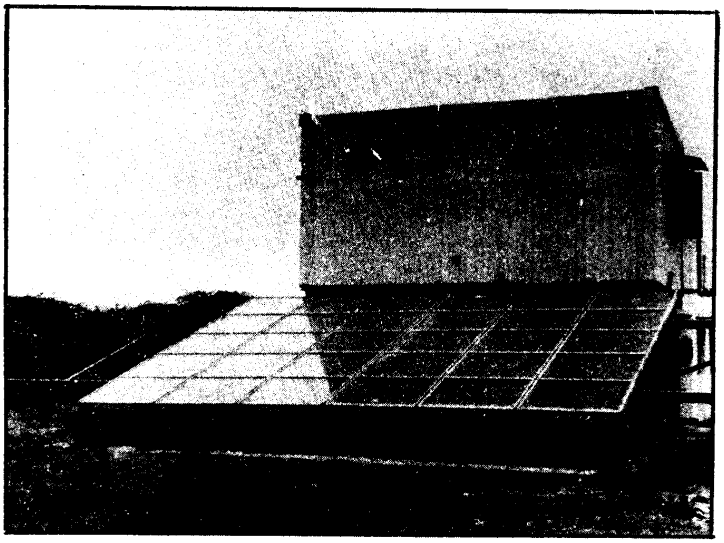
Water Purification, Storage and Supply

PACKAGE WATER TREATMENT PLANT

Code PS 2006

CLASSIFICATION

Lab. stage	
Prototype field tested	●
Manufactured	



SCOPE

This is a compact plant for providing safe drinking water in rural areas.

SALIENT FEATURES

The package plant consists of units for chemical coagulation, sedimentation and filtration. There are three concentric cylindrical compartments, the inner most is the flocculator and central compartment is a hopper bottom sedimentation basin. The flocculator is equipped with paddles. The alum solution is fed by a constant head gravity device, the flocculated and settled water is filtered. A bleaching powder solution is added to the flocculated water before it flows into the sedimentation basin.

The plant produces clean water with less than 2 NTU turbidity from raw water having turbidity upto 400 NTU. The plant can supply safe drinking water to small communities. The alum required is about 7–20 mg/l.

The advantages of the system are that it is easy to operate, maintain and does not need sophisticated mechanical equipment. The units can be pre-fabricated and assembled in the villages.

PRE-REQUISITE FOR ADOPTION

A short training in maintenance and operation is required.

PRECAUTIONS IN USE

Periodic maintenance is required.

FIELD APPLICATION

Evaluation studies on the equipment are carried out in villages of Nagpur district.

CONTACT AGENCY

Director
National Environmental
Engineering Research Institute
Nehru Marg, Nagpur, 440 020

Appendix 5A-7

Drinking Water	Water Purification, Storage and Supply	TRADITIONAL WATER PURIFYING SEEDS

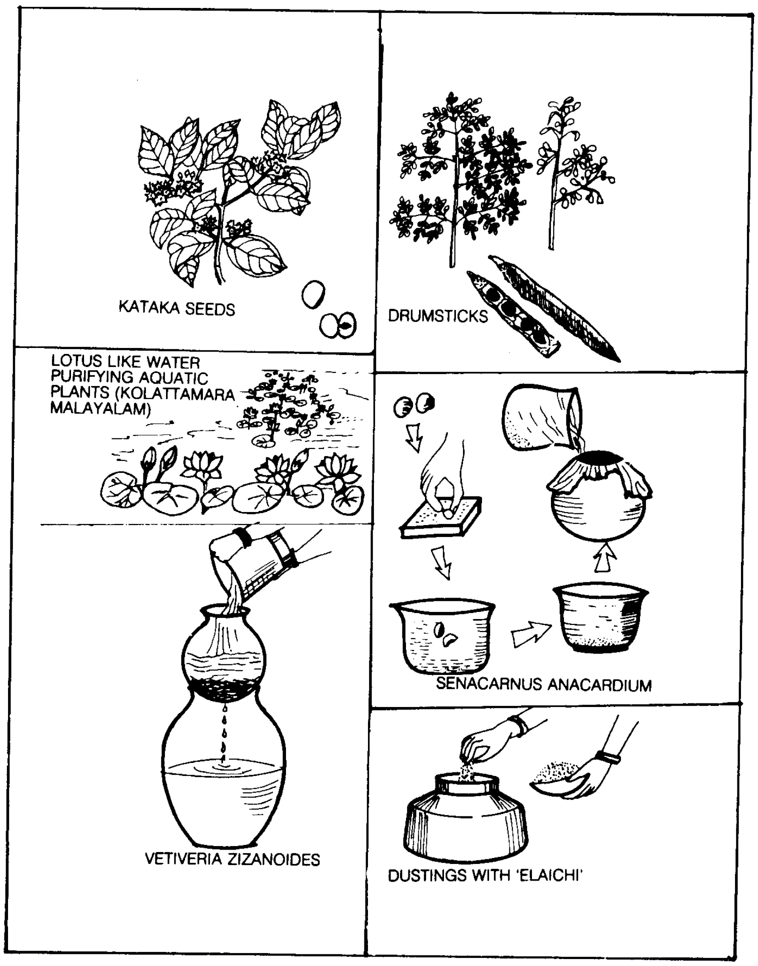
Code PS 2007

CLASSIFICATION

Lab. stage

Prototype field tested •

Manufactured



SCOPE

Good potable water can be had by using some purifying seeds.

SALIENT FEATURES

Many local seeds are used as natural coagulants for treating muddy water. Eg. Kataka Seeds (Strychanos potatorum). A dose of 1.5 mg of seed extract per litre of water is used (seed extract is prepared from a thick paste of crushed seeds with clean water). The water is stirred for 3–5 minutes after putting the seed extract, and it is then treated with 10–15 mg/l of alum. This treatment reduces the turbidity to safe limits.

Similarly, drumsticks (Morenga olifera) seeds are crushed and the powder is mixed with a small quantity of purified water in a glass and stirred vigorously for 5 minutes. The suspension is used for treating turbid water. Generally 30 seeds are required for treating 40 l of raw water.

Seeds of Bhela (Semacarnus anacardium) are rubbed on stone and made into a thick paste, this can be added to turbid water for treatment.

Sometimes coagulants like plant ashes, earth from termite hills, paddy husk or crushed seed of Elaichi (Eletaria cardamum) is dusted on water surface to treat turbidity.

Wiry roots of the rhizome from Ramaccham (Vetiveria Zizanoides) are laid in a pitcher with small holes in the bottom and the pitcher is used as a filter.

Some aquatic plants like Kolattamara may be introduced in the ponds or wells. These plants check the pollution created by animal wastes.

CONTACT AGENCY

Director
Appropriate Technology Development Association
Gandhi Bhawan
Lucknow-226 001

Appendix 5A-8

Drinking Water

Water Purification, Storage and Supply

CHLORINE TABLETS FOR DISTRIBUTION

Code PS 2008

CLASSIFICATION

Lab. stage	
Prototype field tested	
Manufactured	●

CHLORINE TABLETS

SCOPE

Water in rural areas can be made safe for drinking by addition of chlorine tablets.

SALIENT FEATURES

The chlorine tablets are compounded with normal harmless salts and chemicals. For quick dissipation, powdering of the tablets before addition to the water is recommended. For uniform distribution water, slowly stirred about 30 minutes are allowed after administrating the tablet, so that complete disinfection takes place. The shelf life of tablet, stored in a dark place is about two years.

SPECIFICATIONS

Chlorine tablets are available in different sizes and conform to IS: 9825–1981

Weight of a tablet (in gm)	Strength of chlorine (per tablet)(in mg)	Capacity to treat water (in litre)	Packing (Tablets)
5.0	600	550	500
2.5	300	300	1000
0.5	30	25	1000
0.25	5	5	1000

Note: A packet of 1000 tablets of 0.25 gm costs approx. Rs. 80/-

PRECAUTIONS IN USE

Water should be filtered or cleaned and should not have much suspended solids and turbidity.

FIELD APPLICATION

Widely used in villages in India.

CONTACT AGENCY

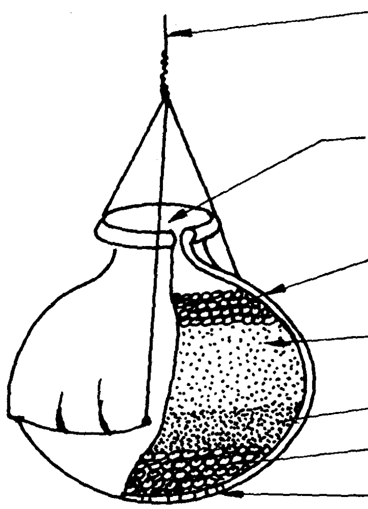
Director
National Environmental Engineering Research Institute
Nehru Marg
Nagpur-440 020

Appendix 5A-9

Code PS 2009

CLASSIFICATION

Lab. stage
Prototype field tested
Manufactured •



SCOPE

It is an inexpensive technique for continuous disinfection of water in open wells.

SALIENT FEATURES

It consists of a plastic pot of 5 l capacity, filled with 2.0–2.5 cm size gravel to a height of 5 cm from bottom. A mixture of bleaching powder and coarse sand (1:4 by weight) is placed on top of the gravel to about an equal height. Again an equal layer of gravel is put and the pot is covered. Two half centimetre holes are made in the cover, and is put in a stout wire cage and is lowered in the centre of the well which is to be disinfected. The length of the rope is so adjusted that the pot is kept submerged in water to a depth of about 1 m. The chlorine from the bleaching power oozes out slowly maintaining a residual concentration of about 0.2–0.5 mg/l for a period of about a week (when the draw off rate of water from well is about 1200 l/day. The process is simple and can be operated by villagers themselves.

PRECAUTIONS IN USE

1 Care should be taken to keep the surroundings of the well to be clean, disinfected and avoid contamination of water through spillage or seepage of used water.

2 Repeat fillings of the pot are required periodically, generally when the residual chlorine is about 0.005–0.002 g/l (it can be estimated by a chloroscope). Otherwise pot may be recharged at intervals of 7–10 days.

FIELD APPLICATION

Being used in a number of villages in Maharashtra, Uttar Pradesh, Madhya Pradesh, Andhra Pradesh, Tamil Nadu etc.

CONTACT AGENCY

Director
National Environmental Engineering Research Institute
Nehru Marg
Nagpur-440 020

Appendix 5A-10

SOLAR WATER DISINFECTION

Putting solar energy to its simplest of uses, for water disinfection, has caught on in several countries. In fact, it is one technique that must score over the more costly ones as potable water in most developing countries need treatment before it is consumed. Interestingly, nature has bestowed such countries with adequate sunshine during most part of the year.

Solar Water Disinfection (SODIS) technique has been perfected. Dissemination of SODIS is not a technological problem, but rather a question of marketing and dissemination of information. Adequate information and controlled demonstration are necessary for large-scale SODIS use. It is the volume of water versus the exposure to solar radiation that needs to be conveyed to the masses.

Yayasan Dian Desa, an NGO in Indonesia, introduced SODIS in the initial phase to 29 families only. However, at the end of the two-year official project phase in 1998, this number increased tenfold to 330 participating households. But wherever government institutions were involved in SODIS dissemination, the impact was far from desirable.

The objective of the SODIS demonstration projects, conducted by local institutions in seven different countries, was to study the socio-cultural acceptance and affordability of this treatment option. The recently carried out survey in Colombia, Bolivia, Burkina Faso, Togo, Indonesia, Thailand and China revealed that an average of 84 per cent of the users will certainly continue to use SODIS after conclusion of the project, and about 13 per cent consider using it is in future. Only 3 per cent thought otherwise because they were presumably not affected by the present water quality.

The most interesting part of the project was the preference of local people for reusing plastic bottles for this purpose – a waste product of the soft drinks industry. An average of 66 per cent of users favour SODIS plastic bottles as these are easy to handle, sturdy and durable. With bottles being available in plenty, replacement of broken bottles is not a problem. The project, which was implemented by the Swiss Federal Institute for Environmental Science and Technology, is now negotiating with the local plastic bottle industry for a joint venture with the SODIS project.

Reference:Rural Energy Journal. `Solar Water Disinfection'. Volume 9 (2) 1999. AFPRO partners

SECTION VI

SCOPE OF FURTHER WORK

Based on the extensive literature review and survey data reported herein, the following work elements have been identified for further action:

1. A number of test kits are available in India and abroad for rapid testing of water samples. There have been modifications and improvements on bacteriological and other tests for water contaminants. In India some institutes/organizations like UNICEF, RGNDWM, CPCB, Sriram Institute for Industrial Research and CAPART have made comparative evaluation of water testing kits. Information from all these agencies will be pooled. The Indian partners will also buy some of these kits and test them. (Already some work has been initiated in this direction). Based on this, recommendations will be made on the choice of kits for rapid testing and most suitable methods for laboratory testing of water samples, specifically in context of DRWH.

2. A questionnaire for collecting field level data on WQM has already been sent to all partners. Each partner is requested to collect available data and also gets sample survey done with regard to WQM in their respective areas of work.

   The Indian partners have already compiled available WQM data based on literature survey and personal interaction and have shown that there is paucity of authentic and accurate information in this regard. They have collected names and addresses of institutes/organisations in India who have put up DRWH in different parts of the country. DRWH systems will be tested with the help of water form some of these institutions. Special attention will be paid to existing units in Kerala and nearby states. A thorough study will be made in these places in collaboration with Sh. P.K. Sivanandan and Mrs. V.K. Sulochana.

3. The Indian partners are to collaborate with Warwick University on storage structures. It has been noted that extensive work has been done by Dr. P.C. Sharma (SERC, Ghaziabad) in this regard. They have essentially used ferro-cement storage tanks of capacity from 600-25000 litre. The shape is cylindrical and the DRWH has inbuilt first flush rejection and filtration facility. To gain from their experience, a training programme is being organised on ferro-cement technology for building some model structures at I.I.T., Delhi. The Indian partners are also working on computer modelling of storage systems.

4. Various parameters contributing to deterioration of water quality have already been identified. WQM has to be related to different roofing materials. (Research has been initiated by consctucting small sheds with different roofing material. New DRWH designs suggested by University of Warwick will be field tested). By inoculation of E. coli and other bacterial strains in water samples and observing their shelf life in aerobic and anaerobic conditions, data will be generated for DRWH designs.

5. Water treatments by sunlight (UV), chlorination and slow sand filter will be studied further.

6. Studies on mosquito breeding will be initiated. To begin with a review will be made and will be submitted under the **Milestone 2: Report C2.**

Milestone 2: Report C2

"DRWH AND INSECT VECTORS: A LITERATURE REVIEW"

Prepared by

PROF. PADMA VASUDEVAN – Principal Investigator

DR. NAMRATA PATHAK - Project Scientist

and

DR. P.K. MITTAL

Centre for Rural Development & Technology

Indian Institute of Technology, Delhi, India

and

Malaria Research Centre,New Delhi

Sub programme C:Health Implications

Project sponsored by

EUROPEAN COMMISSION

"DRWH AND INSECT VECTORS: A LITERATURE REVIEW"

Milestone 2: Report C2 of the project

Section I: Mosquito - The Major Disease Vector

1.1 Introduction

The health implications of widespread use of DRWH are divided into two aspects , namely;

a) Concerns regarding water quality (WQ) and possible direct health implications due to contaminants.

b) Insect vector breeding related to water storage and health implications arising out of it.

A literature review on DRWH- WQ has been presented in milestone report C1. The current report C2 is on the second aspect.

A review of the literature indicates that mosquito is the major insect vector which needs to be considered in the context of DRWH especially in humid tropics. The current review hence deals essentially with mosquito breeding and related diseases. Since designing for mosquito control requires an understanding of breeding habits, life cycle and behavioral patterns of mosquitoes as well as modes of disease transmission, these are discussed below.

1.2 Mosquito borne Diseases

Mosquitoes belong to the Class - Insecta, Order -Diptera and Family - Culicidae. They are thin brown, sized 6.4-12.7 mm, long legged winged insects; adults have three pairs of long, slender legs, and elongate 'beak' or piercing proboscis. Most mosquitoes remain close to the lake, pond or clogged gutter. Rainy season provides plenty of breeding places for them. Breeding of mosquito depends on various factors which include temperature, relative humidity and rainfall pattern. While certain characteristics are common to all mosquitoes, there are differences among different genus and species. Habitat and climate conditions determine the dominance of a particular species in any given area. Larval requirements can also be quite specific to the species. The number of mosquito species under 4 major genus are listed in table 1 along with the number of species which are major disease vectors.

Table 1: Distribution of Mosquito species

Genus	World		India	
	No. of Species	**Major Vector**	**No. of Species**	**Major Vector**
Anopheles	422	60	58	6
Aedes	888	25	111	1
Culex	715	12	57	3
Mansonia	23	7	4	1

Anopheles, *Aedes* and *Culex* are the most common genus and are known to cause various diseases. The major diseases caused by these various species belonging to the sub-family: Anophelinae (Anopheles) and Culicine (Culex, Aedes) are shown in fig. below, and their mode of transmission is discussed briefly.

Diseases caused by mosquitoes

Protozoan disease → Malaria

Filarial disease → Heart worm

Viral disease →
- Yellow fever
- Dengue fever
- Encephalitis

(a) Malaria: caused by protozoan parasites of the genus Plasmodium

- *P. falciparum* - cause most wide spread and dangerous form of malaria
- *P. vivax*
- *P. ovale*
- *P. malariae*

Parasites are transmitted from one person to another person by female *Anopheles* mosquito (males do not transmit the disease as they feed only on plant juices). Parasites develop in the gut of the mosquito and are passed on through the saliva of an infective mosquito, each time it takes a new blood meal. The parasites are carried by the blood stream to the victim's liver, where they invade the cells and multiply. After 9-16 days they return to blood and penetrate the RBC where they multiply again, progressively breaking down RBC. The infected individual is affected by bouts of fever and anaemia. Anopheline species in selected areas of India are listed below.

Major Vector	**Zone of Influence**
An. culicifacies (rural vector)	Vector of rural malaria in the north, south & central India. All over India except north-east
An. stephensi (urban vector)	All towns except north-east, rural areas of arid/semi-arid zone except in the north
An. minimus (foot hill vector)	North-east states, north West Bengal
An. fluviatilis (foot hill vector)	Foot hills all along the Himalayan range, seepages in irrigation channel
An. dirus (forested areas)	Deep forests in north-east region
An. sundaicus (coastal areas)	Andaman & Nicobar Islands

Anopheline vectors related to various countries in the world are available in the book entitled "The anophelines of India", by T. Ramachandra Rao, ICMR, New Delhi (1981).

b) Lymphatic filariasis

Infection of the human lymphatic system by filarial nematodes (round worms) is vectored by mosquitoes. Humans serve as the reservoir of this disease. Mosquitoes taking a blood meal ingest micro-filaria (larval stage) which are present in the blood. There are many mosquitoes species which act as vectors for the different species of filarial nematodes e.g.

- *Wuchereria bancrofti*: vectors are - *Culex quinquefasciatus*, *Aedes gambiae*, *Aedes polynesiensis* etc.
- *Brugia malayi*: is vectored by *Anophelus barbirostris*, several species of *Aedes* & *Monsonia*.

Heart worm disease: found in dogs and related canines is caused by a filarial nematode (a large thread like round worm), *Dirofilaria immitis*. The adult worms live in the right side of the heart (right ventricle) and adjacent blood vessels (pulmonary arteries) and because of their location are commonly "dog heart worms".

c) Yellow fever

This viral disease is transmitted to man by a specific mosquito, *Aedes aegypti*. One gets yellow fever on being bitten by infected *Aedes* mosquito which injects yellow fever virus through the bite.

d) Dengue

Dengue is caused by an RNA flavivirus exhibiting many stereotypes. *Aedes aegypti* is the main vector of dengue. It is a container breeder adapted to urban life. As people move away from the proximity of natural water and start to sequester water in containers, this mosquito finds a good breeding ground. *A. aegypti* transmits dengue via bite only. A mosquito feeding on a person, who is in the 1st-5th day of the disease symptoms, can vector the disease to another person. The dengue virus does not affect the mosquito in any way.

(e) Encephalitis

i) Eastern equine encephalitis (EEE): The EEE virus is maintained in enzootic bird- mosquito cycles centered around hardwood. Passerine birds in and around these swamps serve as reservoirs for the virus. Mosquitoes, especially *Culiseta melanura* feed on these infected birds thereby amplifying the amount of virus. (*Aedes sollicitans* and *Coquillitidia perturbans* - mosquitoes are referred to as bridge vectors due to their ability to bridge the gap between birds and mammals and carry the virus out of bird mosquito cycle and introduce it to mammals.

ii) St. louis encephalitis (SLE): is a mosquito borne viral infection. The virus is known to occur in natural enzootic bird mosquito cycles. The mosquito species responsible for transmitting the infection is the common house mosquito *Culex pipiens*. Occassionally, these mosquitoes will feed on a variety of mammals, including humans, thereby, transmission of the virus can occur through infected mosquitoes that which had previously fed on infected birds. *Culex pipiens* breeds in stagnant water that collects in a variety of places, including catch basins, dis-repaired pools, buckets and other containers, and road side ditches. The virus is thought to enter new areas through migrating birds, because these mosquitoes do not normally fly very far from their breeding sources.

iii) West Nile encephalitis (WNF): is transmitted through a mosquito borne flavi virus. It is found in Africa, Europe and Asia. Typically, this virus causes clinical disease in only a small percent of humans infected. A number of mosquito species have been shown to transmit WNF, the most notable belonging to the genus *Culex* and *Aedes*.

1.3 Characteristics of Mosquito species

For prevention and control of mosquitoes, it is important to know their general morphology/ physiology and other general features and variations among different species.

Life cycle

Adult female (only) mosquitoes seek a blood meal and produce a batch of eggs/individual eggs. Some mosquitoes lay eggs on the sides of tree holes or discarded containers, or in depressions in the ground that can hold water. The eggs can lay dormant for several years and hatch when they are flooded by rainfall. Several flooding and drying cycles are usually required to hatch all the eggs laid by a particular mosquito. Other mosquitoes lay eggs directly on the surface of water.

The life cycle from the stages of eggs to larvae to adults is depicted in Fig. 1. The eggs are often attached to one another to form a raft. The individual eggs or rafts float on water. They hatch in 24-28 hours releasing larvae that are commonly called 'wrigglers'. Larvae can be often seen wriggling up and down from the surface of water.The larval stage is always aquatic and they shuttle from the sub-surface to obtain oxygen through a snorkel-like breathing apparatus. Generally, the larvae feed on micro-organisms and organic material in water. In about 7-10 days after the eggs hatch, larvae (which go through 4 instars or developmental stages) change to the pupal or 'tumbler' stage in preparation for adult life. The pupal stage does not feed but unlike most insect pupae it is extremely active. The adult emerges from the pupal case using air pressure and assumes a terrestrial existence. Male mosquitoes mate with females one to two days after they emerge. A few days after emerging from water, female mosquitoes begin to seek an animal to feed on. Males do not bite, but feed on plant juices. Mosquitoes can travel a mile or more from their breeding spot to find a meal.

The differences in the different stages of life cycle between various species of mosquitoes is important for DRWH design. All mosquitoes go through the stages of egg-larvae-pupae-adult. The egg and pupae are in water, but once the head develops, the adult goes in the air. However notable differences are seen among various genus. These details and differences are summarised in table 2 and further schematically presented in figures 2 and 3.

Conclusions

From the above review it is clearly seen that all mosquitoes have one key requisite; they need water to complete their life cycle. Since DRWH involves water collection and storage, there is a potential for mosquito breeding. All the available literature pertaining to the aspects of mosquito breeding and control with reference to DRWH are compiled in the subsequent section-II: 'DRWH design and insect breeding'.

Under the project, steps have been taken to examine closely the various measures which are to be taken for the prevention of mosquito breeding and control in DRWH system. These are presented in section-III.

Table 2: Characteristics of major mosquito species

PARAMETERS	***ANOPHELES***	***AEDES***	***CULEX***
Sub-family:	Anophelinae	Culicine	Culicine
Life span	7-10 days (male) 4-5 weeks (female)	Average of 20-30 days Females deposit ~ 4 batches of eggs	-
Breeding site	River margins, river bed pools, canal, seepage from water from canal/dam, rainwater (burrow/pits) Low lying grounds, hoof marks & wheel ruts, rice fields, wells, ponds, brackish water pools	Containers, discarded tins, empty pots, broken bottles, coconut shells, artificial collection (of waters) e.g. coolers, tyres (discarded)	Domestic & peridomestic sources such as cesspools, open ditches, sewage and waste water
EGGS	Laid singly (has lateral floats)	Laid singly, no floats, breed in clean/unpolluted water (man made/artificial containers)	Clusters/rafts of 100-250 eggs, no floats
Viability	1-2 days at normal temperature. May be extended upto 1 week in winters	If dried under natural conditions, are viable for 6 months or longer	~ 2-3 days
LARVAE	(yellowish, green/ grey) remain parallel to surface of water, no siphon (breathing tube)	Remain vertically hanging to water surface, has a siphon (breathing tube)	Remain vertically hanging to water surface, has a siphon
No. of Instar & size			
I Instar	< 1 mm	-	-
II Instar	1-2 mm	-	-
III Instar	4-5 mm	5 - 10 mm	-
IV Instar	5-7.5 mm	~ 10 mm	-
To what water depths it can reach	~ upto 1 m	Few meters	
Emergence	7-10 days normally	7-10 days	-

PARAMETERS	***ANOPHELES***	***AEDES***	***CULEX***
Sub-family:	Anophelinae	Culicine	Culicine
ADULT Size	~ 0.5 cm	0.4-0.5 cm	~ 0.5-0.9 cm
Color	Grey, rest at 45o angle to surface	Black with white patches on the legs, rests parallel to surface	Dark grey, rests parallel to surface
Sitting posture Resting habits	• Body parallel to resting surface. • In cattle sheds, under bushes & in tree holes, an indicator of outdoor resting	-	-
Eating habits i) (male) ii) (female)	 Nector/fruit juices Haematophagus	 Nector/plant juices Haematophagus	 Nector/plant juices Haematophagus
Biting habits/time	Both indoor/outdoor Through out night but peak biting occurs between 19 and 4 hrs. Predominantly zoophilic	Mostly indoor than out doors Diurnal, day biters with 2 peaks of biting (1st at the dawn after the sunrise and 2nd at dusk before sun set)	Both indoor/outdoor From dawn to dusk
Flight range	1-3 km	50 -200 m	1-2 km
ADULT DENSITY (needed to maintain transmission)	1 infective bite	No estimate of adult density is known for transmission possibly one infective bite	Usually very high density is required Needs several infective bites for transmission
Diseases caused	Malaria Filaria	• Yellow fever • Dengue • DHF • Filaria • Chikungunya fever	• Ban croftian filarias • Japanese encephalitis • West nile fever • Viral arthrites • Epidemic/polyarthritis

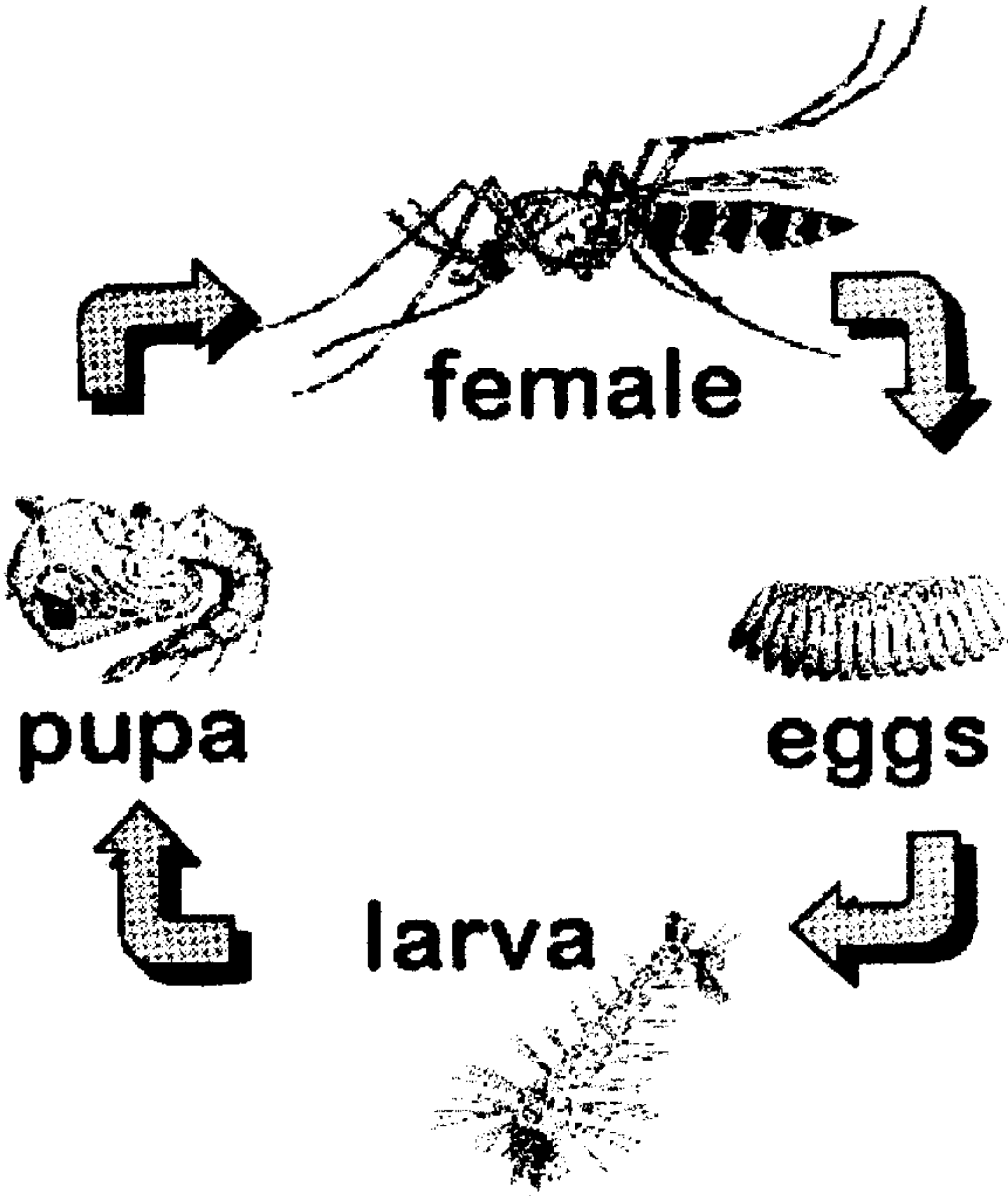
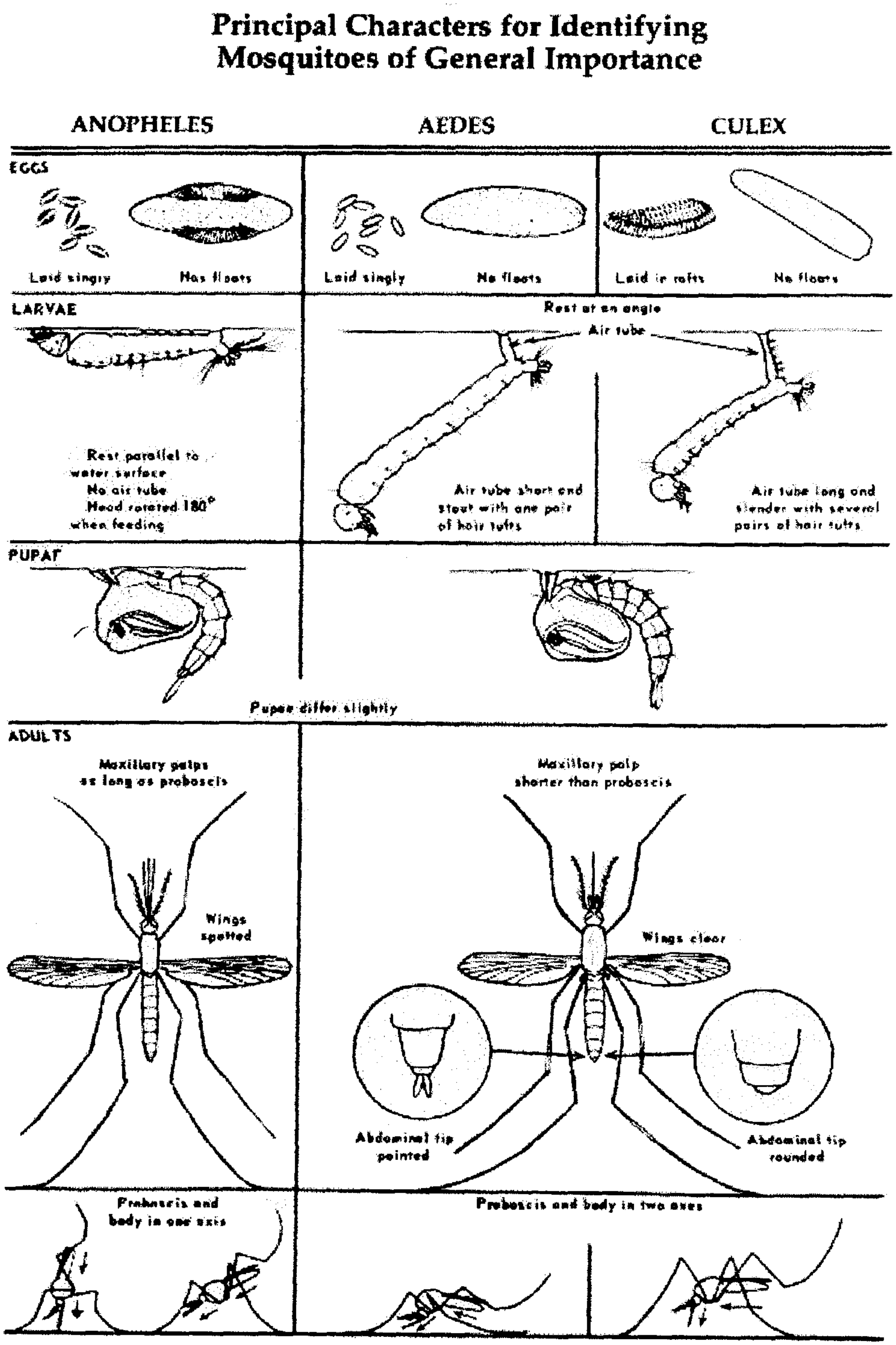


Fig. 1: Life cycle of a mosquito

Fig. 2


Principal Characters for Identifying
Mosquitoes of General Importance
ANOPHELES
AEDES
CULEX
EGGS
Laid singly
Has floats
Laid singly
No floats
Laid in rafts
No floats
LARVAE
Rest at an angle
Air tube
Rest parallel to water surface
No air tube
Head rotated 180°
when feeding
Air tube short and stout with one pair of hair tufts
Air tube long and slender with several pairs of hair tufts
PUPAE
Pupae differ slightly
ADULTS
Maxillary palps as long as proboscis
Maxillary palp shorter than proboscis
Wings spotted
Wings clear
Abdominal tip pointed
Abdominal tip rounded
Proboscis and body in one axis
Proboscis and body in two axes

Section -II: DRWH and Mosquito Breeding

It is known that any water source is a potential ground for breeding of mosquito and hence adequate precaution for mosquito control must be taken in designing DRWH. Available reports on this aspect are chronologically presented below.

* One of the first references available is from engineering construction, malaria control bulletin. Though not in the context of DRWH, the authors describe a mosquito proof domestic cistern, with protection of the opening from entry of mosquitoes. [(Health Bulletin No. 32 (Malaria Incidental to Engineering Construction) (see Appendix-1)].
* Waller (1989) in a study on RWCS in Nova Scotia, Canada, and Bermuda, asserts, that if the cistern surface is not covered, open storage may provide a breeding place for mosquitoes, which may act as vectors for diseases including haemorrahagic fever. Exposure to sunlight may promote algal growth. He recommends disinfection of cistern water using UV light including occasional addition of chlorine. Filtration devices have been used in some systems, but these should be properly constructed, designed and maintained.
* In a discussion on how mosquito breeding can be prevented, Noriyuki Fukano (1994) has suggested installation of insect -proof mosquito nets or other insect proof devices in the vent routes (drain pipes, overflow pipes etc.). If tank water is continuously consumed and replenished, no batch of water would stay in storage for a long time. Therefore, even if mosquito enters into the tank and spawns eggs, there is virtually no likelihood of mosquito proliferation. However this is not always the case in rain water storage. Also mosquitoes breed easily in rain water drainage pits. As rain water from roofs and the ground surface drain into sewers many larvae accumulate. To change conventional rain water drainage pits to ones made of permeable matter is a good solution to avoid mosquito problems, even if some cost is involved.
* Commenting on prevention of mosquito breeding, Appan (1994) recommended that a mosquito proof device has to be installed between the filter and the rain water storage tank. He presented data on the use of simple and inexpensive roof water collection systems in some South East Asian countries. Collected samples in most locations were positive for Total and Faecal Coliform. It is proposed that collected roof water be boiled, disinfected with household bleach or subjected to radiations from sunligh. In a certain RWH systems in Indonesia ,fishes are being reared to prevent mosquito breeding in tanks .

* According to Daulat et al. (1997) sometimes the water in the container becomes contaminated by toads, frogs, mosquitoes, cockroaches and rats. He suggested that the lid of the containers should be kept closed at all times or polyethylene sheet may be used to close the mouth of the container to prevent breeding of mosquito and other insects. Periodic cleaning is needed.
* Bambrah & Haq (1997) pointed out that a tight cover ensures dark storage conditions preventing growth of algae and breeding of mosquito. Open containers or storage ponds are generally unsuitable for storing drinking water.
* Hartung in his study at Rwanda (1999) suggests that the tank should be tightly closed to prevent mosquito breeding in RWH tanks. He is of the opinion that water in the tanks should not be exposed to direct sunlight in order to avoid algal growth.
* Gould's (1999) in the chapter on `Rainwater Quality Issues' has highlighted `Mosquito breeding and control. The relevant details are presented in Appendix 2.
* Malaria Research Centre is an apex body in India working on the prevention and control of mosquito and malaria. Details on how mosquito can be prevented by physical, chemical and biological methods are summarized in table 2. Some of these methods may prove useful in designing DRWH.

Table 2 General measures for Mosquito Control *

CONTROL MEASURES	*ANOPHELES*	*AEDES*	*CULEX*
Physical	Adults: House screening and mosquito nets (1.5 mm size or 25/26 mesh holes). Recommended doses for impregnation are: Permethrin: 0.2-0.5 gm/sq. m Deltamethrin: 0.025 gm/sq. m Cyfluthrin: 0.05 gm/sq. m Larvae: oiling and environmental management	Environmental management Container management • Elimination of breeding sites • Preventive measures for breeding in containers	Environmental sanitation Mosquito proofing
Chemical	Larvicides: abate (temophos) & chloropyriphos Adulticides: pyrethrins, Spraying and residual insecticides, repellants are also used for personal protection	Chemical larviciding: 1% temephos (Abate) granules @ 0.1 g/l Focal sprays (Pyrethrins-0.1 %) Insecticide treated curtains/nets Thermal fogging ULV	Chemical larviciding: Temephos, Fenthos, Chlorophyriphos etc. ULV thermalfogging
Biological	• Larvivorous fishes (*Gambusia affinis, Lebister reticulatus*) • Invertibrate predators Microbial insecticides: Bti (*Bacillus thuringiensis*) & B.S (*Bacillus sphareicus*)	• Larvivorous fishes • Bti (*Bacillus thuringiensis*) • Invertebrate predators such as mesocyclops	• Larvivorous fishes (*Poecilia reticulata*) and *Oreochromis mossambicus* (Peters) control breeding in cow dung pits. • Bti (*Bacillus thuringiensis*) & B.S (*Bacillus sphareicus*)

* These are general control measures. Not all of these could be considered safe for DRWH

REFERENCES

1. Malaria incidental to engineering construction, Health Bulletin No.32, Malaria Bureau No.12, by Col. Jaswant Singh and I.M. Puri. pp. 63.
2. Water -An alternative source in developing and developed countries. D.H. Waller. Water International, 14 (1989), pp. 27-36.
3. Tokyo International Rainwater Utilization Conference,Sumida city, Summer,1994, pp 14.
4. A. Appan, The Journal Royal Society of Health. October, 97, Vol. 117 No. 5, pp 319-323 Roof water collection systems in some southeast Asian countries: Status and water quality levels'.
5. M.D Daulat Hussain, MD. Ahiduzzaman, Thomas Rozario, Proceedings of the 8th International Conference on Rainwater Catchment Systems, April 25-29, 1997, Tehran, Iran, pp 648-653.
6. G.K. Bambrah and Ms. S. Haq. Quality issues in rainwater harvesting in Kenya. Proceedings of 8th International Conference on RWCS, April 25-29th, 1997, pp 552.
7. Rainwater harvesting in resettlement villages of Rusamo/Rwanda. Hans Hartung, September 1999, pp 1-8.
8. Rainwater catchment systems for domestic supply, by John Gould and Erik Nissen -Petersen, IT Publications Ltd., 1999.

Mosquito breeding and control

The three main groups of mosquitoes and their associated diseases are *Aedes* associated with dengue and yellow fever, *Anopheles* with malaria and *Culex* with filariasis and extreme nuisance. Larvae from members of all three groups may be found in tanks containing stored rainwater, especially in tropical regions. It is the *Aedes* group, and particularly *Aedes aegypti*, the vector responsible for transmission of dengue and yellow fever, which is most commonly found in water storage containers both large and small in and around the house. While evidence suggests that covering containers can significantly reduce the prevalence of mosquito larvae in the water, it is difficult to prevent the problem completely.

In a survey of 150 households in three villages in Khon Kaen Province, Thailand, *Aedes* mosquito larvae were present at all households. They were found in 95 per cent of small indoor clay storage containers, in 32 per cent of all rainwater jars and 4 per cent of rainwater tanks (Chareonsook et al., 1985). Hewison and Tunyavanich (1990) reported that villagers complained of increased mosquito infestation in north-east Thailand following the widespread introduction of ferrocement water jars. While this may have been due to the increased availability and presence of water in the village, it also seemed to be much worse where containers were not covered. To reduce the problem significantly it was recommended that all containers, large and small, and both inside and outside the house, must be covered and screened with some form of mesh. In Queensland, Australia, a survey of 1349 premises revealed that rainwater tanks provided important breeding sites for immature *Aedes aegypti* mosquito larvae (Tun-Lin et al., 1995). A study conducted in two villages in southeast Nigeria, where all households use rainwater stored in earthen ware pots around the house, revealed the widespread presence of *Aedes aegypti* mosquito larvae. In this case, the vector was associated with the transmission of yellow fever, and between 53 and 76 containers per 100 households were found to be infested during the wet season (Bang et al., 1981).

Various approaches to mosquito control can be used. The addition of small amounts of domestic kerosene (5ml per 1000 litres) works well, but can give the water an unpleasant taste and may not be suitable for tanks lined with plastic. [Note: Commercial or industrial kerosene should *not* be used]. Various forms of biological control, such as keeping fish and dragonfly larvae in the tanks to consume mosquito larvae, have also been tried with some success (Skinner, 1990; Corbet, 1986). While biological control may be a useful and effective tool at specific locations over limited time periods, the best guarantee against preventing mosquitoes from laying eggs in rainwater jars, tanks or other water storage containers is to make sure they are inaccessible. To exclude mosquitoes, containers must be tightly covered and any openings properly screened with fine nylon or metallic mesh. Nevertheless, regular inspection is also essential to alert users to potential problems and should prompt immediate action when necessary. Leaving rainwater tanks, jars and in-house water storage containers unscreened or uncovered in areas where malaria, dengue or yellow fever are endemic is to court danger.

Section III: Designing of Mosquito Control in DRWH

Based on the literature review, it is seen that mosquito control measures have to be applied at all stages of the mosquito life cycle, with suitable ovicidal, larvicidal and adulticidal interventions. (see Figure 4). The preventive measures with regard to DRWH may be divided into three groups:

i) Prevention of mosquito breeding in the surroundings.

- Appropriate physical, chemical and biological means of control can be used.
- Plants which repel mosquitoes can be grown around the DRWH site.
- If there are depressions in the surrounding B.S., BTI, larvivorous fishes, aquatic plants, plant extracts (oil layer), kerosene oil etc. can be used. Some of the well tested chemicals may also be used.

ii) Prevention of mosquito breeding in the DRWH system.

- Avoid all the factors which result in attracting mosquitoes.
- Tightly closed lids may be provided to the water storage system, so that there are no openings for the entry of mosquitoes.
- Screen may be used (with hole size less than 1 mm) to bar entry of mosquito larvae.
- Filter should be disinfected (eg. with household bleach).
- No stagnating water should be allowed around the DRWH site.
- Gutter leading to the storage should have a free flow of water.

iii) If inspite of the above, eggs or larvae have entered DRWH, various ovicidal and larvicidal measures have to be considered.

- The measures practiced for killing bacteria may result in dying of mosquito eggs and larvaee. These include high temperature, boiling and use of botanicals.
- Algal bloom both promote or discourage mosquito larvae depending on certain conditions.
- Measures for protection from mosquito biting should be undertaken.

Suggestions for further work

Some of the above ideas on mosquito control are being evaluated on an experimental scale under the project. Following environmental friendly mosquito control measures for preventing the entry of eggs, larvae and adult mosquitoes are being integrated in DRWH design:

- Prevention of breeding in the surroundings using certain aquatic and terrestrial plants which deter mosquitoes. In this context use of certain plants and extracts used traditionally in mosquito control would be examined.
- Screening of the entry points by mesh / nylon of appropriate mesh size (size of holes<1mm).
- Filter and filter treated with bleaching powder, at the entry point for water storage.
- Use of food grade plastic sheet/beads and botanical extracts for topping the water and prevent development of mosquito larvae.

C-3 Report by IIT Delhi, July 2000

WATER QUALITY IN DOMESTIC ROOFWATER HARVESTING SYSTEMS (DRWH)

ABSTRACT

The biological and chemical quality of water samples from experimental DRWH set up at IITD, New Delhi were analysed. Data reported in the literature on DRWH water quality were compiled by world region.

Based on experimental and compiled data it is seen that physico-chemical quality of water is generally acceptable and this can be easily monitored using field kits.

As for biological quality, there is a need for further discussions at global levels, on the indicator bacteria used for testing rain water. Under the current project, water quality from different types of experimental DRWH systems was tested by rapid field tests and compared with lab tests by standard procedures. H2S strip test is suitable at field level for detection of high level bacterial contamination. In 70% of the samples tested the bacterial content exceeded the potability standards. Avoiding the first 2 mm of rain as first flush was found to reduce contamination. It appears advisable to treat the water from DRWH before drinking.

Bacterial growth and decline, kinetics of bacterial decay, conditions for algal growth in water stored in DRWH, are also under study and preliminary results are reported herein.

CONTENTS

1 INTRODUCTION

1.1 Background

The sub-programme under the project on DRWH, namely, "Health issues related to DRWH" is led by I.I.T., Delhi. The focus is on the following issues:
1) To see if the quality of water in DRWH storage meets potable water standards.
2) To study various design parameters which influence the quality of water.
3) Suggest modifications and practices to be introduced to make the quality of water acceptable.
4) Suggest design modifications and practices needed for insect/mosquito control.

Early in the current year the C-2 report on "DRWH and insect vectors: A literature review" was submitted.

The following activities were taken up with respect to water quality:
a) The review of literature on "Water Quality in DRWH systems" was updated.
b) The efficacy of various measurement techniques for determining water quality was experimentally examined. The results for different types of tests were correlated.
c) DRWH systems with different design modifications were set up and water quality was monitored and related to design aspects.

The results of water quality tests are presented in this report (C-1). The water quality in DRWH as reported by different researchers from different countries is also recorded. Some additions and modifications have been made to this report based on additional information obtained in the current year. Experimental facilities were set up for rapid and in depth analysis of inorganic, organic and biological contaminants in water.

1.2 Water quality in DRWH

As seen from the literature survey the percentage of DRWH samples which meet potability standards vary from 10-70% in terms of bacterial quality. Thus in the worst case 90% of the samples do not meet the standards (Appendix I). Clearly in no case 100% of the samples were found to meet bacteriological standards. On the other hand chemical quality of DRWH samples were acceptable in majority of cases except in the of first flush, and also when toxic metals or chemicals from roof or the atmosphere, was there.

Hence, DRWH would be especially useful where the ground water is highly contaminated by chemicals such as arsenic, fluoride or other chemical contaminants, but the bacteriological quality would still require monitoring. In this context rain water harvested from the roof directly would indeed be a valuable source. It is relatively easy to eliminate bacterial contamination by boiling, but elimination of dissolved chemicals even in small concentrations is quite costly.

Under the project it was proposed to systematically test the biological quality of water from different DRWH systems, using different measurement techniques, and look for correlations. Among bacteriological contaminants, total and faecal coliform, faecal streptococci and E. coli are used as indicator bacteria for faecal contamination. H_2S producing bacteria can also be monitored. Details on H_2S strip tests, MPN and other standard methods were presented in Report C-1. In addition the following tests which are useful in the detection of biological contamination are available. Details of these are appended to report C-1:
i) Coliphage detection test
ii) β Galactosidase/enzymatic test
iii) Viral detection in water

The biological quality has to be tested immediately after sampling. In this regard rapid tests are very useful. Different authors have used different tests as were described in the Report C-1. Of these, the H_2S strip test is considered promising. In the present study, water quality as measured by the H_2S test was compared with results obtained by laboratory MPN method/plate count method. Based on this the reliability of the rapid test was examined.

1.3 WQ measurements in experimental DRWH systems

Various design parameters have been identified as contributing to the contamination of rain water. Specifically the effect of the following needs attention:

i) whether there is first flush diversion.
ii) the types of roof, gutter, tank, filter
iii) the nature of the roof's surroundings: e.g. overhanging trees, accumulation of leaves, animal and bird droppings

Five DRWH systems with varying design features were constructed on the I.I.T., Delhi Campus. The salient features of these systems are shown in Table 1.

2 BIOLOGICAL QUALITY OF WATER SAMPLES

2.1 Indicator bacteria and correlations between tests using them

It may be noted that the H_2S producing bacteria like *Clostridia perfringens* are normally present in faeces. Though in much smaller number as compared to *E. coli*, they can survive in water longer than coliform bacteria. The H_2S strip test depends on their presence. Although the H_2S strips have a good shelf life, it was noted that, while freshly prepared strips become black in 24 hrs in the presence of H_2S producing bacteria, in the case of shelf-stored strips an incubation of 48 hrs may be needed. Hence, while establishing a correlation between the results of H_2S strip tests and MPN tests, a time period of 48 hrs was chosen.

The ratio of faecal coliform to faecal streptococci gives an idea of the source of faecal contamination. If FC:FS < 1, it suggests that the contamination is of non-human origin i.e. animals and birds. But if FC:FS > 4, it is implied that faecal contamination may be of human origin, which could also be secondary. Primary contamination by human excreta may however be ruled out where the roof is inaccessible to humans. It may be noted that in stored water samples, streptococci can produce enough lactic acid and other organic acids from sugar fermentation that their medium becomes acidic and their growth is inhibited. This may lead to faster reduction in streptococci as compared to coliform, thus raising the FC:FS ratio for the stored sample above that in the fresh sample. Hence the time elapsed between water contamination and testing becomes crucial.

Water samples from each of the tanks were analysed at different times to see the water quality, with or without intermittent rains. The samples were drawn in sterilised bottles and tested for potability. A total of 54 samples were analysed. The results are compiled in tables 2A, 2B, 2C, 2D and 2E respectively for tanks MAA, MAP, BLC, BLT and MIC (the tank features are shown in table 1). Table 2F shows some rainfall data. The data for the first flush samples are shown in table 3A. As expected, the bacterial count in the first flush was very high.

H_2S strip test is an excellent test suitable for field level as it is easy, cheap and the villagers can see the black colour. MPN test is quantitative as it gives the most probable count of bacteria. The correlation between rapid H_2S strip test and various MPN test was examined (tables 3B-3F).

P_1 signifies a positive correlation when both the tests give negative results i.e. bacterial count below limits specified.
P_2 is a positive correlation when both the tests give positive results i.e. indicate bacterial counts above the limits specified.
N_1 signifies a situation where strip test does not give black colour in 48 hrs but MPN shows a bacterial count.
N_2 is signifies a situation where H_2S strip test gives black colour in 48 hrs but MPN does not give a bacterial count.
Thus N_1 and N_2 stand for negative correlation between the two tests.

Out of the total different samples analysed, 65-75% gave a positive correlation ($P=P_1+P_2$) between MPN and H_2S strip tests; the correlation is slightly lower where an MPN-FS test is used than when MPN-FC or MPN-TC is used. The correlation is generally higher at low levels of bacterial contamination, i.e. in most cases $P_2>P_1$.

In the remaining (35-25%) cases ($N_1 + N_2$), the two tests did not correlate (table 3B). Out of these 15-25% gave positive results for MPN but negative ones for the H_2S strip test (N_1) and ~15% gave negative results for MPN test but positive ones for H_2S (N_2). In other words in ~20% of cases even though the H_2S test shows that the water is potable this is really not so in terms of MPN. This negative correlation is more pronounced when bacterial counts are low. Roger Fujioka has evolved an MPN test based on H_2S strip method under the project. We will also evolve a MPN test based on H_2S and study the correlation with MPN for other bacteria.

Out of the water samples tested on the current project, only 13% of the samples met WHO standards for all the three indicator bacteria as well as H_2S test, and at best 25-30% samples met the relaxed standards (Table 3F). In a report (see idrc.ca/library document 008918) from International Development Research Centre (IDRC), the H_2S strip test and MPN tests with regards to total coliform and faecal coliform have been compared. The samples analysed in this report included rain water, shallow well water, pond water and deep well water. About 39% of the rain water samples were found to meet the potability standards, while for shallow well and deep well the potable samples were 3.4 and 83.3% respectively. Thus rain water was much better than shallow well water, whereas deep well water was the best. It is noted that contamination in case of rain water may be due to birds and rats. In this report correlations between H_2S and MPN tests are also presented. This varies with the type of water, i.e. shallow-well, deep-well or rain water, reflecting perhaps on the differences in the bacterial consortia present in the sample. Further work is needed in this direction.

2.2 Correlation between turbidity and number of bacteria

Pure E. coli was inoculated in 1,000 ml of sterile water and serial dilution of it was performed, as indicated in Table 4A. Each of the diluted samples (standard samples) and rain water samples were spread plated on nutrient agar plates and incubated at 35°C for 24 hrs. The number of colonies were counted in a cell counter. For the same samples turbidity was measured in a nephelometer.

As seen from the table, a rainwater sample with a low NTU of 1 had 1.632×10^6 bacterial colonies, whereas sterile water with a NTU of 1.5 had no colonies. This is because suspended colloidal particles and organic matter, contribute to turbidity besides bacteria. If turbidity was only due to bacteria even at 5 NTU the bacterial count could be greater than 3×10^7.

2.3 Decay rate of total coliform, faecal coliform and faecal streptococci in water samples stored in the tanks and in PET bottles

Three tanks were chosen from different locations having different number of initial bacterial count.

i) MIC, a cement tank with high bacterial count.

ii) BLT, a segmented cement tank at Institutional Area with high bacterial count.
iii) BLC, a cement tank at the Institutional Area with low bacterial count.
1,000 ml of samples were drawn from each into sterilised bottles.

66.6 ml of the sample was subjected to MPN test for detecting the presence of coliform group of bacteria and streptococci. The procedure was repeated everyday to observe the fall of bacteria with respect to time period. The results are shown in Tables 5A, 5B and 5C for MIC, BLT and BLC respectively. A decay rate curve was plotted between the time period (in days) and the MPN. It was observed that the total coliform and faecal coliform count fell in 20-25 days and faecal streptococcus fell in about 10-16 days. Therefore, on an average, the bacteria decay in 10-25 days of storage.

2.4 Decay rate of bacteria in the presence and absence of nutrients (Isolation of pure strains from rain water samples)

(a) Decay of faecal coliforms

Isolation of E. coli from rain water

33.3 ml of rain water was subjected to the MPN test to detect the presence of coliform in MacConkey Broth for 48 hrs at 35^{o}C in the incubator. The tube with maximum growth was taken and presence of *E. coli* was confirmed by inoculating a loop full of bacteria into the Brillinat Green Lactose Bile Broth and further grown at 35^{o}C for 24 hrs. From the BGLB tube 2 loops-full of bacteria was spread plated and streaked onto a series of nutrient agar plates and allowed to grow at 35^{o}C for 24 hrs.

The growth of 2 types of colonies typical (nucleated with or without blue green metallic sheen) and atypical (opaque, un-nucleated, mucoid, pink) were observed. From each of these plates one or more well isolated *E. coli* colonies were picked up and transferred to series of nutrient agar and Eosin methylene blue agar plates to grow separate typical and atypical colonies at 35^{o}C for 24 hrs. The process was repeated for a few generations to obtain a pure strain of *E. coli* and the presence was confirmed by performing MRVP test.

Inoculation of the pure strain of E. coli into samples with different nutrient level

Six loop fills each of the pure strain was first dissolved in 1 ml of sterile water and made up to a volume of 5 ml. This 5 ml of pure strain of *E. coli* was transferred into 1,000 ml each of (1) double distilled, sterile water, (2) rain water samples which was boiled to kill the micro-organisms and (3) rain water which was not boiled. Each of these samples were immediately subjected to MPN test and again at an interval of 4 hrs and subsequently every day to observe the bacterial decay rate with respect to nutrient availability. The data are shown in table 6A and 6B. It is observed that in complete absence of nutrient i.e. in sterilised water, the coliform bacteria survived for about 10-11 days. But in the presence of limited nutrition, the coliform bacteria decayed in 20-25 days.

(b) Decay of faecal streptococci

Isolation of faecal streptococcus from rain water sample

33.3 ml of rain water was subjected to MPN test to detect the presence of streptococcus by growing in Azide Dextrose Broth for 48 hrs at 35^{o}C in the incubator. The tube with maximum growth was taken and a loop full of bacteria was transferred on to the PSE (Pfizer Selective Enterococcus) agar plates at 35^{o}C for 24 hrs. A loop full of growth was used for generations in several plates of PSE agar to obtain a pure strain.

Inoculation of pure strain of streptococcus into samples with different nutrient levels

Six loop fulls each of the pure strain was first dissolved in 1 ml of sterile water and the volume was made to 5 ml. This was transferred into 100 ml each of (1) double distilled, sterile water (2) rain water after boiling to kill the micro-organisms and (3) rain water without subjecting to boiling.

Each of these samples were immediately subjected to MPN and again at an interval of 4 hrs and subsequently everyday to observe the bacterial decay rate with respect to nutrient availability. In case of the faecal streptococci the experiments indicated that in the total absence of nutrient i.e. in sterilised water, the streptococci count fell within 2 days. But in the presence of limited nutrition the streptococci count fell over 17-20 days approximately. The data are presented in Table 6.

(c) Decay rate constant

As discussed earlier the coliform group of bacteria decays at a slower rate as compared to the faecal streptococcus. It has been reported that the faecal streptococci when provided with sufficient nutrition grow abundantly and in this process the streptococci release toxins which inhibit the growth of their own kind. As a result, the decay rate of streptococci is much faster than the coliform group.

When the decay rate was studied for the same group of bacterium in the total absence of nutrient it was observed that due to self generated toxicity the streptococci count falls rapidly but the coliform group survive. In the presence of limited nutrition, both the bacteria fell at a slower rate as compared to the rate of decay in the total absence of nutrition. This trend was seen even on boiling the water and inoculating pure culture, the dissolved solids (nutrients) are not eliminated by boiling. The decay rate constant (often expressed as the generation/hour) was calculated.

In our experiments, the fall of bacteria under different conditions was studied and the decay was noted in (i) 1,000 ml of sterile water (ii) a sample of stored rain water from the storage tank MAP (iii) a sample of the same rain water, which was boiled to eliminate the existing bacteria (see table 6A & 6B). A graph was plotted between number of days (time period) and number of cells (MPN count). The rate constant/decay constant was found to approximately -1 generation/ day (see table 6C).

2.5 Testing for heterogeneity of the samples with respect to bacterial distribution

The following test was performed to see the distribution of bacteria on storage. Two types of samples were taken as described below:

i) Sample was kept standing for 24 hrs and water drawn from different heights/levels of the bottle.
ii) Sample was shaken vigorously and water drawn in triplets.

1,000 ml of rain water was collected and stored in two PET bottles up to a height of 18 cm in each case. Markings were made at distances 1 cm (L3), 8 cm (L2) and 16 cm (L1) respectively from the base. One of the bottle was kept standing. A 50 ml sample was withdrawn from different levels and taken into separate sterilised bottles. The second bottle of rain water sample was vigorously shaken and samples were randomly drawn from the PET bottle in triplets to observe the distribution of bacteria. All the samples drawn were subjected to the standard methods of MPN test and H_2S strip rapid test. The results are shown in table 7.

It was seen that on mixing the water by shaking, an average count of TC (46), FC (240) and FS (0) was seen. On the other hand on allowing the water to stand, there is a gradation in bacterial counts with more bacteria at the lower level closer to the base. But the gradation was not linear with depth. Further work is needed in this direction.

2.6 Water treatment methods

Since, it was observed that rainwater was found potable only 30% of times even with relaxed standards, the following physical treatments methods were applied to bring it to potable standards:

i) Heating the water at 100^oC or 60^oC
ii) Exposing the transparent storage bottles to sunlight and UV.

Rain water with a TC count of 1,000 and FC count of 240 from BLT-78 was used for the experiments for 15 minutes (table 8A). To see the effect of duration of heating, rainwater from the storage tank BLC was taken in 12 (200 ml each) sterile conical flask and subjected to different heating conditions i.e. boiling at 100^oC for 1, 5, 15 and 20 minutes and heating at 60^oC for 1, 5, 10, 15 and 20 minutes. Simultaneously, control was also taken where no heating was carried out. The water samples were subjected to H_2S strip test and MPN test for total coliform, faecal coliform and faecal streptococcus. The results (Table 8B & 8C) show that boiling even for one minute is sufficient to eliminate total coliform, faecal coliform and faecal streptococcus. The H_2S strip test which gave black colour in 24 hrs when no treatment was done, did not give black colour when the rainwater was boiled for one minutes. Heating at 60^oC for 1 minute eliminated total coliform and faecal coliform but faecal streptococci survived and were eliminated only on heating for 15 minutes. However, H_2S strip did not give black colour after 1 minute of heating at 60^oC.

In another treatment method the storage bottles were exposed to sunlight under different conditions. The samples from storage tank MAA was drawn 3.3.2000 and 11.4.2000 in sterile bottles of volume 200 ml, were kept in sun over for 16 hours over two days under the following three conditions. (a) covering the bottles totally with carbon paper, (b) covering them partially by carbon paper and (c) bottles were left totally transparent without any covering (d) Bottles kept inside with no treatment of sunlight were used as control.

The water samples were subjected to standard H_2S strip test and MPN test for total coliform, faecal coliform and faecal streptococcus. It was observed that covering the bottles partially with carbon gave good results (table 10) as it eliminated total coliform, faecal coliform and faecal streptococcus for both the samples drawn on 3rd March and 11th April. H_2S strip also gave negative test in these cases. In water samples from the bottle which was totally covered with black carbon in one case, the H_2S strip became black in 24 hrs. and TC, FC and FS persisted. In this case the temperature was only 27^oC. In the second case, the temperature in the bottle reached 45^oC. The H_2S strip gave negative result and only faecal streptococcus was seen. Transparent bottles showed better results than totally covered bottle. Thus the effect of sunlight depends on two parameters (a) heating and rise in temperature due to this and (b) exposure to UV rays. Partially covered bottles get heated and also receive UV rays of the sun, whereas the totally covered bottles get heated more but UV rays are blocked. SODIS (Solar Water Disinfection) has also been reported in the literature. Water is disinfected by radiation and by solar thermal water treatment by exposing small to full sunlight for 5 hrs or for 2 consecutive days even with 50% cloudy sky.

2.7 Algal growth in rainwater samples

It was observed that rainwater samples stored over a period of 3-6 months in transparent PET bottles and glass bottles, and exposed to sunlight supported algae growth, when sufficient nutrition was available. On the other hand sterile water (autoclaved) kept in transparent glass/PET bottles as control had no algal growth. Samples subjected to boiling were also devoid of any algal growth. When sample bottles were completely blackened by carbon paper there was no algal growth. The data are reported in Table 9. Thus both sunlight and nutrients are needed for algae to grow. To prevent algal infestation, the tanks must be kept closed without exposure to sunlight.

3 CHEMICAL QUALITY OF WATER SAMPLES

The essential chemical characteristics such as pH, EC (electrical conductivity) TDS (Total Dissolved Solids), TH (total hardness) and content of Ca^{+2}, Na^{+1}, NO^{-}_{3} and Cl^{-} are reported for storage tanks labelled MAP, MIC, BLC and BLT and MAA respectively (tables 12A, 12B, 12C, 12D, 12E). These were measured by rapid method in kits and also detailed analysis was undertaken – see Tables 12A-12D. The two data are compared. Essentially the chemical water quality was acceptable. In all the cases no significant differences could be seen with material and design changes in roof, gutter and storage type.

Specifically the following conclusions were reached concerning the chemical/physical quality of stored roofwater:

- It was be seen that the collected samples were generally clear without colour. Only a few samples had a yellowish hue when contaminated due to leaves from overhanging trees. Odour and taste wise also the water samples were acceptable.
- The pH range of water samples measured by a pH meter was in the range 7-8.5 and the values obtained by rapid testing kit was also in range 7-8. Thus the pH was well within the acceptable limits.
- Chloride and total hardness measured by laboratory method and by testing kits were in agreement. The values were below limits allowed for potability within the experimental error.
- The TDS and all other cations/anions concentrations were within acceptable limits.
- On storage in ferrocement tanks, no significant change in TH/TDS/pH was observed.
- Out of five DRWH, only in one case, the first flush showed a high TDS/TH value. In the other cases, even the first flush had acceptable TDS.

Note: Detailed procedures for using laboratory and rapid tests are described in report C-1.

4 SUMMARY AND CONCLUSIONS

Analysis of literature data available on the quality of water stored in DRWH systems, field level survey as well as experimental work done under the current project, indicate the following:

1) Generally, the physico-chemical quality of water in terms of colour, odour and taste, pH, total dissolved solids (TDS) and total hardness (TH), meet the prescribed standards. Occasionally pH has been reported to be low (acidic) or high (alkaline).
2) Toxic metal ions and toxic chemicals are reported only in rare cases and may arise from material used for the roof or atmospheric pollutants adsorbed on dust.
3) Most of the material used for storage tanks e.g. cement, iron, wood and plastics do not negatively affect the physico-chemical quality, with a few exceptions.
4) The physico-chemical parameters can be tested easily by using available field kits.

On the other hand, the main problem with the quality of stored water in DRWH lies with its bacteriological quality. The following are the main issues:

1) Total coliform (TC), faecal coliform (FC) and faecal streptoccocus (FS) are used as indicators of faecal contamination. Total coliform test is rendered difficult in the presence of large counts of other bacteria. Hence, E. coli is considered a better indicator. All the bacteria can be quantified at the lab level by the most probable number (MNP) method and plating in specific media after micro filtration and counting the colonies. It is not easy to perform these tests at the field level. The rapid tests available at field level are essentially useful for indicating the presence/absence of bacteria. The H_2S strip test based on production of H_2S by sulphur reducing bacteria such as *Clostridum perfringens* is considered most useful as it correlates well with plate count or MNP test for faecal coliform.

2) Dust from the soil, and droppings of birds and animals can also be the source of contamination by the above bacteria. Thus, the indicator bacteria need not necessarily be of human origin. It is reported that faecal streptococcus is more in bird/animal dropping. Thus if FC/FS ratio less than 1, contamination may be of non-human origin, and if FC/FS is greater than 4, then contamination may be of human origin. However, it may be noted that on storing rainwater, FC & FS counts fall at different rates. As FS counts fall faster, FC/FS ratio may increase with storage.
3) In any case where first flush eliminating devices are absent, all the indicator bacteria are generally present in water samples in numbers beyond what is acceptable by any standards. First flush is invariably contaminated. Since bird dropping and dust particles do not depend on roof type, it is difficult to categorically state whether any one type of roof is better than the other. However, experiments under the current project show that contamination from brick tiled roof was more than with other kind roof, like plastic, metallic and asbestos roof. Also it is possible that a rough surface may carry more dust and thus cause greater contamination. Higher temperature reached by a metallic roof due to solar heating may lead to reduction in bacteria. However, it is difficult to rely on such issues in designing for water quality.
4) The same goes for the gutter materials. From the health point of view it is important to clean the gutter from time to time and ensure that water does not stagnate. This leads mosquito breeding.
5) Tree hanging in the vicinity, definitely enhances the possibility of contamination due to increased access of the roof to birds and animals. Also leaves contribute to organic loading of the water samples, which in turn act as nutrient for bacterial growth.
6) On storage, generally due to limitation of nutrients, bacterial count falls. Different indicator bacteria under study decay over 7-20 days depending on the initial amount of bacteria, nutrient availability and other storage conditions.
7) Increase of temperature due to sun's heat or exposure to UV radiation of sun, reduces and ultimately eliminates bacteria. However, exposure to sunlight in the presence of nutrients can lead to algal growth, especially when the storage is open.
8) Mosquito breeding generally occurs if mosquitoes are already available in the vicinity of storage. Water quality deteriorates with the breeding of mosquito. The only way to prevent mosquito in the tank is by covering the openings by appropriate screens. The holes in the mesh should be small enough not only for preventing access to mosquitoes, but also to mosquito eggs which could be washed off.

Thus the basic conclusion from the study, substantiated by actual experimentation under the project are that DRWH must be designed, taking the following into consideration:

i) Convenient first flush device must be integrated.
ii) Storage must be tightly lidded and all entry points must be closed by a mesh to prevent entry of mosquitoes and eggs.
iii) It is preferable to allow the water to stand for some time before drawing. The bacterial count is more at the bottom. Hence the water drawn may be from a higher level, e.g. with drawing water from an over flow system may be useful. The sediments accumulated may be removed from time to time. Thus, instead of one tank of large capacity, more tanks in a series may be used, but increase in total cost has to be considered.
iv) Some rapid testing methods like H_2S test methods are useful in the field for indicating presence of biological contamination. However even, when the indicator tests are negative, it is preferable that some treatment is given for elimination of bacteria, before drinking the water. The safest methods of treatment are exposure to UV & boiling. From the angle of design parameters, a slow sand filter at inlet or outlet could be used. It would be easier to use such a filter at the outlet. However, the sand filter should be maintained properly and kept clean.

5 FURTHER WORK PROPOSED

Further research is in progress under the project to determine the following:

1) What really constitutes the first flush i.e. how much water is needed for the roof to be completely washed clean so that the subsequent precipitation of rain is clear. The design and usefulness of first flush eliminating devices integrated with DRWH, will also be examined.
2) Whether storage will lead to total elimination of bacteria in the absence of nutrients in the rain, and render the water potable. It must be noted that bacterial decay patterns vary for different pathogens and indicator bacteria. Also, so far separate tests are not being conducted for virus in water. The absence of indicator bacteria is taken to signify the absence of human contamination including human viral contamination.
3) Further studies will be conducted on settling pattern of bacteria on storage, for deciding on suitable design inputs.

Acknowledgement

The P.I. would like to thank Dr. Roger S. Fujioka, Water Research Centre and the School of Public Health, University of Hawaii, Honolulu, Hawaii, USA, for helpful discussions with regard to water quality measurements techniques, and correlating design parameters with water quality.

TABLE 1: DESIGN FEATURES OF EXPERIMENTAL DRWH SYSTEMS SET UP AT I.I.T. DELHI FOR WQM

Tank No.	Tank label	Type of Roof	Type of Gutter	Type of Tank	Filters	Over-hanging tree?	Mosquito Breeding in the vicinity?	Location
1	BLT	Tin (CI)	Alumin'm	Segmented ferrocement (2,500 l)	Cement inlet	A tree at some distance	No	Block-IV, Institut'l Area
2	BLC	Cement (planar)	PVC	Ferrocement (2,500 l)	Cement inlet	Yes	No	Block-IV, Institut'l Area
3	MAP	Plastic roof corrug'd	Alumin'm	Sintex (300 l) (LDPE)	Earthen pot inlet	No	No	Resident'l Area
4	MAA	Asbestos corrug'd	PVC	Sintex (300 l) (LDPE)	Earthen pot inlet	No	No	Resident'l Area
5	MIC	Cement (planar)	Alumin'm	Ferrocement (500 l)	Cement inlet	Yes (number of trees)	Yes	Micro-model

Roof area: Actual/inclination/horizontal projection

TABLES 2A to 2F – WATER QUALITY IN DIFFERENT TANKS

Table 2A: WQM in tank MAA

Code	Date of sample with drawing	H_2S strip				Date for experim't for MPN	MPN for E. coli		MPN for faecal strepto-coccus FS
		Date of Experim't	24 hrs	48 hrs	72 hrs		Total coliform TC	Faecal coliform FC	
1 (14)*	06.08.99	06.08.99 11.08.99	- -	+ -	 -	11.08.99	4	0	0
2 (21)*	12.08.99	16.08.99	-	-	-	30.08.99	0	0	0
3 (31)	24.08.99	24.08.99	-	-	-	30.08.99	0	0	0
4 (35)*	06.09.99	06.09.99	-	+		07.09.99	≥ 1,100	11	4
5 (54)	27.09.99	27.09.99	-	-	-	27.09.99	93	93	20
6 (62)	25.10.99	25.10.99	-	-	-	25.10.99	≥ 1,100	28	15
7 (67)	09.11.99	09.11.99	-	-	-	09.11.99	≥ 1,100	0	210
8 (82)	11.01.00	11.01.00	-	-	-	11.01.00	≥ 1,100	≥ 1,100	460
9 (83)	17.01.00	18.01.00	-	-	-	18.01.00	210	210	9
10 (87)	01.02.00	02.02.00	+			02.02.00	460	240	9
11(90)	14.02.00	15.02.00	+			15.02.00	460	460	9

Table 2B: WQM in tank MAP

Code	Date of sample with drawing	H_2S strip				Date for experim't for MPN	MPN for E. coli		MPN for faecal strepto-coccus FS
		Date of Experim't	24 hrs	48 hrs	72 hrs		Total coliform TC	Faecal coliform FC	
1 (15)*	06.08.99	06.08.99 11.08.99	- -	+ +		11.08.99	≥ 1,100	43	4
2 (22)*	12.08.99	16.08.99	-	-	+	30.08.99	4	0	0
3 (32)	24.08.99	24.08.99	-	+		30.08.99	9	0	0
4 (37)*	06.09.99	06.09.99	-	+		07.09.99	460	43	4
5 (55)	27.09.99	27.09.99	-	-	-	27.09.99	93	9	20
6 (63)	25.10.99	25.10.99	-	-	-	25.10.99	≥ 1,100	3	15
7 (68)	09.11.99	09.11.99	-	-	-	09.11.99	0	0	4
8 (81)	11.01.00	11.01.00	-	-	-	09.0100	0	0	1,100
9 (84)	17.01.00	18.01.00	-	-	-	18.01.00	0	0	9
10 (88)	01.02.00	02.02.00	-	-	-	02.02.00	0	0	15
11 (91)	14.02.00	15.02.00	+			15.02.00	4	4	4
12 (95)	07.03.00	07.03.00	-	-	-	07.03.00	4	4	210

Table 2C: WQM in tank BLC

Code	Date of sample withdrawing	H_2S strip				Date for experim't for MPN	MPN for E. coli		MPN for faecal strepto-coccus FS
		Date of Experim't	24 hrs	48 hrs	72 hrs		Total coliform TC	Faecal coliform FC	
1 (2)*	20.07.99	20.07.99	-	+			-	-	
2 (3)	23.07.99	26.07.99	-	-	-	26.07.99	1,100	0	0
3 (9)	30.07.99	30.07.99	-	-	-	04.08.99	20	7	0
4 (19)*	06.08.99	06.08.99 11.08.99	- -	+ -	- -	- 11.08.99	- 75	- 4	- 0
5 (28)*	12.08.99	16.08.99	-	-	-	30.08.99	4	3	0
6 (34)	24.08.99	24.08.99	-	-	+	30.08.99	150	150	0
7 (41)*	06.09.99	06.09.99	+	-	-	07.09.99	210	43	0
8 (57)	27.09.99	27.09.99	-	-	-	27.09.99	4	4	0
9 (65)	25.10.99	25.10.99	-	-	+	25.10.99	150	0	0
10 (70)	09.11.99	09.11.99	-	-	+	09.11.99	93	21	9
11 (72)	16.11.99	16.11.99	+			16.11.99	39	23	7
12 (80)	11.01.00	11.01.00	-	+		11.01.00	43	23	0
13 (85)	17.01.00 (rained 14.01.00)	18.01.00	+			18.01.00	1,100	1,100	1,100
14 (86)	01.02.00	02.02.00	+			02.02.00	≥ 1,100	≥ 1,100	150
15 (89)	14.02.00	15.02.00	+			15.02.00	≥ 1,100	460	43
16 (94)	03.03.00	03.03.00	+			03.03.00	460	4	43
17 (97)	24.03.00	24.03.00	+			24.03.00	≥ 1,100	4	460

Table 2D: WQM in tank BLT

Code	Date of sample with drawing	H_2S strip				Date for experim't for MPN	MPN for E. coli		MPN for faecal strepto-coccus FS
		Date of Experim't	24 hrs	48 hrs	72 hrs		Total coliform TC	Faecal coliform FC	
1 (1)*	20.07.99	20.07.99	-	+					
2 (4)	23.07.99	26.07.99	-	-	-	26.07.99	0	0	0
3 (27)*	12.08.99	16.08.99	-	-	+	30.08.99	0	0	0
4 (33)	24.08.99	24.08.99	-	+		30.08.99	43	9	0
5 (40)*	06.09.99	06.09.99	+	-	-	07.09.99	≥ 1,100	240	4
6 (56)	27.09.99	27.09.99	-	+		27.09.99	39	4	9
7 (64)	25.10.99	25.10.99	-	-	+	25.10.99	93	7	150
8 (69)	09.11.99	09.11.99	-	+		09.11.99	1,100	23	1,100
9 (71)	16.11.99	16.11.99	+			16.11.99	1,100	1,100	1,100
10 (78)	07.12.99	09.12.99	-	+		09.12.99	1,100	240	93
11 (79)	21.12.99	21.12.99	+			21.12.99	240	0	43

Table 2E: WQM in tank MIC

Code	Date of sample with drawing	H_2S strip				Date for experim't for MPN	MPN for E. coli		MPN for faecal strepto-coccus FS
		Date of Experim't	24 hrs	48 hrs	72 hrs		Total coliform TC	Faecal coliform FC	
1 (8)*	30.07.99	30.07.99	+			04.08.99	≥ 1,100	≥ 1,100	1,100
2 (11)	03.08.99	03.08.99	-	+		04.08.99	240	4	9
3 (18)*	06.08.99	06.08.99 11.08.99	+ -	-		11.08.99	≥ 1,100	≥ 1,100	≥ 1,100
4 (38)	06.09.99	06.09.99	+			07.09.99	≥ 1,100	≥ 1,100	≥ 1,100
5 (66)	25.10.99	25.10.99	-	+		25.10.99	≥ 1,100	≥ 1,100	≥ 1,100

Table 2F: Rainfall during June, 99 - May, 2000

S.No.	Date	Type of rain
1.	20.06.99	1st rain (light)
2.	18.07.99	Heavy rain
3.	29.07.99	Rain
4.	5&6.08.99	Heavy rain
5.	11&12.08.99	Rain
6.	5&6.09.99	Heavy rain
7.	19.09.99	Rain
8.	30.09.99	Rain
9.	14.01.00	Rain
10.	12.05.00	Heavy rain

TABLES 3A to 3F - WATER QUALITY TESTS BY DIFFERENT METHODS

Table 3A: Biological quality of first flush

Code	Date of with-drawing sample	H_2S strip				Date for experim't for MPN	MPN for E. coli		MPN for faecal strepto-coccus FS
		Date of Experim't	24 hrs	48 hrs	72 hrs		Total coliform TC	Faecal coliform FC	
1 MAA-13 new roof	06.08.99	11.08.99	-	-	-	11.08.99	0	0	0
2 MAP-36	06.09.99	06.09.99	-	+		07.09.99	≥ 1,100	≥ 1,100	≥ 1,100
3 BLT-42	07.09.99	07.09.99	+			07.09.99	≥ 1,100	≥ 1,100	≥ 1,100
4 BLC-43	07.09.99	07.09.99	+			07.09.99	210	210	9

Table 3B: Correlation between H_2S strip test and MPN according to different standards

S.No.		Total Coliform (TC)				Faecal Coliform (FC)				Faecal Streptococcus (FS)			
		P_1 %	P_2 %	N_1 %	N_2 %	P_1 %	P_2 %	N_1 %	N_2 %	P_1 %	P_2 %	N_1 %	N_2 %
1.	*WHO standard* TC 0-10 FC 0 FS 0	24	46	26	4	24	46	26	4	24	41	28	7
2.	*Relaxed standard I* TC 0-50 FC 0-10 FS 0-10	26	39	24	11	39	35	11	15	32	24	19	26
3.	*Relaxed standard II* TC 0-100 FC 0-10 FS 0-10	35	39	15	11	39	35	11	15	31	24	19	16

Table 3C: Correlation between H_2S strip test and MPN test according to total coliform

	Total No. of samples	P_1		P_2		Total % $P_1 + P_2$	N_1		N_2		Total % $N_1 + N_2$
		Actual no.	%	Actual no.	%		Actual no.	%	Actual no.	%	
WHO TC: 0-10	54	13	24	25	46	70	14	26	2	4	30
Relaxed standard I TC: 0-50	54	14	26	21	39	65	13	24	6	11	35
Relaxed standard II TC: 0-100	54	19	35	21	39	74	8	15	6	11	26

Table 3D: Correlation between H_2S strip test and MPN test according to faecal coliform

	Total No. of samples	P_1		P_2		Total % $P_1 + P_2$	N_1		N_2		Total % $N_1 + N_2$
		Actual no.	%	Actual no.	%		Actual no.	%	Actual no.	%	
WHO standard FC: 0	54	13	24	25	46	70	14	26	2	4	30
Relaxed standard I or II FC: 0-10	54	21	39	19	35	74	6	11	8	15	26

Table 3E: Correlation between H_2S strip test and MPN test according to faecal streptococcus

	Total No. of samples	P_1		P_2		Total % $P_1 + P_2$	N_1		N_2		Total % $N_1 + N_2$
		Actual no.	%	Actual no.	%		Actual no.	%	Actual no.	%	
WHO FS: 0	54	13	24	22	41	65	15	28	4	7	35
Relaxed standard I or II FS: 0-10	54	17	31	13	24	56	10	19	14	26	44

Table 3F: Acceptable samples which fulfilled all the parameters for potability

S.No.	Standard	Total No. of samples	No. of samples acceptable	% of acceptable samples
1.	WHO TC: 0-10 FC: 0 FS: 0	54	7	13
2.	Relaxed standard I TC: 0-50 FC: 0-10 FS: 0-10	54	15	28
3.	Relaxed standard II TC: 0-100 FC: 0-10 FS: 0-10	54	16	30

Table 3G: Coliform and faecal coliform count in rain water as reported in IDRC report

No. of coliform per 100 ml of rain water	Total coliform count				Faecal coliform			
	MPN total coliform positive		Hydrogen Sulphide Test		MPN faecal coliform positive		Hydrogen Sulphide Test	
	No. of samples	%	+	-	No. of samples	%	+	-
Rain water (54 samples) 0 or < 2 = 0					17	32	10	7
0 - 10	21	38.9	11	10	5	10	-	-
11 - 100	15	27.8	-	-	16	30	-	-
101 - 2,400	18	33.3	11	22	16	30	-	-

Table 4: Correlation of turbidity and number of colonies

S.No./ Code No.	Bacterial dilutions (20 ml samples mixed as shown)		Turbidity* (NTU)	No. of colonies in 20 ml
	Bacterial solution ml	Sterile H_2O ml		
A (1)	0	20	1.5	0
M (2)	20	0	50.0	Non-countable - colonies forming a lawn
B (3)	0.5	19.5	2.0	5632×10^3
C (4)	1.0	19.0	3.0	5824×10^3
D (5)	2.0	18.0	4.4	27648×10^3
E (6)	3.0	17.0	5.9	53568×10^3
F (7)	4.0	16.0	7.0	Non-countable - colonies forming a lawn
G (8)	5.0	15.0	9.0	"
H (7)	6.0	14.0	10.0	"
I (8)	7.0	13.0	16.0	"
J (9)	8.0	12.0	20.0	"
K (10)	9.0	11.0	20.0	"
L (11)	10.0	10.0	25.0	"
(N) Bacterial (12) 1,000	Bacterial 1,000 NTU equivalent	-	200.0	"
Samples from rainwater tanks				
MAA-100			1.9	418×10^3
MAP-101			1.4	3024×10^3
MIC-18 (yellow & dirty water)			1.0	1632×10^3

* 10 NTU is the maximum permissible limit according to Rajiv Gandhi National Drinking Water Mission; 5 NTU is the maximum permissible limit according to WHO standards

Table 5A: Bacterial decay rate for water sample (MIC-66)

Days	Coliform		Faecal Streptococcus
	TC	FC	FS
1	≥ 1,100	≥ 1,100	≥ 1,100
2	≥ 1,100	≥ 1,100	≥ 1,100
3	≥ 1,100	≥ 1,100	210
4	460	240	75
5	460	240	28
6	460	210	-
7	240	93	-
8	240	93	3
9	240	93	11
10	93	43	11
11	93	23	7
12	93	9	4
13	150	9	0
14	93	4	0
15	75	9	0
16	75	4	0

Table 5B: Bacterial decay rate for water sample (BLT-71)

Days	Coliform		Faecal Streptococcus
	TC	FC	FS
1	1,100	1,100	1,100
2	≥ 1,100	≥ 1,100	93
3	≥ 1,100	≥ 1,100	150
4	≥ 1,100	≥ 1,100	14
5	≥ 1,100	≥ 1,100	23
6	≥ 1,100	≥ 1,100	23
7	≥ 1,100	≥ 1,100	43
8	1,100	1,100	43
9	1,100	1,100	43
10	1,100	1,100	7
11	1,100	1,100	23
12	1,100	1,100	23
13	1,100	1,100	23
14	1,100	1,100	23
15	460	460	23
16	460	460	9
17	240	240	4
18	240	240	-
19	240	240	-
20	210	210	-
21	210	210	-
22	93	93	-

Table 5C: Bacterial decay rate for water sample (BLC-72)

Days	Coliform		Faecal Streptococcus
	TC	FC	FS
1	39	23	7
2	75	23	9
3	43	23	9
4	39	4	4
5	43	9	4
6	43	9	4
7	43	23	0
8	23	9	4
9	15	0	9
10	7	7	4
11	0	0	0
12	4	4	0
13	4	4	0
14	4	4	0
15	93	4	0
16	9	3	-
17	4	0	-
18	4	0	-
19	4	-	-
20	4	-	-
21	4	-	-
22	9	-	-

Table 5D: Decay time constants derived from Tables 5A and 5B

(Time constant is the parameter T in the fit: $N = K.\exp(D/T)$ where N is MPN and D is time in days)

Tank	Measure	No of days	Time Constant in days	Correlation coefft R2
MIC-66	TC	14	5.31	0.86
MIC-66	FC	14	2.46	0.94
MIC-66	FS	12	2.28	0.51
BLT-71	TC	14	5.17	0.89
BLT-71	FC	14	5.17	0.89
BLT-71	FS	17	5.58	0.51

Table 6A: Decay rate of coliform group (E. coli) in absence and presence of nutrient

S.No.	Time	Sterile Water MPN	Tank Water MAP-95 MPN	Tank Water after boiling MAP-95 MPN
1.	Initial sample before inoculation of bacteria	0	4	0
2.	O minute. Sample immediately after putting the inoculum	≥ 1,100	≥ 1,100	≥ 1,100
3	4 hrs	≥ 1,100	≥ 1,100	≥ 1,100
4	1 day	≥ 1,100	≥ 1,100	≥ 1,100
5	2 days	≥ 1,100	≥ 1,100	≥ 1,100
6	3 days	≥ 1,100	≥ 1,100	≥ 1,100
7	4 days	1,100	≥ 1,100	≥ 1,100
8	5 days	1,100	≥ 1,100	≥ 1,100
9	6 days	460	≥ 1,100	≥ 1,100
10	7 days	210	≥ 1,100	≥ 1,100
11	8 days	93	≥ 1,100	≥ 1,100
12	9 days	93	≥ 1,100	≥ 1,100
13	10 days	9	≥ 1,100	≥ 1,100
14	11 days	9	≥ 1,100	≥ 1,100
15	12 days		≥ 1,100	≥ 1,100
16	13 days		1,100	≥ 1,100
17	14 days		1,100	≥ 1,100
18	15 days		460	1,100
19	16 days		460	460
20	17 days		210	240
21	18 days		240	210
22	19 days		93	93
23	20 days		93	93
24	21 days		75	93
25	22 days		43	43
26	23 days		23	15
27	24 days		4	9
28	25 days			4

Note: ‘Time constant’ (for exponential decay) is 1.44 times ‘Decay Time’ in days per gen.

Water	Measure	No of days	Time Constant in days	Correl coeft R2
Distilled water	E-Coli MPN	8	1.33	0.94
Tank water	E-Coli MPN	11	1.99	0.96
Sterilised tank water	E-Coli MPN	12	2.28	0.93

Table 6B: Decay rate of faecal streptococcus in absence and presence of nutrient

S.No.	Time	Sterile Water MPN	Tank Water MAP-95 MPN	Tank Water after boiling MAP-95 MPN
1	Initial sample before inoculation of bacteria	0	210	0
2	O minute. Sample immediately after putting the inoculum	≥ 1,100	≥ 1,100	≥ 1,100
3	4 hrs	≥ 1,100	≥ 1,100	≥ 1,100
4	1 days	1,100	≥ 1,100	≥ 1,100
5	2 days	4	≥ 1,100	≥ 1,100
6	3 days	0	≥ 1,100	≥ 1,100
7	4 days		≥ 1,100	≥ 1,100
8	5 days		≥ 1,100	≥ 1,100
9	6 days		≥ 1,100	≥ 1,100
10	7 days		≥ 1,100	≥ 1,100
11	8 days		≥ 1,100	≥ 1,100
12	9 days		≥ 1,100	≥ 1,100
13	10 days		1,100	≥ 1,100
14	11 days		1,100	1,100
15	12 days		1,100	210
16	13 days		210	93
17	14 days		43	93
18	15 days		93	75
19	16 days		9	93
20	17 days		4	43
21	18 days			43
22	19 days			23
23	20 days			4

Table 6C: Decay constant of bacteria

S.No.	Sample which was Inoculated with bacteria	Total no. of generation	Mean Decay Time	Rate Constant/ Decay constant
1.	Sterilised water	- 8.103 gen/2 days	- 1.78 hr/gen (or -0.073 day/gen)	- 0.56 gen/hr
2.	MAP-95 not boiled	- 8.103 gen/17 days	- 17.76 hr/gen (or - 0.74 day/gen)	- 1.34 gen/day
3.	MAP-95 (boiled)	- 8.103 gen/20 days	- 15.12 hr/gen (or - 0.63 days/gen)	- 1.58 gen/day

Table 7: Testing the heterogeneity of the stored samples

S.No.	Sample MAA-87	Coliform		Streptococcus	H_2S strip test
		TC	FC	FS	
1.	Shaking	460	240	9	Black in 20 hrs
2.	*L-1: 16 cm from base	240	15	0	Black in 20 hrs
3.	*L-2: 8 cm from base	1,100	1,100	7	Black in 20 hrs
4.	*L-3: 1 cm from base	1,100	1,100	9	Black in 20 hrs

* Sample was kept standing for 24 hrs and then the water drawn from different layers, L_1, L_2, L_3 . The water level is 18 cm from (above) the base.

Table 8A: A comparative study of standard treatment procedures

S.No.	Sample	Treatment	H_2S strip test	MPN for Coliform bacteria		MPN for Faecal streptococcus
				Total coliform	Faecal coliform	
1.	BLT-78* Initial	No treatment	Black in 48 hrs	1,00	240	15
2.	BLT-78-TS	Sunlight for 2 days (16 hrs)	No black colour	4	0	0
3.	BLT-78-TB	Boiling for 15 min	-do-	0	0	0
4.	BLT-78-TU	UV in laminar flow for 1 hr	-do-	43	0	4

* Sampling on 7.12.99

Table 8B: Treatment by boiling (100^{o}C) of stored tank water (BLC-94)

S.No.	Time (min)	MPN for Coliform bacteria		MPN for Faecal streptococcus	H_2S strip test
		Total coliform	Faecal coliform		
1.	0 (no heating)	460	4	43	Black in 24 hrs
2.	1	0	0	0	No black colour
3.	5	0	0	0	"
4.	10	0	0	0	"
5.	15	0	0	0	"
6.	20	0	0	0	"

Table 8C: Treatment by heating stored tank water (BLC-97) at 60°C

S.No.	Time (min)	MPN for Coliform bacteria		MPN for Faecal streptococcus	H_2S strip test
		Total coliform	Faecal coliform		
1.	0 (no heating)	≥ 1,100	4	460	Black in 24 hrs
2.	1	0	0	7	No black colour
3.	5	0	0	4	"
4.	10	0	0	4	"
5.	15	0	0	0	"
6.	20	0	0	0	"

Table 8D: Treatment of sunlight for 16 hrs

S. No.	Sample no.	Treatment of sunlight	Temp. in the bottle	H_2S strip test	MPN for Coliform bacteria		MPN for Faecal streptococcus
					Total coliform	Faecal coliform	
1.	MAA-94* initial	No treatm'nt by sunlight (control)	25°C	Black in 24 hrs	460	4	43
2.	MAA-94-ST	Transparent bottle kept in sunlight	27°C	No black colour	0	0	3
3.	MAA-94-SP	Bottle partially black	29°C	-do-	0	0	0
4.	MAA-94-SB	Bottle totally black	32°C	Black in 24 hrs	7	4	7
5.	MAA-99** initial	No treatm'nt by sunlight	39°C	No black colour	≥ 1,100	≥ 1,100	3
6.	MAA-99-TS	Transparent bottle	41°C	-do-	0	0	0
7.	MAA-99-PS	Bottle partially black	43°C	-do-	0	0	0
8.	MAA-99-BS	Bottle totally black	45°C	-do-	0	0	3

* MAA-94: Sampling on 3.3.2000
** MAA-99: Sampling on 11.4.2000

Table 9: Study of algal growth in rainwater sample under storage and different treatment procedures

S. No.	Sample	Description of sample	Date of sampling	Observation for algae (dates)			
				12.1.00	12.2.00	26.4.00	22.5.00
1.	BLT-78	100% storage tank water	7.12.99	+			
2.	BLT-78-A	(1:1) BLT-78: sterile H_2O	7.12.99	+			
3.	BLT-78-B	(1:3) BLT-78: sterile H_2O	7.12.99	+			
4.	BLT-78-C	(1:9) BLT-78: sterile H_2O	7.12.99	-	+		
5.	BLT-78-D	(1:99) BLT-78: sterile H_2O	7.12.99	-	-	+	
6.	BLT-78-TS	BLT-78 kept in sunlight (2 days)	7.12.99	++			
7.	BLT-78-TB	BLT-78 boiling for 15 min	7.12.99	-	-	-	-
8.	BLT-78-TU	BLT-78 kept in UV of laminar flow (1 hr)	7.12.99	+			
9.	Sterile H_2O	sterile H_2O	7.12.99	-	-	-	-
10.	BLC-94-SB	Kept in sunlight (2 days), totally covered with carbon	3.3.00	-	-	-	-
11.	BLC-94-SP	kept in sunlight (2 days), partially covered black	3.3.00			-	+
12.	BLC-94-ST	Kept in sunlight (2 days) in transparent	3.3.00	-	-	+	

TABLES 10 A to D – CHEMICAL QUALITY OF WATER IN RAINWATER TANKS

Table 10A: Chemical quality of water in storage tank MAP by laboratory method and Development Alternatives (DA) kit

Lab'y methods	PH meter	EC	TDS	Total hardness (ppm)	Ca as $CaCO_3$ (ppm)	Ca^{2+} (ppm)	Mg^{2+} (ppm)	NO_{3-} (ppm)	Cl^- (ppm)
MAP-15	7.6	1.36	101	40	16	6.4	5.8	20	14.2
MAP-22	8.23	0.49	36	20	9.5	3.3	2.5	9	14.2
MAP-32	7.7	0.86	64	44	17.6	7.04	7.6	5	11.3
MAP-37	7.2	0.41	30	16	4.8	1.92	14.1	2.2	8.5
*MAP-23	7.9	0.83	62	36	14.4	5.8	5.2	2	8.5
*MAP-36	8.45	0.54	40	16	6.4	2.6	13.4	2	23
MAP-55	7.4	0.54	40	16	11.2	4.5	11.5	3	14.2
MAP-87A	8.0	0.812	60	184	12.8	5.12	41.6	25	8.51
MAP-88	7.5	0.587	44	176	14.4	5.8	39.3	35.5	8.51
MAP-91	7.5	0.66	49	172	12.8	5.12	38.7	32.5	8.51

DA kit	pH (strip)	Turbidity	TH	Cl^-
MAP-15	7.0	~ 10 NTU	72	28
MAP-22	7.5	~ 10 NTU	44	17
MAP-32	7.0	~ 10 NTU	64	17.7
MAP-37	7.5	Nil	32	21.3
MAP-23	8.0	= 10 NTU	44	21
MAP-36	8.0	Nil	44	17.7
MAP-87	7.5	Nil	48	28.4
MAP-88	8.0	Nil	56	32
MAP-91	7.0	Nil	64	28.4

Table 10B: Chemical quality of water in storage tank MIC by laboratory method and Development Alternatives (DA) kit

Lab'y methods	pH meter	EC	TDS	Total hardness (ppm)	Ca (as $CaCO_3$) (ppm)	Ca^{2+} (ppm)	Mg^{2+} (ppm)	NO_{3-} (ppm)	Cl^- (ppm)
MIC-8	7.7	1.47	109	16	4.8	1.9	2.7	18	22.7
MIC-11	8.05	4.3	320	152	43.2	17.3	26	20	99.3
MIC-18	8.35	2.69	200	56	19.2	7.68	9	39	36.9
MIC-38	7.51	0.93	69	40	6.4	2.6	37.4	5	14.2

DA kit	pH (strip)	Turbidity	TH	Cl^-
MIC-8	7.0	Nil	44	33
MIC-11	7.0	Nil	160	141
MIC-18	7.5	Nil	76	76
MIC-38	7.0	Nil	40	17.7

Table 10C: Chemical quality of water in storage tank BLC by laboratory method and Development Alternatives (DA) kit

Lab'y methods	pH meter	EC	TDS	Total hardness (ppm)	Ca as $CaCO_3$ (ppm)	Ca^{2+} (ppm)	Mg^{2+} (ppm)	NO_{3-} (ppm)	Cl^- (ppm)
BLC-2	8.07	0.97	72	36	14.4	5.8	5.25	10.5	14.2
BLC-3	8.29	1.61	120	56	21	8.3	8.5	35.5	19
BLC-9	7.98	1.75	130	56	22.4	9	8.2	37.5	14.2
BLC-19	6.6	2.53	188	56	16	6.4	10	50	19.8
BLC-28	8.09	1.25	93	36	11.2	4.5	6.1	26.5	11.34
BLC-34	8.34	1.61	120	25	6.4	2.6	4.3	29	8.5
BLC-41	8.23	0.75	55.8	24	14.4	5.8	18.2	9	5.7
BLC-86	7.5	1.77	132	276	48	19.2	55.4	100	12.8
BLC-29	7.81	1.1	82	36	19.2	7.7	4.1	9.5	11.3
BLC-43	7.6	0.61	45	32	17.6	7.0	25	2.0	11.4
BLC-89	7.3	0.7	52	180	16	6.4	39.8	37.5	5.8

DA kit	pH (strip)	Turbidity	TH	Cl^-
BLC-2	7.0	Nil	64	33
BLC-3	6.0-7.0	Nil	88	38
BLC-9	7.0	Nil	96	70
BLC-19	7.0	Nil	64	17
BLC-28	7.5	Nil	58	21.3
BLC-34	7.5	Nil	60	17.7
BLC-41	7.5	Nil	36	17.7
BLC-86	7.0-8.0	Nil	164	32
BLC-29	7.0	Nil	64	21
BLC-43	7.0	Nil	60	17.7
BLC-89	7.0-8.0	Nil	68	30.1

Table 10D: Chemical quality of water in storage tank BLT by laboratory method and Development Alternatives (DA) kit

Lab'y methods	pH meter	EC	TDS	Total hardness (ppm)	Ca as $CaCO_3$ (ppm)	Ca^{2+} (ppm)	Mg^{2+} (ppm)	NO_{3-} (ppm)	Cl^- (ppm)
BLT-1	6.92	0.48	36	8	5.6	2.2	0.6	8	14.2
BLT-4	8.02	0.66	50	28	8	3.2	5	19	11.3
BLT-27	6.72	0.54	40	20	14.4	5.8	1.4	27	8.5
BLT-33	7.68	0.87	65	28	12.8	5.1	3.7	40	5.7
BLT-40	7.34	0.46	34	24	8	3.2	21	20	8.5
BLT-42	6.8	1.46	108	136	38.4	15.4	121	30	8.5
BLT-56	7.1	0.57	42	24	11.2	4.5	19.5	3	14.2

DA kit	pH (strip)	Turbidity (NTU)	TH	Cl^-
BLT-1	7.0	Nil	40	35
BLT-4	7.0	Nil	64	49
BLT-27	7.0	Nil	56	17
BLT-33	7.0	Nil	44	17.7
BLT-40	7.0	Nil	30	17.7
BLT-42	7.0	Nil	192	17.7

Table 12E: Chemical quality of water in storage tank MAA by laboratory method and Development Alternatives (DA) kit

Lab'y methods	**pH meter**	**EC**	**TDS (ppm)**	**Total hardness (ppm)**	**Ca as $CaCO_3$ (ppm)**	**Ca^{2+} (ppm)**	**Mg^{2+} (ppm)**	**NO_{3-} (ppm)**	**Cl^- (ppm)**
MAA-14	7.97	6.42	480	16	4.8	1.9	2.7	7	11.3
MAA-21	7.54	0.88	65	24	11.2	4.5	3.1	1.7	16
MAA-31	8.07	1.12	83	28	12.8	51	3.7	1.2	14.2
MAA-35	8.27	0.86	64	26	6.4	2.6	23	1.5	42.5
MAA-54	7.7	0.72	53	20	9.6	3.8	16.2	3.1	17.01
MAA-90	7.5	0.70	52	172	12.8	5.12	38.7	32.5	8.51
MAA-13	8.6	4.3	320	48	14.4	5.76	8.2	39	31

DA kit	**pH (strip)**	**Turbidity (NTU)**	**TH (ppm)**	**Cl^-**
MAA-14	7.0	10	20	19
MAA-21	7.5	10	44	17
MAA-31	7.5	Nil	40	21.3
MAA-35	7.5	Nil	44	16
MAA-13	7.0	10	84	75
MAA-90	7.5	Nil	56	24.4

APPENDIX I: WATER QUALITY IN DRWH SYSTEMS (data from the literature, by region)

A1 AFRICA

(a) Biological Quality

Country	Author	Total samples	Water quality	Ref.	Remarks
Botswana	J.E. Gould	-		J.E. Gould. Rain water catchment possibilities for Botswana. April, 84, pp 10-12. BTC (Botswana Technology Centre).	• Generally high quality of properly stored rain water is seen. Periodic chlorination is the most economical solution for maintaining water quality.
Botswana	J.E. Gould	13 roof tanks 8 ground catchments		J.E. Gould and H.J. McPherson. Bacteriological quality of rain water in roof and ground catchments system in Botswana. Water International 1987, 12, pp. 135-8.	• Rain water from corrigated iron roofs is of high quality.
Tanzania (Dares Salam)	Mayo & Mahuri	Oct. 1988 & Dec. 1989	86% samples were free of FC. 45% samples tested positive for TC. FS were obtained in 53% of samples.	A.W. Mayo & D.A. Mashauri. Rainwater harvesting for domestic use in Tanzania. A case study. University of Dar es salaam staff houses. Water International 16 (1991), pp 2-8.	
Kenya	Otieno			F.O. Otieno. Quality issues in rainwater collection. Raindrop, June, 94.	Need for maintaining water quality is stressed.
Kenya	Bambrah & Haq	Review of existing literature on rain water quality.	Guidelines for drinking H_2O quality and various physical & chemical treatments have been described.	Dr. G.K. Bambrah and Ms. S. Haq. Quality issues in rain water harvesting in Kenya. Proceedings of 8th International Conference on RWCS, April 25-29, 1997, pp 547-553.	Suitability of using untreated rain water for human consumption is discussed.
Kenya	Bambrah & Haq			Dr. G.K. Bambrah and Ms. S. Haq. Quality issues in rain water harvesting in Kenya. Proceedings of 8th International Conference on RWCS, April 25-29, 1997, pp 547-553.	

(b) Chemical Quality

Tanzania	Mayo & Mahuri	Samples from rain water cisters	pH, total hardness turbidity & colour were analysed	A.W. Mayo & D.A. Mashauri. Rainwater harvesting for domestic use in Tanzania. A case study. Universityof Dar es salaam staff houses. Water International 16 (1991), pp 2-8.	• pH range was 9.3-11.7 & was above standard limits • About 54% of consumers raised objections over taste of water

A2 N. AMERICA

(a) Biological Quality

USA	Roger S. Fujioka and Robert D. Chim	18 cistern systems 36 samples	First flush was not eliminated. FC was recorded in 34 out of 36 samples. FS were present in 16 of the 18 cistern. Chemical quality was also examined and was acceptable.	The microbiological quality of cistern waters in tantallis area of Honolulu Haiwa. Proceedings of the 3rd International conference on 'Rain water cisterm', Jan. 14-16, 1987, Khon Kaen, Thailand	More FS than FC indicating contamination is by birds rather than humans. Bacterial count was reduced in transit from storage tank to the bucket. Turbidity did not always correlate with indicator bacteria.
USA	Roger S. Fujioka, S.G. Insena and Robert D. Chim	15 cistern systems 28 water samples	Significant fraction of faecal coliform recovered from water sample is E. coli. Chemical quality was acceptable.	'The bacterial content of cistern water in Hawai'. Proceedings of Int. conference on rain water system, 1991, Taiwan.	Natural sunlight is an effective disinfectant. Indicator bacteria was not off the roof. Faecal indicator bacteria are present in soil house in to the roofs.
USA	E.W. Faisst and Roger S. Fujioka	Water samples from corrugated metal roof into four 55 gallon plastic cistern samples. 16 samples were analysed over an8 month period.		'Assessment of four rain water catchment designs on cistern water quality'. International Conference, 1994 proceedings of the 6th International Conference on rain water catchment system, Nairobi. Kenya, 1-6 April, 1993	H_2S strip test correlated with FC as compared to TC after a 24 hr incubation. H_2S producing bacteria are more likely to be found in cisterns with sediment build up sand/charcoal/gravel filter reduced bacteria, but still some samples did not meet US standards. The algae film inside cistern was indicated high nutrient level.

A3 S.E. ASIA

(a) Biological Quality

South-East Asia

Thailand	Komol Sivaborrorn	503 were analysed. Out of this 54 were rain water samples.	Bacteriological quality analysis was conducted by different methods	Final technical report submitted to ITRC on development of simple tests for bacteriological quality of drinking water (WQ) control for south east asia. Available on the web.	• H_2S strip test is an excellent test for it is easy, cheap & villagers can see the black colour. • β-galactosud showed good results. The method can be modified to use in the field. • Coliphage test was not so effective. • Out of 54 samples tested positive for coli, faecal coliform, E. coli counts.
Thailand	Appan	709 samples were collected from tiled roofs & gutters, containers located in homes, jars and the point of consumption		A. Appan, Roof water collection systems in some southeast Asian countries: Status and water quality levels. The Journal Royal Society of Health. October, 97, Vol. 117 No. 5, pages 319-323.	• More than 76% of the samples had values exceeding WHO standard • Samples other than those collected from the container showed an FC/ FS ratio of less than one. Possibly 79%-82% of the contamination could have emanated from animal droppings.
Phillipines	Appan	25 ferrocement tanks	24% of total samples had F. coli exceeding WHO guideline limit		
Malaysia	Appan	72 samples from 2 types of roofs in West Malaysia	F. coli values far exceeded the WHO value		
Singapore			• TC & FC values exceed guideline value		•
China	Xijing et al.	Total coliform become fewer in rain water contained in concrete cellais with tile surfaces			
Japan	Kita & Kitamura			Ichiro Kita and Kunihiko Kitamura. Fluctuation of the quality of container stored rainwater during storage. 7th International RWCS, Conference, June 21-25, 1995, Beijing, China, page 27-32.	• Results of coliform group of bacteria were positive throughout the periods.

(b) Chemical Quality

Thailand	Appan		Physico-chemical parameters performed were (pH, colour, turbidity, iron, manganese, lead & cadmium) More than 83% of the samples were satisfactory. However 43% of the samples exceedes the allowable limits of lead	A. Appan, Roof water collection systems in some southeast Asian countries: Status and water quality levels. The Journal Royal Society of Health. October, 97, Vol. 117 No. 5, pages 319-323.	

Indonesia	Appan				• Fish rearing within the tanks was tried.
Malaysia		72 samples were collected from 2 types of roofs in West Malaysia	pH value of rain water samples had a tendency to lie towards lower range of the guideline values		
Khon Kaen		709 samples were collected from tiled roofs & gutters containers located in homes, jars & the point of consumption	Only manganese (2-20% of the samples) and zinc (4-26% of samples) did not meet the guidelines levels		
Singapore		Roof water quality was monitored from a high rise building in Nanyange Tech. University for 6 years from May, 1989.	The values appear to be acceptable in all the physico-chemical parameters except pH which is low. Earlier field investigations have also shown that during January 1974 to July 1983, the range of pH in 11 monitoring stations distributed throughout Singapore was 4.8-5.5.		
China	Bo-ling		The alternative water source (rain water) which had normal fluoride levels was adopted to control fluoride endemics		
Japan	Tachi-Kawa		Metals (Fe, Zn, Pb) concentrations exceeded the standards in a study of rain water collected from roof tops		
Japan	Chiyuda Cets		1st flush of rain water after a 11 day dry spell was analysed. Physico-chemical (Fe, Cd, Cr, Cu, Mn, Ni, Pb, Zn) parameters are tabulated		• Initial rain fall upto 1.5 mm shows that the concentration of pollutants had accumulated on the roof for a 11 day dry spell
Japan	Nariyuki Inkano				• Tokyo International Rain Water Utilization Conference, Sumida city, Summer, 94, page 9-11. • Metals in rainwater in Tachikawa Saiwai-Cho Housing Complex • Quality of first flash of rainwater (Aug.-Sept., 86). • Prevention of mosquito breeding.
Japan	Kitamura		Stored rain water pH were almost constant except for the last 3 weeks duration Turbidity varied during first two weeks Ammonia nitrogen varied, decreasing on storage, which was probably caused by the bacterial activities	Kunihiko Kitamura, Ichiro Kita and Isao Minami. The effects of storage on rain water quality. Proceedings of the 8th International Conference on RWCS, April 25-29, 1997, Tehran, Iran, page 590-595.	• Bird dropping and insect caracasses were found in collected rain water (a sanitary problem)

A4 S & W ASIA

(a) Biological Quality

Srilanka	Heijen & Mansur			Han Heignen and U. Mansoor. Symposium on rainwater harvesting for water security, Feb. 28, 1998. "Rainwater harvesting in the community water supply and sanitation project".	• E. coli count was not high. Only 21% of samples showed 100 colonies/ 100 ml of TC
Srilanka	NWSDB			National Water Supply and Drainage Board, Sri Lanka. Chemical and bacteriological analysis report (15.10.98, 14.12.98 and 16.12.98).	• Bacteriological analysis (TC, EC & FS) was done and contaminantless was found. The board recommended boiling of water before consumption
Mitraniketan (Kerala)				All India Coordinated project report, sponsored by CAPART, Science and Society Division, Mitraniketan, Kerala.	• Bacterial examination was done on a few samples. The stored water had potable status.
Palestine (West Bank)	Sharekh		Level of total coliform contamination were found 27% > 0 coliform	M.S. Abu Sharekh. Rain water roof catchment systems for domestic water supply in south of West Bank. 7th International RWCS, Conference, June 21-25, 1995, Beijing, China, page 65-90.	• Rain water stored in cisterns was used for drinking & domestic purpose

A5 AUSTRALIA

(a) Biological Quality

South Australia	Fuller et al.	Vineyard & orchard areas - 7 cities Industrial areas - 4 cities Residential areas - 2 cities	• Coliform bacteria were present in 12 out of 41 tanks. Upto 500 coliform/ 100 ml were recorded • E. coli was detected in 6 tanks. 15% of 41 tanks levels upto 220 E. coli/100 ml were recorded. • Plate counts in most rain water tanks were in excess of 1,000/ml		C.O. Fuller, R.P. Walters and T.J. Martin. Domestic rainwater tanks working party, March, 81. Quality aspects of water stored in domestic rainwater tanks (a preliminary study).

(b) Chemical Quality

Australia	Ghafouri & Phillips			R.A. Ghafouri and B.C. Philips. Urban storm water re-use opportunities and constraints. Proceedings of 8th Int. Conference on RWCS, April 25-29, 1997, pp 554-565.	• Roof water is seen as one of the possible sources of collecting rain H_2O
South Australia	Fuller et al.	Water stored in domestic rain water tanks from 13 cities. Galvanised iron tanks in the range of 10,000-25,000 lit. were used.	A) Heavy metals Cadmium - one of the tanks reported relatively high cadmium concentration (0.018 mg/l) Lead - concentrations of lead in rain water from tanks were significantly higher (0.061 & 0.072 mg/l). This could be result of dust from surrounding country sides washed from roof tops with each rain fall. Zinc - zince concentrations were found to be in excess (15 mg/l) B) Pesticides were not detected in majority of the samples. C) Suspended solids - concentrations were negligible in all samples D) pH - range was 6.1-9.2 low pH values can accelerate corrosion problems in domestic appliances while high pH is an indication of under viable biological activity in the tank E) TDS - only samples taken from 2 rain water tanks had TDS concentrations in excess of 100 mg/ml (caused by sea spray)		

A6 EUROPE

(a) Biological Quality

Denmark	Per Jacobsen		Concentrations of lead (0.1 mg/l) & zinc (0.1-1.0 mg/l) exceeded the standards for drinking water in Denmak	Per Jacobsen. Metals in Rainwater in Denmark. Tokyo International Rainwater Utilization Conference, Sumida City, Summer, 94, page 9.	
Germany	Wilhelm Meemken		Parameters measured were pH, Fe, Mn, Cu, Pb, Zn, Ca, Mg, Na, K, NH_4^+, SO_4^{2-}, Cl^-, NO_3^- and electrical conductivity	Wilhelm Meemken. Quality examination of rainwater collected from roofs and stored in tanks. Tokyo International Rainwater Utilization Conference, Sumida City, Summer, 94, page 9-10.	• Rain water collected from the roofs could be used for flushing toilet washing cloths and watering plants without special treatment
Switzerland	Yakl Mason, Adrian A. Ammann, Andrea Ulrich & Laurasigg		Concentrations of Cr, Cu, Cd, Zn, Pb, Ca, Mg, Na, K, Cl, NH_4 were determined in rain water roof run off and in infiltrating water at various depths of soil.	Behaviour of heavy metals, nutrients & major components during roof run off infiltrat.	• The concentration of most parameter in roof run off were highest during the first flush at the beginning of rain events.

Milestone Report C4

DRWH DESIGN AND INSECT BREEDING

prepared by

PROF. PADMA VASUDEVAN – Principal Investigator
DR. NAMRATA PATHAK - Project Scientist
MR. PRABHAT CHAUHAN - Project Assistant
and
DR. P.K. MITTAL

Centre for Rural Development & Technology
Indian Institute of Technology, Delhi, India

and

Malaria Research Centre, New Delhi

INCO-DEV Programme ERB IC18 CT98 0276 - Rainwater Harvesting
Sub-programme C: Health Implications

Project sponsored by European Commission

CONTENTS

TEXT

1 Introduction

Task C of this contract, examination of the health implication of widespread use of DRWH, addresses two aspects namely:
i) concerns regarding water quality and possible direct implications due to contaminants
ii) insect breeding related to water storage and health implication arising out of it.

With regard to water quality, two reports have already been submitted. They are Reports C1, *'DRWH-Water Quality: A literature review'*, and C3, *'Water Quality in DRWH Systems'*.

With regard to insect breeding, Report C2, *'DRWH and Insect Vectors:A literature review'* has already been submitted. In it *Anopheles*, *Aedes* and *Culex* mosquito species were identified as the major carriers of diseases and their behaviour was summarised. Dr. Mittal from Malaria Research Institute, India, collaborated in this. The characteristics of the major mosquito species were examined at the different stages of their life. Based on these, a background paper was made for an email conference on "Health Issues related to Water Quality" which was held in October, 2000.

The current report C4 brings out further findings (up to January 2001) on DRWH design parameters for controlling insect breeding. Experimentation continues.

2 Parameters which control insect breeding

The following issues were raised for discussions in the email conference coordinated by I.I.T., Delhi on Water Quality and Health. A number of people from all over the world participated in the email conference responding to the same.

a. The mosquito is the major vector to be considered in the context of DRWH water storage, although entry of lizards, rats and other small animals also need attention.
b. Breeding parameters and behavioural patterns differ for different species of mosquitoes; design for mosquito control must take these into consideration.
c. Whatever the species of mosquitoes, denying access to water is universally effective in controlling breeding. So all openings in DRWH should be closed by suitable meshes, preventing the entry of not only adult mosquitoes, but if possible also that of eggs and larvae which may be washed off from the gutter. It is important not to allow water to stagnate in the gutter. If water in storage gets heated to 50^{o}C, the viability of egg and larvae are reduced.
d. In spite of preventive measures, mosquitoes may enter storage. What would be the appropriate measures to deter ovi position (egg laying) and larval growth? The outcome of the e-conference discussions are shown in appendix 1. It was seen that the web site http://www.ent.iastate.edu/mailinglist/mosquito-1 deals specifically with mosquitoes. The site is being scanned for information useful for DRWH.

Based on the above interactions and on experiments conducted by I.I.T. Delhi, the issues related to control of mosquito breeding in DRWH will be dealt with under the following heads:

- Barriers for preventing the approach of adult mosquitoes to water in DRWH (Sect. 3)
- Quality of water and other parameters which discourage or encourage ovi-position and larval growth (Sect. 4)
- Treatment of water in storage (Sect. 5)

3 Barriers for preventing approach of adult mosquitoes to water in DRWH tanks

Three types of barriers may be envisaged to prevent entry of mosquitoes:

i. Have repellents in the surrounding areas so that mosquitoes are deterred from entering DRWH sites. Burning of leaves, wood smoke and other repellents (e.g. mint, *Vitex negundo* & other essential oils) are deterrents. Further research is needed in this regard.
ii. Have traps with suitable attractant so that the mosquito reaching the site gets attracted and enters the trap (Use of CO_2 trap and other attractants, www.mosquitoes.com/biting/menacing2.htm,www.ent.iastate.edu/department/research/vandyk/hostseek.html). Availability of reagents and cost economics would however be an issue.
iii. Have physical barriers such as screens which will physically prevent entry of mosquito and larvae into the DRWH site.

3.1 Repellants and traps

Studies on the issues (i) & (ii) pertain to general measures for mosquito breeding and control. The I.I.T., Delhi team has some experience in this regard.

A large number of aquatic fauna (e.g. *Spirogyra* sp., *Hydrilla, Ipomea, Eichhornia* and *Pistia*) may support mosquito breeding. On the other hand Covell (1941, malaria control by anti mosquito measures. edn. 2, Thacker Spink and Co., Calcutta) in his book on anti malaria measures has grouped aquatic vegetation preventing mosquito breeding into three types.

- Thick growths on the surface actually preventing breeding, e.g. *Lemna, Azolla, Wolffia, Anacharis, Trapa* etc.
- Those which act as traps, e.g. *Utricularia* i.e. bladderworts, which are well known to entrap and digest insects including mosquito larvae
- Those which are actually poisonous e.g. *Chara*.

3.2 Barriers

However, specifically for DRWH design, issue (iii) on integrating barriers for denying entry to mosquitoes was taken up for study. The following questions were addressed:

(a) Can the adult mosquito be deterred from reaching water surface by depths or torturous paths (U bends)?
(b) What is the role of gutters?
(c) How effective are the screens in preventive entry of the mosquitoes.

Depth and path-related issues Literature review and discussions with experts in Malaria Research Centre indicated that mosquitoes can descend by at least 300 ft (in air) and also go through curvaceous paths.

"There are records of mosquitoes found in deep mines, particularly *Culicine* mosquitoes. They have been found at depths of over 1,000 m in the Kolar gold mines in Karnataka.

The following reports are available on different species:

- *Anopheles annularis* and *A. vagus* have been reported at depths of 300-600 ft.
- *A. culicifacies, A. nigerrimus, A. stephensi* and *A. subpictus* at depths of 300 ft.
- *Culex fatigans* was found breeding at 600 ft."

(Source: *The Anophelines of India*, ICMR, New Delhi, 1981 by T. Ramachandra Rao)

These data indicate that in DRWH, the mosquitoes can easily travel down to water through the down pipes.

Gutters It is well known that mosquito will breed even on 1 mm layer (minimal depth) of water. So if there is stagnation of water, breeding will occur. Thus, gutters form a very important breeding site. Infact, for houses it is even advocated that gutters may be avoided or painted with larvicidal components. (http://www.env.gov.sg/cop/dd/ cop4/hb-7.html).

However, in DRWH it is imperative to have gutters, nor can it be painted as above! Hence, at least the gutter design should allow for smooth flow of water and must be accessible to inspection. Dry leaves should not remain on it and hold water. Inspite of all these precautions if eggs are hatched and water is available they can reach larval stage. Hence it is important to study the effectiveness of screens for preventing entry of not only mosquitoes but also of eggs and larvae.

Screens There is sufficient evidence that screens are useful in preventing the entry of mosquitoes and larvae. However, reports were not available relating to the effectiveness of different mesh sizes with respect to not only adult mosquitoes but also larvae and eggs. The size of adult, larvae and eggs are reported in C2. In *Culex*, where egg come in rafts, the overall size is higher although the individual eggs are small. Besides the size of eggs, the stretchability of mesh and speed with which water flows pushing the screen, are factors which can affect the entry of larvae and eggs.

Under the project, experiments were undertaken on the efficiency of screens differing in mesh size, in preventing the entry of adults. Results of laboratory level experiments with different screens with varying hole sizes are shown in table 1. The screens, made of nylon or cloth net, were bought from the local market. Actual number of holes per inch square on each of these was determined by counting. It was noted that all the mesh sizes used prevented the entry of adult mosquitoes. Since adults are of size 0.5 cm or 0.2 inches in length and the hole size on the screens are in the range of 0.01 - 0.05 in square, the screens were effective in filtering out the adults.

Experiments on passage of mosquito larvae and eggs were also conducted and the results are shown in table 1. It is seen that I to III instar larvae of all the mosquitoes passed easily through the screens. Only the late third instar as well as fourth instar larvae were filtered out. This is because, by the time they moult into fourth instar, they are comparable or even bigger than the adult in size. Only muslin cloth with very small hole size (more than 500 holes per square inch) was able to prevent entry of larvae at all its stages of growth. As for the eggs, the eggs of *Aedes* and *Anopheles* pass easily through all meshes. Even muslin cloth does not prevent passage of *Anopheline* eggs. On the other hand *Culex* lays eggs in rafts which are bigger in size. The entry of eggs of this species is prevented by all meshes.

Thus, while all the screens used were preventing entry of adult mosquitoes, there is enough chance of early instar larvae of various species being washed off into the storage tanks. Only muslin cloth is effective in this regard. However, it is to be seen whether muslin cloth when placed on the inlet, allows for the passage of water at a flow rate of 50 l per min. Designs have to be evaluated, in terms of the roof area, area of the inlet, as well as rate of rain fall at its peak. Also debris will collect with time on the screen and it has to be washed periodically. However, use of the cloth may be better than wire mesh made of iron or other metals. It may be used at the mouth of first flush device or at the junction between first flush device and storage. It could filter out mosquito, egg and larvae as well as dust particles. Further work may be done in optimisation.

4 Quality of water and other parameters which discourage or encourage ovi-position and larval growth

The effects of the following were studied:

a) Type of container
b) Quality of water in terms of oxygen availability and nutrient status

4.1 Effect of type of container

Common types of water container used in DRWH are made of ferrocement, plastic and clay. In a set of experiments, tap water (A), rainwater harvested from ferrocement tanks (B) and from syntex tanks (C) were withdrawn into clay pots and plastic tubs. To investigate the effect of both water type and cover type upon mosquito egg-laying, a set each of the containers was (i) left open, (ii) covered with clay lid, (iii) covered with iron mesh or (iv) covered by a plastic sheet. The results are presented in table 2. It was seen that in all cases when the containers were left open, mosquitoes entered and laid eggs. Wherever, there was a complete closure e.g. by a clay lid, which did not sag, there were no eggs/larvae. In the case of plastic cover held tight there was no ovi-position. However, where plastic cover was loosely held, small amounts of water accumulated where the cover sagged. This was sufficient for mosquito to lay eggs. An iron mesh under these circumstances did not encourage ovi-position. So the above experiments reiterate that water availability is the most important factor. Nature and shape of material used have no bearing on ovi position.

4.2 Effect of quality of water in terms of oxygen availability, nutrient status and temperature

Mittal et al. (Annual report, 1982, Malaria Research Centre, ICMR) have studied extensively the egg-laying of mosquitoes in water samples with different levels of salts and dissolved oxygen. The relevant data are reproduced from their report (Table 3). It is clear from this that the average dissolved oxygen requirement for *Culex* is 2.1 and for *Aedes* and *Anophele* is 6.2 and 6.6 ppm respectively. It has also been quoted by others that mosquito larvae can tolerate 4 ppm dissolved oxygen or less (www.mp.usbr.gov/geospat/olympiad/olyimage/larvae.html). Generally, rain water has sufficient dissolved oxygen at the time of harvest as well as storage. Hence oxygen levels are not likely to deter ovi-position in rain water. As for nutrients, for ovi-position high levels may not be needed initially.

However nutrient levels are a significant issue for larval growth. It is in fact known that *Anopheles* and *Aedes* prefer clean water while *Culex* can breed in water of higher BOD. To further study these issues, experiments were conducted on ovi-position and larval growth using rainwater samples as well as tap water and double distilled water for comparisons (Tables 4 & 5). Ovi-position in rain water and tap water was of a similar order whereas in freshly double-distilled water ovi-position was less. It was also seen that tap water, rainwater and double-distilled water supported larval growth up to third instar. In the case of double-distilled water and tap water there were no fourth instar larvae and in the rain water a few larvae could develop into fourth instar. However, in none of these cases there was emergence of adults, indicating that nutrient availability could be the limiting factor. When nutrient was added in the form of yeast and dog biscuits to tap water, out of 25 larvae, a good number (i.e. 14) emerged as adults whereas in the control (tap water) no larvae developed beyond third instar. In another set of experiments, rainwater and groundwater samples were compared. It was again seen that rainwater supported ovi-position (Table 6 & 7) and larval growth up to second instar, but larvae did not grow to adults.

Thus, although rain water supports ovi-position and larval growth, nutrient availability may be the limiting factor for adult emergence. Wherever a roof is under overhanging trees and gutter holds leaves and other materials which can provide nutrients, larval growth in rain water could be high enough for adult emergence. Also the growth of algae in storage may provide nutrients for mosquito breeding.

4.3 Effect of temperature on larval growth

Water tanks may get heated up by solar radiation, so we should examine the effect of temperature upon larval development. The speed of larval growth is accelerated in warm water and lessened in cold water. Warmer temperatures also stimulate growth of aquatic plankton and provide more food to the larvae than cold water. However, the larvae can not survive extremely high temperature. The upper limits of temperature at which no larvae can survive is called "thermal death point" (TDP). For example, for *Anopheles minimus,* the TDP is 41^{o}C and for *Anopheles vagus*, it is 44^{o}C. At 52^{o}C all larvae die immediately. This is the limit of biological tolerance of high temperature (*The Anophelines of India*, ICMR, New Delhi, 1981 by T. Ramachandra Rao).

5 Treatment of water in storage

WHO guidelines on use of chemicals for preventing mosquito breeding are presented in table 8 & 9. However, most of these are not suitable for use in DRWH, i.e. with respect to potable water. Only Abate (temephos) has been recommended by WHO as a larvicide for potable water also and the dose for application is given in the table. This requires further discussions.

It has been shown that kerosene, in small amounts sufficient to cover entire surface area of water kills larvae, by blocking oxygen. (www.nt.gov.au/nths/publich/entomology/programs/dissure/dengueph.html). Some experiments were done at the laboratory level at I.I.T., Delhi on the effect of kerosene oil addition. The results are presented in table 10. While kerosene is effective as larvicide, the kerosene oil locally available in developing countries may not be suitable for adding to drinking water.

6 Conclusions

- Stored water in DRWH can be a potential breeding site for *Aedes* and *Anopheles*. The amount of oxygen, light as well as nutrients in rain water are sufficient for ovi position and larval growth. However nutrient availability may be the limiting factor for adult emergence. In experiments with rainwater from roofs *no* larvae achieved adulthood and it seems that only where organic content is significant (or perhaps where light levels permit algal growth) will successful breeding occur.
- First-flush devices and the use of a screen between the first-flush system and storage could be tried as a barrier for preventing entry of adult mosquitoes. However the finest practical (peak-flow-permitting) screens are too coarse to hold back the eggs of *Aedes* and *Anopheles* or the first instar larvae of any mosquito species. To do that Nylon mesh or muslin cloth with 500 holes or more per square inch would be needed.
- There are several surface or whole-fluid agents that can be (and are) used to safely kill mosquito larvae or prevent ovi-position in water tanks. However none is simple to apply automatically nor, once applied, will last for many months.

7 Plan for further action

Some of the experiments initiated will be continued. The findings will be discussed in the dissemination workshop scheduled from April 18th-20th, 2001. Based on these discussions, additional experiments may be undertaken up to June, 2001. The literature review will also be continued. Information already presented in C1, C2, C3 and C4 will then be integrated with additional data obtained, in the final report to be submitted in July 2001.

TABLES

Table 1: Passage of mosquito eggs and larvae through different nylon/cloth screens

Mesh sample label	**Mesh size (No. of holes/inch^2**		***Culex quinfasciatus* (raft)**	***Aedes ageypti***	***Anopheles stephensi***
A	**Muslin mesh cloth (more than 500)**	Egg	x	x	√
		I	x	x	10% can pass
		II	x	x	x
		III	x	x	x
		IV	x	x	x
B	**364**	Egg	x	√	√
		I	√	√	√
		II	√	√	√
		III early Late	√ x	-	√ x
		IV	x	-	x
C	**360**	Egg	x	√	√
		I	√	√	√
		II	√	√	√
		III early Late	√ x	-	√ x
		IV	x	-	x
D	**351**	Egg	x	√	√
		I	√	√	√
		II	√	√	√
		III early Late	√ x	-	√ x
		IV	x	-	x
E	**324**	Egg	x	√	√
		I	√	√	√
		II	√	√	√
		III early Late	√ x	-	√ x
		IV	x	-	x
F	**136**	Egg	x	√	√
		I	√	√	√
		II	√	√	√
		III early Late	√ √	-	√ x
		IV	√	75% passed	x
G	**130**	Egg	x	√	√
		I	√	√	√
		II	√	√	√
		III early Late	√ √	-	√ x
		IV	√	30% passed	x

Note: √ is passes through and x does not pass

Table 2: Ovi-position of the mosquitoes in rain water in different containers (clay and plastic) and use of screens (clay cover, iron mesh and plastic sheet)

Container type	Open/covered	Presence/absence of larvae
A. Laboratory tap water		
Clay pot	Open	√
	Clay lid	x
	Plastic (tightly covered)	x
Plastic	Open	√
	Iron mesh	x
	Plastic cover (sagging)	√ (on top of cover, not inside tub)
B. Ferrocement tank 1 (rain water) tinned roof		
Clay pot	Open	√
	Clay lid	x
	Plastic (tightly covered)	x
Plastic	Open	√
	Iron mesh	x
	Plastic cover (sagging)	√ (on top of cover, not inside tub)
Ferrocement tank 2 (rain water) cement roof		
Clay pot	Open	√
	Clay lid	x
	Plastic (tightly covered)	x
Plastic	Open	√
	Iron mesh	x
	Plastic cover (sagging)	√ (on top of cover, not inside tub)
C. Syntex tank 1 (rain water) Plastic roof		
Clay pot	Open	√
	Clay lid	x
	Plastic (tightly covered)	x
Plastic	Open	√
	Iron mesh	x
	Plastic cover	√ (on top of cover, not inside tub)
Syntex tank 2 (rain water) Asbestos roof		
Clay pot	Open	√
	Clay lid	x
	Plastic (tightly covered)	x
Plastic	Open	√
	Iron mesh	x
	Plastic cover (sagging)	√ (on top of cover, not inside tub)

Note: √ is larvae present, x is no larvae

Table 3: Chemical analysis of waters supporting breeding of different species of mosquitoes

Mosquito		pH	Hardness	Sodium	Chloride	Potassium	Bromide	Nitrates	Nitrites	Free ammonia	Dissolved oxygen
Anopheles	Range	7.97-9.525	73.5-1030	60-2500	25-1760	7.8-175	0.27-8.22	3.8-13.3	1.4-43.8	1.2-6.6	4-10.6
	Average	(8.6)	(295.4)	(766)	(606.8)	(63.9)	(3.13)	(8.2)	(8.52)	(2.81)	(6.66)
Culex	Range	7.63-8.7	115.256	28-490	23.5-545	10-109	0.26-5.03	4.2-8.2	1.57-8.2	0.7-21.5	0.6-5.1
	Average	(8.1)	(188)	(256)	(236)	(35)	(1.79)	(5.4)	(3.8)	(6.99)	(2.1)
Aedes	Range	8.1-8.5	82.5-130	17-150	22.5-52	4.25-5.3	0.15-0.8	3.9-7.3	0.7-3.44	0.22-0.7	4.7-7.4
	Average	(8.26)	(102)	(52)	(36.3)	(4.75)	(0.4)	(5.3)	(1.94)	(0.45)	(6.2)
Mixed (*Anopheles* + *Culex*	Range	7.9-10.8	61.3-355	45-910	65.7-961	7.3-67.8	0.76-4.73	3.01-10	0.7-27.50	1.1-16.8	1.0-10.2
	Average	(8.9)	(177)	(446)	(446.5)	(23.0)	(2.36)	(6.55)	(5.7)	(4.301)	(5.25)

Table 4: Effect of water quality on ovi-position

Water type	***Anopheles culicifacies***	***Aedes agepyti***
Rain water	+++++	+++++
Tap water	++++	+++++
Double distilled water	+	++

Note: Female mosquitoes were blood fed and were placed inside a mosquito cage containing different water samples for ovi position (16.2.2001)

Table 5: Effect of water quality on growth of *Anopheles stephensi* first instar larvae

Date	**Double distilled water**						**Tap water**						**Tap water + food (yeast powder + dog biscuits)**						**Rain water**					
	I	**II**	**III**	**IV**	**P**	**A**	**I**	**II**	**III**	**IV**	**P**	**A**	**I**	**II**	**III**	**IV**	**P**	**A**	**I**	**II**	**III**	**IV**	**P**	**A**
07.02.2001	23	2					20	5					22	3					20	5				
08.02.2001	13	11																						
09.02.2001	5	18					7	16					2	21	2				9	16				
10.02.2001							9	9	5															
12.02.2001		20						18	5					2	2	21				13	8			
13.02.2001		10	8					17	6					1	2	21	2			8	13			
15.02.2001		1	17					3	4						2	17	5	2		4	13			
16.02.2001		1	17					3	4						2	10	8	4		2	11			
19.02.2001		3	3					3	3						1	8	2	13	2	8				
20.02.2001		1	5					3	3						1	7	1	14	1	9				
22.02.2001		5						1	3							7				2	5	2		

I - IV: instar larvae P: Pupae A: Adults

Note: 25 first instar larvae were placed in 200 ml water samples in a plastic bowl and the larval growth was observed as a function of time. Experiment was initiated on 5.2.2001

Table 6: Effect of water quality on ovi-position

Water type	*Anopheles culicifacies*
Rain water	+++
Water pumped with motor (Micromodel)	+++
Hand pumped water (Micromodel)	++

Note: Female mosquitoes were blood fed and were placed inside a mosquito cage containing different water samples for ovi position (16.2.2001)

Table 7: Effect of water quality on growth of *Anopheles stephensi* first instar larvae

Date	Rain water		Tap water		Water pumped with motor		Hand pumped water	
	I	II	I	II	I	II	I	II
19.02.2001	23	1	17	-	25	-	23	-
20.02.2001	3	20	16	-	20	-	-	-
22.02.2001	1	21	10	3	11	1	-	-
23.02.2001	1	21	7	6	10	1	-	-

Note: 25 first instar larvae were placed in 200 ml water samples in a plastic bowl and the larval growth was observed as a function of time. Experiment was initiated on 16.2.2001

Table 8: Insecticides suitable for interior treatment against mosquito vectors

Insecticide	Chemical type[a]	Dosage of a.i.[b] (g/m^2)	Duration of effective action (months)	Insecticide action	Toxicity: [c]oral LD_{50} of a.i. for rats (mg/kg of body weight)
Alphacypermethrin	PY	0.02 - 0.03	4 - 6	Contact	79
Bendiocarb	C	0.1 - 0.4	2 - 6	Contact & airborne	55
Carbusulfan	C	1 - 2	2 - 3	Contact & airborne	250
Chlorpyrifosmethyl	OP	0.33 - 1	2 - 3	Contact	> 3,000
Cyfluthrin	PY	0.02 - 0.05	3 - 6	Contact	250
Cypermethrin	PY	0.5	4 or more	Contact	250
DDT	OC	1 - 2	6 or more	Contact	113
Deltamethrin	PY	0.01 - 0.025	2 - 3	Contact	135
Etofenprox	PY	0.1 - 0.3	3 -6 or more	Contact	> 10,000
Fenitrothion	OP	2	3 - 6	Contact & airborne	503
Lambdacyhalothrin	PY	0.02 - 0.03	3 - 6	Contact	56
Malathion	OP	2	2 - 3	Contact	2,100
Permethrin	PY	0.5	2 - 3	Contact	500
Pirimiphosmethyl	OP	1 - 2	2 - 3 or more	Contact & airborne	2,018
Propoxur	C	1 - 2	3 - 6	Contact & airborne	95

[a] C = carbamate; OC = organochlorine; OP = organophosphate; PY = synthetic pyrethroid

[b] a.i. = active ingredient [c] Toxicity and hazard are not necessarily equivalent

Source: Chemical methods for the control of vectors and pests and public health importance, edited by D.C. Chavasse and H.H. Yap, WHO, Division of Control of Tropical Diseases, WHO Pesticide Evaluation Scheme (WHO/CTD/WHOPES/97.2), pp. 25.

Table 9: Insecticides suitable as larvicides for mosquito control[a]

Insecticide	Chemical type[a]	Dosage of a.i.[b] (g/m^2)	Formulation[d]	Duration of effective action (weeks)	Toxicity: [c]oral LD_{50} of a.i. for rats (mg/kg of body weight)
B. thurigiensis H-14	MI	[f]	AQ,GR	1 - 2	> 30,000
B. sphaericus	MI	F	GR	1 - 2	> 5,000
Chlorpyrifos	OP	11 - 25	EC, GR, WP	3 - 17	135
Chlorpyrifosmethyl	OP	30 - 100	EC, WP	2 - 12	> 3,000
Deltamethrin	PY	2.5 - 10[g]	EC	1 - 3	135
Diflubenzuron	IGR	25 - 100	GR	2 - 6	> 4,640
Etofenprox	PY	20 - 50	EC, oil	5 - 10	> 10,000
Fenitrothion	OP	100 - 1,000	EC, GR	1 - 3	503
Fenthion	OP	22 - 112	EC, GR	2 - 4	586
Fuel oil	-	[h]	Soln	1 - 2	Negligible
Malathion	OP	224 - 1,000	EC, GR	1 - 2	2,100
Methoprene	IGR	100 - 1,000	Slow release suspension	2 - 6	34,600
Permethrin	PY	5 - 10	EC	5 - 10	500
Phoxim	OP	100	EC	1 - 6	1,975
Pirimphosmethyl	OP	50 - 500	EC	1 - 11	2,018
Pyriproxyfen	IGR	5 - 10	EC, GR	4 - 12	> 5,000
Temephos	OP	56 - 112	EC, GR	2 - 4	8,600
Triflumuron	IGR	40 - 120	EC, WP	2 - 12	> 5,000

[a] Pyrethroids are not normally recommended for use as larvicides because they have a broad spectrum impact on non-target arthropods and their high potency may readily potentiate larval selection for pyrethroid resistance
[b] IGR = insect growth regulator, MI = microbial insecticide; OP = organophosphate; PY = synthetic pyrethroid
[c] a.i. = active ingredient
[d] AQ = aqueous; EC = emulsifiable concentrate; GR = granules; soln = solution; WP = wettable powder
[e] Toxicity and hazard are not necessarily equivalent
[f] Dosage according to the formulation used
[g] The lowest levels are recommended for fish bearing waters
[h] Apply at 142-190 l/ha or 19-47 l/ha if a spreading agent is added

Source: Chemical methods for the control of vectors and pests and public health importance, edited by D.C. Chavasse and H.H. Yap, WHO, Division of Control of Tropical Diseases, WHO Pesticide Evaluation Scheme (WHO/CTD/WHOPES/97.2), pp. 27.

Table 10: Effect of kerosene oil on the growth of *Aedes agepyti* IIIrd instar lavae

Dated	Parameters	Rain water	Distilled water
10.09.99	Close lid	All alive (100%)	1 dead (90%)
	Kerosene oil (2 drops)	All dead	All dead
13.09.99	Close lid	5 alive (50%)	3 alive (30%)
	Kerosene oil (2 drops)	All dead	All dead
15.09.99	Close lid	3 alive (30%)	All dead (0%)
	Kerosene oil (2 drops)	All dead	All dead
23.09.99	Close lid	All dead (0%)	All dead (0%)
	Kerosene oil (2 drops)	All dead	All dead

Closed lid: means control water in tightly closed BOD bottles

Note: 10 larvae were placed in 150 ml of rain water and distilled water in BOD bottles and the effect was observed

APPENDIX

Summary of email conference discussions prepared by Padma Vasudevan, I.I.T., Delhi

Mosquito Breeding

It must be noted that firstly availability of mosquito in the vicinity is an important pre-requisite for mosquito breeding. Also only mosquitoes which are carriers vectors of various diseases are a health hazard; otherwise mosquito bite would only be an irritant and not a major risk. Flight range of different species of mosquito (refer report C-2) is of the order of 50 m to 3 km.

John Gould and others have referred to reports available on the presence of mosquito, in DRWH, prevention and control of its breeding (see references). In the e-conference the Indian/ Srilankan partners reiterated that mosquito breeding is a potential threat to health issues if not already one. Arya Bandhu observed that mosquitoes enter the tank even when they are reasonably secure. He invited factual evidence of egg and larvae getting washed off and entering the tanks through filter systems. Innovative methods are needed to free the water tanks from mosquito especially for poor households who do not have proper guttering and sufficient labour to attend to first flush systems. Based on discussions with Malaria Research Centre, India (see report C-2 under the EU project), it is noted that eggs and larvae could be small enough to pass through screens. Further research is in progress in examining whether they could be retained by nets of appropriate mesh size and filters. Terry Thomas has summarised the 5 strategies to reduce the mosquito breeding and control.

1. <u>Sealing</u>

   Terry Thomas felt that it is not easy to properly seal household tanks and maintain the seal. However, Brian Skinner felt that tight cover can be produced for ferrocement tanks. Paper or polythene could be used for this. He suggested that initially casting a slight upstand on the roof around the edge of the access hole, to give a raised flat rim (say 50 mm wide), and then casting a flat overlapping cover (e.g. thin concrete or ferrocement slab) on this upstand will produce a good tight closing access cover. One could of course also use mud or very weak cement mortar under the cover to play safe. Perhaps other alternative could be a gap filler made from a continuous piece of string laid on the upstand.

According to Arya Bandhu old cycle tube seals have provided good protection against mosquito entry. It basically seals the tank to cover interface. These are individual household practices and thus are not widely practiced. In his opinion a light weight galvanised cover over the entire mouth with proper fit on the upstand part would be most appropriate.

2. Screening

Unless the inlet of a storage tank has a mosquito screen, obviously they can enter and leave via the inlet pipe.Having a proper mosquito screen which will not permit entry of mosquito seems to be a very important step. Use of suitable mosquito nets of appropriate mesh size needs to be considered.

An inclined mosquito screen, set into a small chamber (200 mm x 200 mm in plan) position on the roof of the tank directly under the discharge from the down pipe and over hole over the roof would be useful. One side of the chamber, on the low side of the mesh, should be opened so that the debris screened out of the water are washed off the mesh. There should also be a split cover over the chamber, around the pipe to prevent the light entering the pipe through the mesh covered hole into the roof.

3. Surface Barrier

Use of kerosene and plastic balls have been mentioned as surface barrier. Infact Malaria Research Centre, India (see C-2 report) have also recommended the use of expanded polystyrene beads (EPS). Brian Skinner has described an idea successfully used to control mosquitoes in septic tanks and wet pit latrines by floating a 20 mm layer of particles of polystyrene on the surface.

According to Allan D. Weatherall, in Australia, mosquito larvae are being eradicated from domestic water storage tank by using couple of drops of kerosene which forms a thin film and floats on the water surface. However these methods may not be entirely be satisfactorily for DRWH tanks meant for storing drinking water.

4. Use of chemicals

Vince Whitehead sent useful information which can be found at: http://www.ent.iastate.edu/mailinglist/mosquito-1/. The WHO website http://www.who.int/ctd/whopes/progress.htm showed under the heading of the first table 'products under evaluation by WHOPES (Sept., 2000)' that Methoprene or temephos are used as a mosquito larvicide. One comment from Argentina mentioned use of ABATE as a larvicide not toxic at the concentration used for water tanks. During a visit to household in Cambodia, Vince Whitehead came across 1 cu. m tank that having about 6 small packets laid, equally spaced out on the bottom of the tank. The house owner explained to him that these were to kill mosquito larvae. The contents of the packets presumably dissolved into the tank over a period of time. Indian participants suggested the use of herbs. <u>Indepth research on toxicity is needed before suggesting chemicals as larvicides in drinking water</u>.

5. <u>Biological Control</u>

   Dragon fly/fish and other methods of biological control including use of *Bascillus thurigenesis* Bti had been summarised by Padma Vasudevan in report C-2, page 14. Brian Skinner pointed out that if it is not feasible to totally screened the tank from mosquito then they can be controlled in the tank by using dragon fly larvae which consumes mosquito larvae. However, the consumers may not relish the idea of such creatures in the tanks and presumably they need a continual supply of dragon fly larvae for control.

   Shiela Carmen has described use of fish in ferrocement water storage tanks in the St. Vincent Grenadines. The water from these tanks had been used for drinking and cooking and users are of the opinion that this method reports no ill effects, though the quality of water has not been tested.

   A participant indicated that WHO has recommended use of BTi in the storage tanks for potable water. This is a very promising area.

Dynamically-compacted cement stabilised soil blocks for low-cost walling

By

David Edward Montgomery

Submitted for the degree of

Doctor of Philosophy in Engineering

University of Warwick, School of Engineering

January 2002

Dedication

Sometimes at the beginning of a publication one finds a dedication to a certain person or member of the family who has been an influence in the author's life either in general or specifically in generating the work in question. There is one person in my life that immediately springs to mind that is worthy of such a dedication. Furthermore, my experience with this person is not unique as millions of others have found him to be a great inspiration, comfort, guide and friend. "What's his name?" you may be asking yourself and, "Why haven't I heard of this incredibly influential person?" You most probably have but you have never accepted him as such, or welcomed him into your heart and life. Well, now you have an opportunity to do just that. Please read on.

For years the name Jesus was just an everyday swearword to me. The historical individual did not mean anything to me and religion seemed hypocritical, oppressive and irrelevant to modern life. However, during my late teens I was given opportunities to live life to the full and experience many different things. Yet I still seemed unsatisfied and kept searching for something else. I was invited to a Christian gathering at university where I heard about the love of God and Jesus being God's only Son sent into the world to die for the sins of the world. I was told about an individual who had the power to forgive sins and transform lives. He also wanted to forgive my sins and change me into a child of God. That night I welcomed Jesus Christ into my heart and life and accepted Him as my saviour. Jesus suddenly became a real living person in my life and through the Bible He has helped and guided me through life. Trusting in Him was the best thing I ever did, and I cannot recommend Him more highly to anyone. Man has gone a long way away from God, but He still loves us and commands us to return to Him for forgiveness, reconciliation with Himself and rich blessings in this life and throughout eternity. The Lord Jesus Christ is still searching for people willing to trust Him in simple faith, will you be one of those people today? Please ponder the verses below and thank you for taking the time to read this.

David E. Montgomery

"*For God so loved the world, that he gave his only begotten Son, that whosoever believes in him should not perish, but have everlasting life.*" John 3:16.
"*For the Son of man is come to seek and to save that which was lost.*" Luke 19:10
"*And the times of this ignorance God winked at; but now commands all men every where to repent.*" Acts 17:30
"*For whosoever shall call upon the name of the Lord shall be saved.*" Romans 10:13
"*For by grace are ye saved through faith; and that not of yourselves: it is the gift of God: not of works, lest any man should boast.*" Ephesians 2:8,9.
"*Jesus saith unto him, I am the way, the truth, and the life: no man cometh unto the Father, but by me.*" John 14:6.

Table of Contents

List of Figures

List of Tables

Acknowledgements

My sincerest and deepest thanks goes to my supervisor Dr. Terry Thomas, without whose unending support and guidance I would not have completed this work.

The academic and technical staff at the University of Warwick have been a great help in this research and their skills and commitment have helped to make the project forthcoming and successful.

I would also wish to thank my late mother, Elizabeth, who has been an inspiration and a challenge particularly with respect to her writing skills that were executed with a high degree of professionalism and creativity. Thanks also goes to my wife Rachel for her patience and longsuffering throughout the progress of the PhD.

Declaration

I, David E. Montgomery, hereby declare that this thesis reports previously unpublished work that I have personally done during the three-year PhD programme at the University of Warwick.

Summary

This document contains the detailed results and conclusions of work carried out during a PhD to investigate the process, production and performance of dynamically compacted cement-stabilised soil blocks suitable for sustainable low-cost building. An earlier project carried out by the author demonstrated that full-size blocks could be manufactured by dynamic compaction. It was hoped that this technique could be applied to the self-evident need for low-cost housing in the humid tropics. The apparent advantages of this process, over quasi-static compression (slow steady squeezing), have lead to further investigation into the critical factors influencing the production of such building units.

Initial tests on small cylindrical samples produced by both quasi-static compression and dynamic compaction provided a means of comparison and helped to develop relationships between dominant variables. These tests showed that the moisture content of the compact was a critical variable, influencing its consolidation and its final cured strength. Optimisation studies were undertaken to determine acceptable parameters for impactor mass, drop height and number of applied blows. These chosen parameters were then extrapolated to full-size block production with the necessary adjustments for practicality and cost. Full-size block production using the Test Rig indicated similar relationships as those discovered at the smaller scale, including the more effective consolidation offered by dynamic compaction. From this experience a production prototype was designed and disseminated to a collaborator in India for further trials and feasibility studies. These trials demonstrated that dynamic compaction could produce blocks with a 7-day wet compressive strength of between 3-5MPa with only 5% cement. Feasibility studies there indicate dynamic compaction offers potential savings of 40% compared with local high-tech CSSB manufacture.

The dynamic compaction mechanism was more closely analysed to determine the forces delivered during the impact blow. These were found to be fraction (30kN) of the forced delivered by an equivalent hydraulic press (400kN). This results in less complex and less expensive machine manufacture that is amenable to local manufacture and maintenance. Furthermore, dynamic compaction presents an economically viable and sustainable alternative to other methods of block manufacture.

Abbreviations

BS: British Standard
CEB: Compressed Earth Block
C. of V.: Coefficient of Variation (estimate of population unless otherwise stated)
CSSB: Cement Stabilised Soil Block
M.C.: Moisture Content
O.M.C.: Optimum Moisture Content
P.D.D.: Projected Dry Density
Pop'n: Population
S.T.P.: Standard Temperature and Pressure
S.D.: Standard Deviation (estimate of population unless otherwise stated)
W.C.S.: Wet Compressive Strength

Glossary of terms used

Aggregate: Pieces of crushed stone, gravel, etc. used in making concrete.
Block: A larger type of brick not necessarily made of fired clay, but stabilised in some way, sometimes with central cores removed to reduce the weight.
Brick: An object (usually of fired clay) used in construction, usually of rectangular shape, whose largest dimension does not exceed 300mm.
Bulk Density: Density calculated including any moisture present in the material.
Cement: Ordinary Portland Cement (OPC).
Clay: The finest of the particles found in soil, usually of less than 0.002mm in size and possesses significant cohesive properties.
Concrete: The finished form of a mixture of cement, sand, aggregate and water.
Dynamic Compaction: A process that densifies soil by applying a series of impact blows to it.
Fines: General category of silts and clays.
Frog: A tapered addition to a block mould to create a void in the finished block.
Gravel: A mixture of rock particles ranging from 2mm to 60 mm in diameter.
Green: Describing the state of material containing cement and water before it reaches the critical time, after which further plastic deformation hinders the final set strength.
Green Density: The density calculated immediately after ejection prior to any curing, drying or soaking.
Green Strength: The strength of a material immediately after forming and before any drying or curing has taken place.
Impactor: Solid object of known mass that is repeatedly dropped onto the surface of the soil within a mould.
Mortar: The sand/cement mix used to join block courses.
Projected Dry Density: The calculated density at ejection assuming no moisture is present in the formed sample, only solid matter.
Permeability: Describing a material that permits a liquid or gaseous substance to travel through the material.

Porosity: A measure of the void volume as a percentage of the total material volume.
Render: The sand/cement mix used to cover and protect walling.
Sand: A mixture of rock particles ranging from 0.06mm to 2 mm in diameter.
Silt: Moderately fine particles of rock from 0.002mm to 0.06mm in size.
Soil: Material found on the surface of the earth not bigger than 20mm in size, not including rocks and boulders and predominantly non-organic. If soil is to be used for building material it must not contain any organic material and it can be a natural selection of particles or a mixture of different soils to attain a more suitable particle distribution.
Stabilised soil: Soil which has been stabilised (treated to improve structural characteristics) by using one or more of the following stabilisation techniques: mechanical, chemical and physical.

1 Introduction

This is a short chapter that briefly outlines the motivation for this work, and explains why research in this area is of interest to us. This is done by broadly outlining the problem of housing shortage specifically in developing countries. The final section outlines the structure of this thesis and informs the reader of certain conventions used throughout.

1.1 Justification for this work

There is a self-evident need for adequate and durable housing, especially in the urban and peri-urban areas of developing countries. The poor are most adversely affected by this housing shortage. Assuming land availability and planning permission for further development, the need is to deliver more durable housing at lower cost.

The cost of a dwelling can be split into a number of separate areas as follows:

1. Initial land survey
2. Land preparation on paper – division into plots with access, (needs approval)
3. Physical preparation of ground – clearing vegetation, debris, boulders, etc.
4. Installation of services (optional) – water, sewerage, electricity and telephone
5. Purchase of the plot – cost direct to the homebuilder
6. House erection – foundations and walling (entailing materials and labour)
7. Roofing – spanning beams and roof material
8. Openings – windows and doors with fittings

9. Services – connection up to services if available, (optional, may require approval)

Items 6 to 8 constitute the most significant part of the total cost of the dwelling. Furthermore, the walling constitutes the most significant part of the physical structure, 60% according to (Agevi, 1999). From this it makes sense to concentrate work on low-cost walling. Research recently conducted at Warwick University has indicated that dynamic compaction may provide a method of improving the performance of stabilised soil blocks for walling and at reduced cost.

A further motivation for research into stabilised soil blocks is their environmental sustainability. Cement Stabilised Soil Blocks (CSSB) use low quantities of cement, locally available soil and have a low energy requirement. Currently popular alternatives such as clamp fired brick and concrete blocks do not have these advantages. Environmentally unsustainable practices are also sometimes used in their production such as burning firewood and dredging river sand, (Agevi, 1999), (Mbumbia et al., 2000).

Earth construction is very successful in arid areas, but significant stabilisation is required for adequate performance in humid areas. Unfortunately poor production practises of CSSB have resulted in a chequered history and a limited following (International Labour Office, 1987). Research conducted at Warwick by Kerali indicated that a six-fold increase in wet compressive strength could be achieved using improved curing regimes for CSSB, (Kerali, 2001). With good production control CSSB can perform quite adequately, but further improvements in material performance will help to outweigh sloppy production practices.

CSSB block presses have been designed and used for self-build initiatives in the past, for example, the Cinva-Ram (manual block press) and the Brepak (hydraulically assisted block press). These presses require high quantities of cement for adequate performance or are too expensive and complex for local production and maintenance. A less expensive and less complex machine would be more amenable to local production and small-scale capital investment. These are the areas where CSSB production technology needs to be taken and dynamic compaction shows promise.

1.2 Some notes on this thesis

This thesis is designed to report the academic findings from the research carried out during this Ph.D. project. Its function is also to present the information to an examining body for assessment for awarding the degree of Doctor of Philosophy to the author. The thesis has been written to reflect the chronological order of events with a minimum of forward and backward referencing of the different chapters.

The thesis is divided up into 9 chapters and each chapter contains a number of sections and further subsections. These three hierarchical levels are identified by numbers and break down the majority of the text into manageable portions. A further fourth level identified by bold italics is not numbered. Most chapters finish with a chapter summary.

After this introduction comes the literature review in chapter 2. Experimentation on quasi-static and dynamic compaction is described in chapters 3 and 4. The analysis of dynamic compaction is then reported in chapter 5. Chapter 6 was a difficult chapter to fit in chronologically, as it includes design analysis used in the production of the Test Rig. This was used to gain the data presented in chapters 4 and 5. However, this also includes the design suggestions and modifications for the Production Prototype. Chapter 7 details the overseas collaboration and technological dissemination of the Production Prototype, comparing the new machine with existing machines. An economic analysis and feasibility study for the Production Prototype is presented in Chapter 8. Finally chapter 9 summarises the conclusions made throughout the thesis and makes recommendations for further research to be conducted.

Data is presented in three different formats in this thesis. Graphs are used to show trends and to highlight possible relationships. Tables are used to present statistical analysis of the data collected. These two formats appear in the body of the text as close to their point of reference as possible, but not necessarily on the same page. Raw numerical data is recorded in the appendix for cross-referencing if necessary.

2 Literature review

Our having stated the need for low-cost housing in the previous chapter, this chapter of the thesis provides the background to the subject of interest. It will outline some of the existing practices and methodology for brick and block manufacture and analyse them for sustainability. Having established the potential contenders, a summary of raw material selection and characteristics will be given. Existing techniques of stabilisation will be reviewed and suggestions made for areas of possible improvement. Finally the previous research conducted on dynamic compaction will be outlined identifying the gaps in understanding and scope for further research.

2.1 An introduction to brick and block manufacture

Many different materials are used around the world for walling. Where quarried stone and timber are not readily available, earth is the most common material used. Earthen architecture has been used for centuries in many different parts of the world. (Houben & Guillaud, 1994) states: “Thirty percent of the world’s population, or nearly 1,500,000,000 human beings, live in a home of unbaked earth.” Accounts from the Bible (Exodus 1:11-14, 5:6,7) indicate that around 1500BC earth mixed with straw was a typical building material. Earlier accounts from the Bible (Genesis 11:3) also speak of burning bricks and using slime as mortar. Archaeological evidence in very dry areas have also shown that earth building was a highly popular material for dwelling construction. Earth is still used today in many parts of the world where access to other forms of building material is restricted by location or by cost.

Each building material has its own advantages and disadvantages. Some of the problems with existing materials are their poor use of environmental resources, poor quality control of the finished product and consequently a significant variation in durability. The long-term sustainability of some methods is being questioned in many places. Other alternatives are being sought after that are environmentally sustainable whilst also being of a suitable strength and durability for use in humid areas.

2.1.1 Existing processes described

Within this thesis it is not necessary to provide an exhaustive list of building materials as previous authors have already done this, (Houben & Guillaud, 1994), (Stulz & Mukerji, 1993). Instead the focus will be on some of the more popular methods of providing walling at tolerable cost. Hollow and aerated concrete blocks, clamp and kiln fired brick and compressed and stabilised soil blocks (hereafter CSSB) are the five main building materials chosen for consideration. These have been selected because they are well known, have been adequately assessed for performance and have appropriate standards for evaluation. Aerated concrete is considered an advanced material and is included here for comparative purposes as its performance and characteristics are highly desirable for low-cost building.

***Aerated concrete blocks* –** Aerated concrete is a light form of concrete (density around 500kg/m^3) that uses coal ash from power stations and omits the use of coarse aggregate. A cement rich mixture has a foaming agent applied to it before the material is pumped or can be cast into suitable moulds (Craig, 1997). It has been developed into a high performance building material and is currently marketed as aerated concrete blocks or “Aircrete” (Thermalite, 2001). Although these blocks are not considered suitable for heavy load-bearing conditions, (over 7MPa), they are wholly adequate for low-rise structures such as typical homes. Other features such as high wall area per block, low thermal conductivity, easily shaped by hand tools and low moisture penetration make this a highly attractive material.

<u>Figure 2.1 – Aerated concrete blocks</u>

Aircrete　　Thermalite Block　　Thin mortar joints

The above photographs show the structure of aircrete, it’s ease of handling and the high dimensional accuracy required for thin mortar joints. The textured surface of the blocks help to bond the render to the block, (if render is desired as it is not necessary on external walling).

Hollow concrete blocks **–** These are relatively expensive due to their need for graded sand and large amounts of cement (12-17% by weight). If manufactured properly they can have very high strength and good durability. Significant cost and weight reduction is achieved by removing material from the central region of the block. Machinery for production requires a vibrating table to settle the cement mix into the mould. Sometimes, instead, a heavy hinged lid slammed a couple of times or low pressures are applied to compress the material.

Good dimensional accuracy means that these blocks can be laid on a 10mm mortar joint. However, due to the voids in the block, mortar falls down these holes and is wasted. (In calculating the required mortar it has been assumed that the mortar actually used is closer to that needed for the surface area of the entire top surface of the block rather than just the edges where a joint is made with the neighbouring block.) These blocks are sometimes rendered for aesthetic reasons, which we will omit from any calculations for the time being.

Kiln-fired brick **–** Over the centuries the process of burning clay to make brick has become more and more automated, sophisticated and complex, but not necessarily more cost effective, particularly in developing countries. (Parry, 1979) very eloquently and persuasively describes two methods of brick production in terms of cost and shows quite clearly that where labour costs are low, kiln-fired brick production would be economically unsuitable. Kiln-fired brick production requires a high capital investment and a significant amount of infrastructure to support production. Brick production must be located near to high quality clay deposits (often unavailable locally), staff needs to be more highly skilled, spares and servicing is highly specialised and energy requirements are considerable. Production output is very high, typically 10,000 - 30,000 bricks per day and needs to be continuous if to achieve high efficiency and to achieve the greatest return on investment. Modern kilns efficiently recycle heat, giving a modest energy consumption per brick (3MJ) (International Labour Office, 1990).

The characteristics of such kiln-fired bricks are highly desirable as the material has a high wet-compressive strength and does not deteriorate rapidly over time even in the harshest of climates (Hanson, 2001). The material is pleasing to the eye and is sought after as an attractive material for home building.

Clamp-fired brick **–** Can be inexpensive in monetary terms because the raw materials can usually be dug from the ground fairly locally and the energy required firing the brick could come from collected firewood. Clamp fired bricks are of a lower quality than kiln-fired bricks and can tolerate the use of smaller and poorer sources of clay deposits. Forming the blocks requires a wooden or metal mould and after forming they are laid out to dry. After drying they are stacked into a clamp where fires are burnt inside (Parry, 1979), (International Labour Office, 1990). These fires raise the temperature of the blocks to the point where the particles bond together (Stulz & Mukerji, 1993). Thorough burning is necessary to fire all the blocks properly and this takes several days to achieve and uses approximately 16MJ of energy per brick.

The finished blocks can be quite badly misshapen and this requires a thick layer of mortar between the blocks, sometimes as thick as 20mm. Furthermore, if the blocks are poorly fired then in order to achieve adequate durability they may need to be rendered as well. Fired blocks are usually considered attractive and so they are not generally rendered unless necessary.

Figure 2.2 – Clamp fired bricks

This is a poor example of clamp-fired bricks in Uganda, demonstrating thick poorly used mortar. The result is unattractive and wasteful of cement.

However, due to the high cement content of the wall and the fired brick used it will probably achieve adequate durability.

The poor dimensional accuracy of the bricks can be clearly seen in this photograph.

Compressed and Stabilised Soil Blocks – These blocks use the same parent material as plain earth blocks but offer the significant advantage of wet compressive strength. Improved strength and stability in wet climates is generally achieved by a combination of two methods of stabilisation. One method is to compact the soil by applying some mechanical effort to reduce the voids in the material. Increasing the density of the material gives it a higher compressive strength and also reduces the potential for ingress of moisture into the block (Houben & Guillaud, 1989), (Norton, 1997). CSSB are further stabilised with the addition of a chemical stabiliser that helps to bind the particles together. Cement or lime are expensive additives but are generally available and although the practice of adding them to soil is reasonably popular the results can be disappointing unless it is done carefully (International Labour Office, 1987).

CSSB can be compacted using low or high-pressures or dynamically compacted via impact (Houben & Guillaud, 1994). The greater the level of compaction the greater the compressive strength of the block and the more effective any added stabiliser becomes, (Gooding, 1993). CSSB compacted to higher densities are also usually more dimensionally consistent and therefore can be laid using a thinner mortar layer of around 10 – 15mm. Some CSSB need to be rendered or waterproofed in order to enhance their protection from the elements (Yogananda, 1999), but this can usually be avoided with higher levels of compaction and or higher quantities of stabiliser. Making a hollow CSSB can be done by straight-through perforations or deep and shallow frogs (Houben & Guillaud, 1989), (Centre for the Development of Industry, 1998). Each of these reduces the material volume present and therefore reduces the stabiliser quantity necessary for each block.

Figure 2.3 – Compressed and stabilised soil blocks (CSSB)

Here is a good example of a wall made of stabilised soil blocks in Uganda.

The blocks are approx. 0.4 × 0.2 × 0.125m and may have some voids through the centre. No render has been applied to the wall and no significant roof eaves have been used.

A solid cement rich foundation had been used to build the blocks onto. This is a high quality construction and would have been quite costly but not as much as hollow cement blocks.

Figure 2.4 – Interlocking CSSB

The development of interlocking blocks, as shown here in Tanzania, provides a very low cost alternative for walling. These are advertised as “no mortar” and “no render” blocks, reducing the cement requirement to soil stabilisation alone. Very high dimensional accuracy is required for these blocks to be successful.

2.1.2 Processes analysed for sustainability

Some of the materials listed in the previous subsection require different methods of construction in order to produce satisfactory walling. These differences will be assessed and compared for the production of one square meter of wall. Some for example require more mortar between block courses to compensate for the irregularities of the block shape. The issue of durability is only qualitatively explored in the texts and no quantitative results for the durability of these materials has been found. Durability is typically defined in the range of “poor” to “excellent” (Houben & Guillaud, 1994), (International Labour Office, 1987). This could have something to do with the absence of suitable accelerated or short-term tests to indicate potential durability or as a relative measure between different materials.

Throughout the assessment of these materials it is assumed that they are able to perform the basic function of a finished wall, (i.e. support its own weight and the weight of any structures above it for a long period of time withstanding environmental attack). Whilst aesthetics play a part in material selection, it will be considered small by comparison to the material performance and cost and will therefore be ignored.

Possibly one of the most striking differences between these different types of walling units is their width. Some hollow concrete blocks are 250mm (10”) wide whist the clay fired brick is usually only 103mm (4”) wide. A wider block is more stable and can be used to build taller walls with a high slenderness ratio, (width/height). A single-skin brick (103mm thick) wall is not considered to be stable enough except for in-fill walling between columns and beams or for relatively small structures. In our analysis of single-skin brick construction we have included a buttress pillar of two bricks at 1-metre centres, which increases the brick and material requirement by almost 25%. It is more common to make a single-skin block wall of closer to 150mm (6”) thick and this practice has been extended to two storey construction successfully.

We have chosen to assess the walling materials described in the previous section according to the four following measures:

- Primary energy consumption for production and delivery in MJ per m^2 walling
- Cement usage in kg per m^2 walling
- Ranking for suitability for small-scale (‘local’) production
- Ranking for suitability for on-site production using mainly on-site materials

The table and associated notes below is a summary of a spreadsheet used to make the calculations for comparison of the selected materials. The raw data for this comparison can be found in Appendix E.

It is important to notice that the different materials each has a wide variety of different characteristics and the table below does not attempt to normalise these characteristics in any way. For example the wet compressive strength of the hollow cement block will be several times larger than the low-density CSSB and this is not really indicated from the energy and cement cost. The table does however generally give the real costs of each type of walling.

Table 2.1 – Comparison of different walling materials

Material	Dimensions ($l \times b \times h$)	Note	Energy	Cement	Suitability for production 'Locally'	On-site
	Mm		MJ/m^2	kg/m^2	Ranking	(1 = best)
High-density CSSB	290 × 140 × 90	1	290	18.7	2	1
Low-density CSSB	290 × 140 × 90	2	420	34.1	1	1
Brick (kiln-fired)	215 × 105 × 65	3	430	8.1	2	3
Brick (clamp-fired)	215 × 105 × 65	4	1340	11.4	1	2
Hollow Cement block (N)	300 × 150 × 200	5	430	27.0	1	2
Hollow Cement block (F)	300 × 150 × 200	6	590	27.0	1	2
Aerated-cement block	440 × 140 × 215	7	230	12.4	2	3

Notes

0. All cement is assumed to have been transported 100km.
1. High-density (2000kg/m^3) solid blocks manufactured on-site from local soil/cement mix (5% cement), laid with 10 mm of soil/cement mortar (20% cement) and no render.
2. Low-density (1700kg/m^3) solid blocks manufactured on-site from local soil/cement mix (10% cement), laid with 15 mm of soil/cement mortar (20% cement) and 15mm render.
3. Kiln-fired brick (3000MJ/1000 bricks) laid with 10 mm of sand/cement mortar (20% cement) and no render, double brick buttress column at 1m centres.
4. Clamp-fired brick (16000MJ/1000 bricks) laid with 15 mm of soil/cement mortar (20% cement) and no render, wall has double brick buttress column at 1m centres.
5. Hollow (50% voids) cement blocks made from 10% cement mixed with gravel and sand from nearby source, with a 10mm mortar joint, (sand/cement, 4:1 ratio).
6. Hollow (50% voids) cement blocks made from 15% cement mixed with gravel and sand transported from 50km away, with a 10mm mortar joint, (sand/cement, 4:1 ratio).
7. High-tech aeration process using coal ash mixed with cement (15%) to make a very light (480kg/m^3) material. Laid with a 3mm mortar joint using cement rich paste (50% cement). Blocks transported 50km.

There is increasing evidence that local production of hollow concrete blocks and clamp-fired bricks use unsustainable resources. The practices of using river sand for the former (Shan & Meegoda, 1998) and firewood as fuel for the latter (Mbumbia et al., 2000) are environmentally unacceptable, and in any case likely to face rising prices driven by increasing scarcity. Consequently the only small-scale method of block manufacture left deals entirely with the stabilisation of locally available un-graded soil.

Of the materials listed above, only three use less than our assumed target of 15kg of cement per m^2 of walling; two of them are unsuitable for local production and the third has an extravagant energy requirement. High-density CSSB is the only material that uses but a modest amount of cement ($\approx$ 18kg/m^2), has a low energy requirement and is suitable for local and on-site production. The question that now needs to be answered is whether or not methods exist that may further reduce the cement requirement of high-density CSSB to less than 15kg per m^2 of walling.

If we want to concentrate on the field of CSSB block production for its environmental and sustainability advantages, we still need to make significant improvements on performance whilst reducing the cost. It has generally been noted that CSSB walling that is un-rendered and unprotected does not perform satisfactorily over time in the humid tropics. Consequently improved levels of stabilisation are required without a significant increase in cost. This could be done via a combination of improved material compaction and improved stabiliser effectiveness. As the chemical stabiliser is the most expensive ingredient in the block then its quantity should be reduced to the lowest level possible for achieving the necessary strength and durability.

There are a number of options for the chemical stabilisation used. Additives such as cement can be used to make a high-cement but thin-walled block, or a very low-cement mix throughout a very dense solid soil block or as a surface render over compressed soil without any cement present. Mortarless construction is a very attractive proposition and would be quite compatible with in-wall curing of very-low-cement homogenous blocks. Also the production technique employed for producing very-low-cement blocks is quite straightforward and permits immediate stacking after moulding making it more attractive than thin walled blocks that require more careful handling and curing.

Applying render over raw soil doesn’t yield satisfactory results in the long term, as the render permits moisture migration to the soil behind and swelling and shrinking can cause render cracking and failure. This may be improved with higher levels of compaction that resist moisture migration better. The costs of a high-cement render over a soil wall are still higher than a very-low-cement block and would be a costly maintenance requirement if the wall had to be re-rendered. Consequently the very low-cement solid block has been selected as a prime candidate for further research and improvement.

2.2 Raw material selection and characteristics

Soil is readily available over the vast proportion of the landmass on the planet and hence it is not surprising that it has been widely used for dwelling construction. This section of the thesis will briefly summarise the existing knowledge of soil selection for making CSSB. Wide ranges of soil are suitable for this building material and their defining characteristics will be outlined below. The use of cement as a chemical additive is also very common in CSSB manufacture. The use and understanding of cement is widespread and it’s application to soil had received much attention in recent years. Adding cement to soil is very different to adding it to aggregates and the requirements and characteristics of such a union will be described in the following subsections.

2.2.1 Properties and analysis of soil

Soil is found deposited on the surface of the earth and can consist of many different types. The variation in the soils present at the surface can be attributed to a series of natural effects working on the area over time (Craig, 1997) (Houben & Guillaud, 1994). On the very surface of the soil one typically finds material with a large amount of organic compounds present. This is unsuitable for block manufacture and can usually be distinguished by a musty smell especially on heating (Norton, 1997) (Wolfs-kill et al., 1965). Material underneath this organic layer is much better as it usually contains a cross section of particle sizes and includes a proportion of small soil particles called “fines”. These are usually defined as particles passing a 63μm mesh and consist of silt and clay. Clay is necessary in block production because it aids the workability of the mixture, increasing levels of consolidation and improving green strength, (International Labour Office, 1987). Larger particles “sands” found in soil can generally be assessed as minerals that are silicas, silicates or limestones. As well as the solid rock particles and fragments, soil will have a proportion of water and air that fill the gaps between adjoining particles in the soil. This gives natural soil a non-homogenous and porous nature.

Systems for identifying some major characteristics have been developed to define different ranges of soil characteristics. The most common of these is the size distribution of the soil particles. (Houben & Guillaud, 1989) lists the physical characteristics than can define a sample of soil, including: colour, shape, apparent bulk density, specific bulk density, size or texture, moisture content, porosity or voids ratio, permeability, effective surface area, adhesion, specific heat capacity, dry strength and linear contraction. Chemical properties are also sometimes of interest particularly when a chemical additive is used. These chemical properties include the composition, mineral content, metallic oxides, pH levels and sulphates in the soil (Craig, 1997).

With so many different characteristics that one could discover about a sample of soil, it would be foolhardy to try and discover them all in every situation that soil is to be used for making CSSB. Only a small number of characteristics are of real relevance to the scientist testing the soil. The chemical composition of the soil is of little importance once the absence of unstable compounds and organic matter has been established. The physical properties are of greater interest for making CSSB as these will help to determine its ease of mixing, forming, de-moulding, porosity, permeability, shrinkage, dry strength and apparent bulk density.

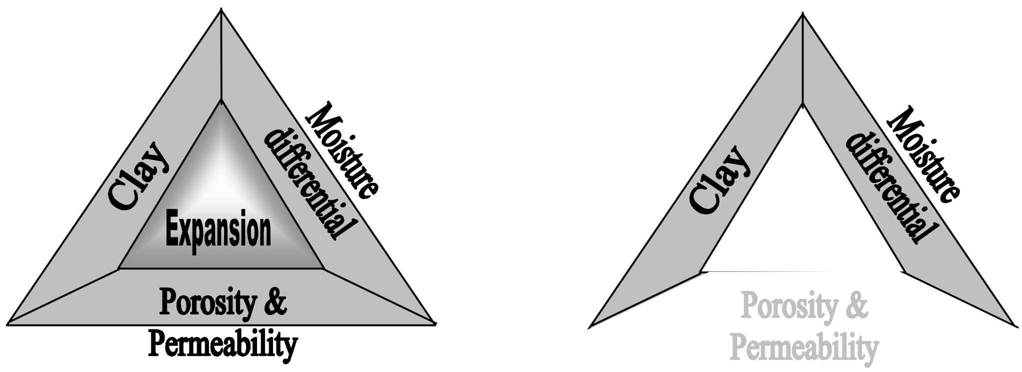
Controlling or monitoring the clay fraction is important in making CSSBs. Too much clay results in unacceptably high expansion upon wetting, requiring excessive amounts of cement to combat this. Too little clay causes low adhesion between particles and hence causes high breakage rates on de-moulding of the CSSB. An optimum fines content for making CSSB was suggested by the United Nations to be about 25% of which more than 10% is clay, (Gooding, 1993). A more useful range of particle sizes suitable for building with earth is given in (Norton, 1997) as follows:

Sand/fine gravel	40 - 75%
Silt	10 - 30%
Clay	15 - 30%

From the literature it is unclear how much a change of say ±5% to the clay content will have on the overall performance of the CSSB. Controlling the moisture content in the mixture is also important, but generally the texts use a simple drop test to determine an acceptable range. The accuracy of this test is fairly low and what effect the possible variation in the moisture has on the finished product is not clear.

The detrimental characteristic of expansion and contraction of a CSSB can only occur if three characteristics are present: "Clays" and "Porosity & Permeability" and "Moisture differential". If any one of those is absent then expansion and contraction will not occur, (ignoring thermal expansion and contraction). See diagram below.

Figure 2.5 – Characteristics of CSSB expansion



We need clay to be present in CSSB and it is impossible in humid climates to avoid moisture differentials so the only characteristics that we can seek to remove or reduce are the porosity and permeability.

2.2.2 Basics of cement usage

As a stabilising material cement is well researched, well understood and its properties clearly defined, (Akroyd, 1962), (Popovics, 1998), (United Nations, 1972). Portland cement is readily available in most urban areas, and usually available in semi-urban areas, as it is one of the major components for any building construction. Earlier studies have shown that cement is a suitable stabiliser for use with soil in the production of CSSB, (International Labour Office, 1987).

Cement is mainly composed of Lime (CaO) and Silica (SiO_2) which react with each other and the other components in the mix when water is added. This reaction forms combinations of Tri-calcium silicate and Di-calcium silicate referred to as C_3S and C_2S in the cement literature, (Akroyd, 1962), (Lea, 1970), (Neville, 1995). The chemical reaction eventually generates a matrix of interlocking crystals that cover any inert filler (i.e. aggregates) and provide a high compressive strength and stability.

The diagram on the following page attempts to illustrate how these crystals actually give the material strength. The basic mechanism is friction of point contacts between the particles taking place at a microscopic level. The duration of time for this reaction to take place is not precisely defined. There is however the definition of the "critical time" after which further working of the mix causes breaking of the crystals that have formed but before the total matrix has gained strength. The flow chart that follows shows the reaction and their effect with respect to time.

Figure 2.6 – Sketch of crystalline cement growth in sandcrete

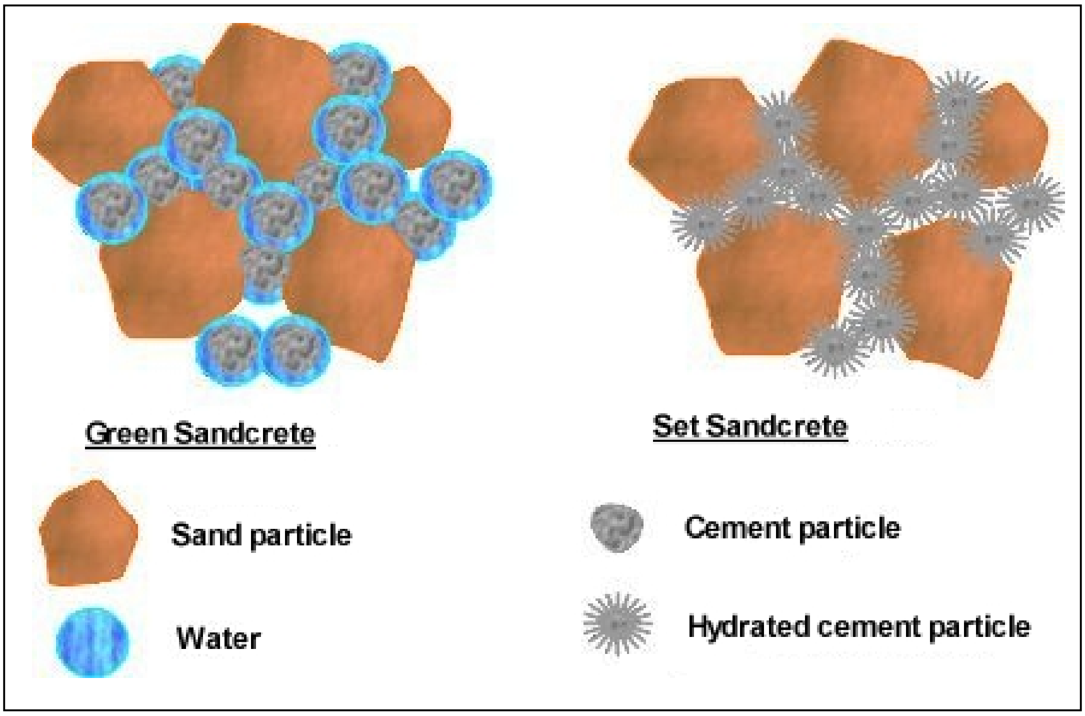
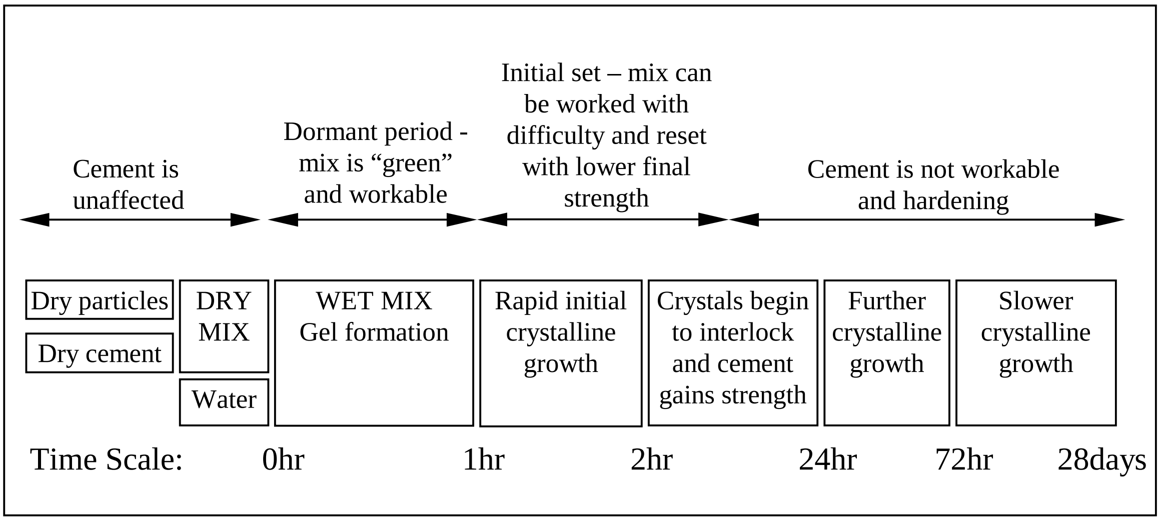


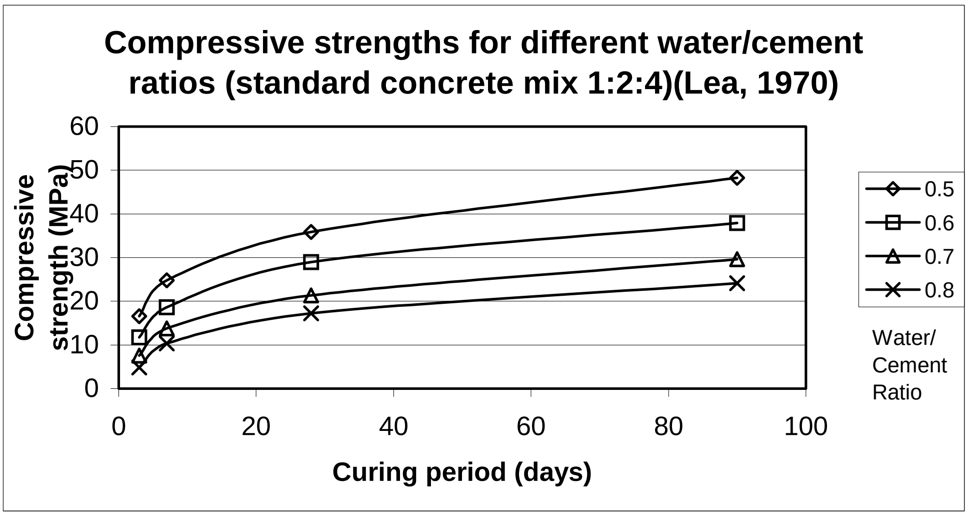
Figure 2.7 – Flow chart of the cement hardening process



Cement is usually mixed with an aggregate to form concrete. The aggregate is usually inert filler that makes up the bulk of the material and the cement coats the aggregate in the gaps (Teychenne et al., 1988). The concrete industry has recognised that the achieved strength of concrete is highly dependent on the quantity of voids present in the mixture before curing. (Akroyd, 1962) suggests that the presence of 5% air voids will reduce the strength of a concrete mix by 33% and 8% voids by 50% compared to a sample with 0% voids present. To aid the particle intimacy, different aggregate grades are mixed together giving a spectrum of particle sizes that reduces the quantity of air voids in the material.

The water used to mix the concrete plays an important role both in placing the material and in achieving strength. The quantity of water used is typically calculated using an appropriate “water-cement ratio”. The minimum water/cement volume ratio is between 0.22 and 0.25 (Akroyd, 1962), for adequate cement hydration, but this is generally increased to the order of between 0.5 and 0.8 for normal mixes, (Lea, 1970).

Figure 2.8 – Compressive strength of concrete with different water-cement ratios



Very low water-cement ratios yield a highly unworkable mixture and more water has to be added to form the mixture into the desired shape. Additional water is called the “free-water” content and is calculated from the slump or Vebe time test. This water does not form part of the chemical reaction and will eventually evaporate from the concrete leaving voids of air throughout the material (Neville, 1995). In order to keep the free-water as low as possible concrete can be compacted or vibrated to aid workability and consolidation.

2.3 Soil stabilisation

The methods of making earthen structures more durable fit into primary and secondary categories. Primary methods stabilise the raw material making it more durable, and subsequently any structure made from it, whilst secondary methods provide protection from the elements rather than enhancing the material properties. Soil stabilisation improves the characteristics of the soil so that it can tolerate greater loading and perform better when it is exposed to the elements. Stabilisation usually involves work of some kind to be done to the soil, and this section will briefly describe some of the methods of soil stabilisation that have been used.

Raw earth can be stabilised in a number of different ways. (Houben & Guillaud, 1994) suggests that there are six different mechanisms for soil stabilisation, namely: raising density, reinforcement, linking, binding, waterproofing and water repellent treatment. The two most common techniques used in block manufacture are binding (with chemical additives such as cement or lime) and raising density (by some method of compaction). In this section the guidelines of cement addition to soil will be summarised, the methods of soil compaction will be explored and the current role of stabilisation will be outlined with a view to further possible enhancements.

2.3.1 Cement addition

By now we have a better understanding of the way cement bonds with itself and other particles in making concrete. We also know some of the important guidelines that need to be followed when making successful mixes of concrete. Furthermore, many of these guidelines and principles should be followed when mixing cement with soil.

The quantity of cement that is required for adequate stabilisation depends on several criteria, namely; the required compressive strength, soil type, environmental conditions and levels of quality control. Cement can very easily be wasted if it is not utilised in the correct manner and significant cement reduction can be attained through good production management and quality control. Controlling the moisture content, level of compaction and the curing regime will play a big part in getting the most from the added cement.

For relatively quick analysis of soil characteristics for cement stabilisation the CSSB literature suggest the use of a linear shrinkage mould, (Houben & Guillaud, 1994), (Norton, 1997), (International Labour Office, 1987), (Rigassi, 1995). Soil is mixed with water to its liquid limit and then left to dry out in a mould with dimensions 40 × 40 × 600mm. The linear shrinkage is measured and the quantity of cement required to adequately stabilise the soil is calculated. The following table is taken from (International Labour Office, 1987), recommending the cement to soil ratio for different soils of known linear shrinkage.

Table 2.2 – Cement to soil ratio for different soils of known linear shrinkage

Measured Shrinkage (mm)	Cement to soil ratio
Under 15	1:18 parts (5.56%)
15 – 30	1:16 parts (6.25%)
30 – 45	1:14 parts (7.14%)
45 – 60	1:12 parts (8.33%)

The volumetric shrinkage of a CSSB will depend on the fraction of clay present and the moisture content of the mix. If the moisture content is low then the shrinkage will also be low when it dries out. This is harmonious with the recommended low water-cement ratio for maximising the cement strength. However, on subsequent wetting the forces exerted by the expansive clay particles must be restrained by the cement matrix in the CSSB. So the cement requirement will also depend on the degree of wetting that the CSSB will experience, hence the environmental conditions.

As mentioned earlier the degree of wetting depends on the ability for moisture to migrate in and out of the material, dependent on the porosity and permeability. Methods of reducing the migration of water to the clay fraction can therefore also provide a method of reducing the cement required for adequate stabilisation. This technique is more commonly referred to as compaction or consolidation and will be main focus of the next sub-section.

2.3.2 Compaction of material

Within the civil engineering industry there are several methods of compaction that are used in ground stabilisation that use methods of static, vibration and dynamic blows to compact soil (Parsons, 1992). Block compaction uses similar methods and similar technology only on a smaller scale and typically compaction takes place in a confined space rather than in unconfined open areas (Houben & Guillaud, 1989), (Norton, 1997). Block compaction has predominantly used vibration or slow steady squeezing (quasi-static) compaction to achieve the desired levels of soil consolidation. Until very recently the dynamic element used in block manufacture has been limited to the compression piston coming into contact with the surface of the soil at some speed followed by static pressure being applied to the material (Houben et al., 1994).

The following three figures demonstrate the different types of compaction, the particle intimacy around the O.M.C. (as found in (Head, 1980)), and the relationship between moisture content and achieved density for different compaction energies.

Figure 2.9 – Unconfined, semi-confined and confined compaction

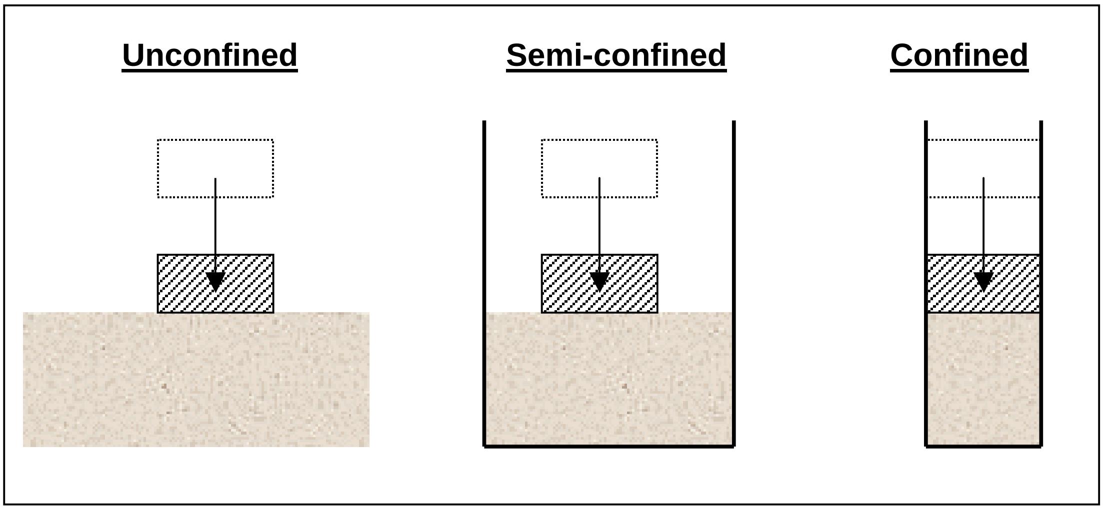


Figure 2.10 – Diagram of particle intimacy around the O.M.C.

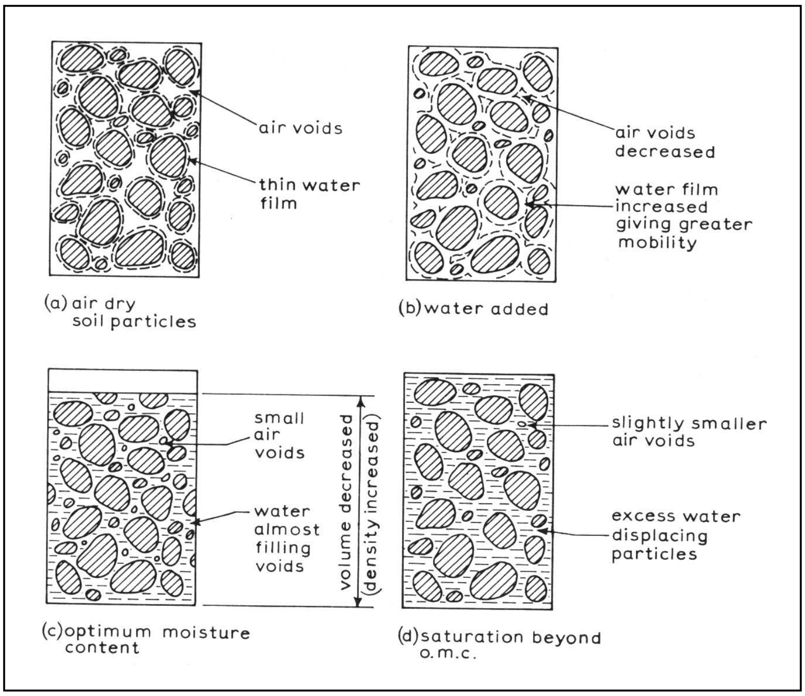
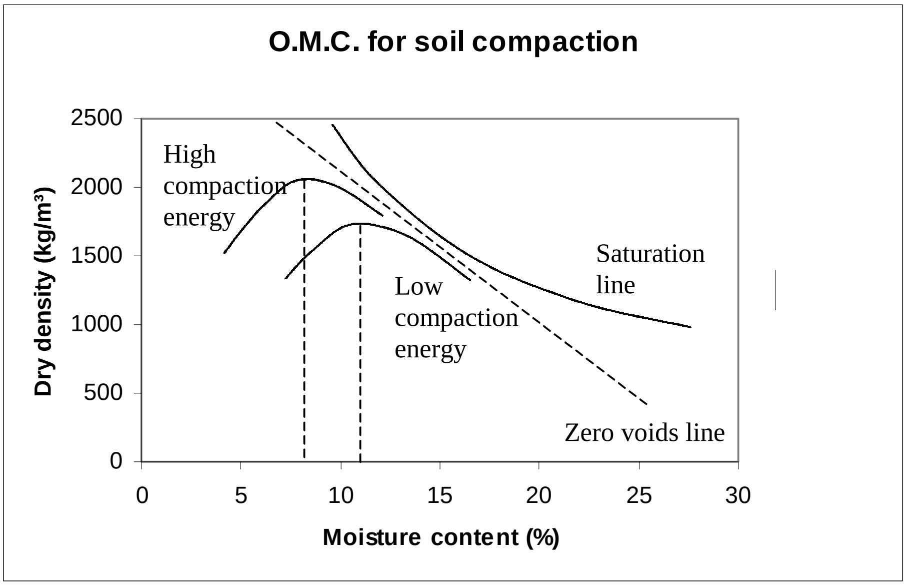


Figure 2.11 – O.M.C. for soil at different compaction energies



Improved levels of compaction have a significant effect on the compressive strength of the sample and on the effectiveness of the cement stabiliser added. The following two graphs are presenting data collected by (Gooding, 1993) to indicate the relationships between cement content, compaction energy (defined in MPa pressure) and the resulting bulk density and subsequent 7-day wet compressive strength.

Figure 2.12 – Relationship between cement content, compression pressure and 7-day wet compressive strength

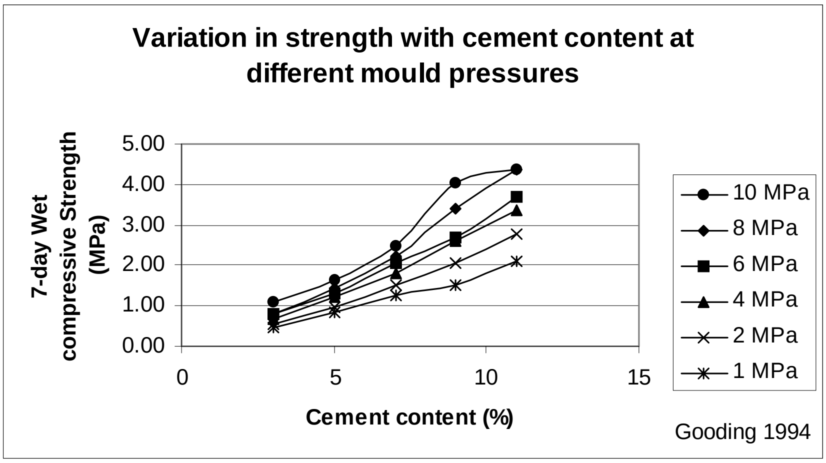
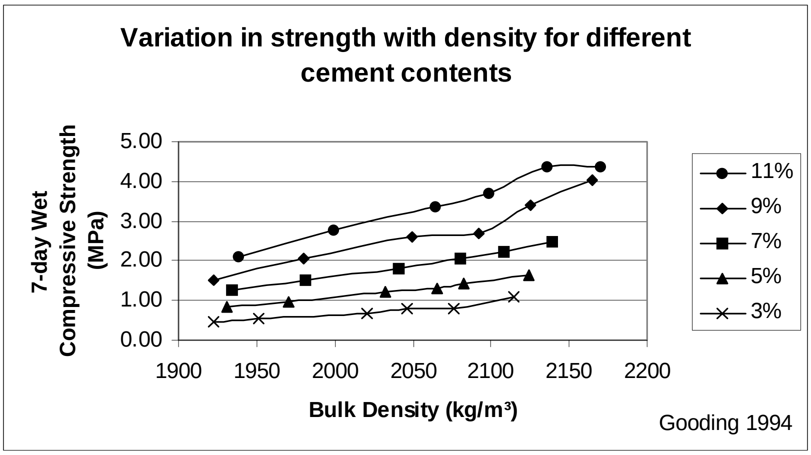


Figure 2.13 – Relationship between bulk density and 7-day wet compressive strength for different cement contents



The above data clearly shows the significant advantages that increased compaction offers. If a CSSB could be compacted to a higher density, then for the same ultimate strength the cement content could be reduced. The trade off is an increased energy cost for a reduction in chemical additives. Another thing that is apparent is the possible miss match of moisture contents desired for optimum compaction for a given energy and optimum moisture content for cement curing. This issue of what is the most appropriate moisture content to be used for a given compaction energy needs to be resolved.

2.3.3 Current role of stabilisation and its possible extension

It is usually the poor or underprivileged that need and build low-cost housing and this has an effect on the processes used to make the building material. Minimising material cost and machine requirements are typically more important than reducing labour costs. Consequently it is not uncommon to find block manufacturers using cheap machinery and minimising the stabiliser content. The photograph below shows some of the poor applications of cement to stabilise soil blocks in what seems to be a well-organised production yard. This illustrates the need for better understanding of the processes at work in soil stabilisation and improved quality control throughout the process of production. Significant savings in cement or much higher quality blocks could be attained if these were put in place. Furthermore, there is little way of knowing the performance of a finished CSSB without conducting crushing tests so the purchaser has to trust the seller as to the quality of the blocks being sold.

Figure 2.14 – CSSB production yard showing poor curing practice

CSSB production yard showing the poor block manufacturing technique of taking cement stabilised blocks and leaving them out to dry in the open rather than curing them in humid conditions. Uneven drying from the edges results in very weak cement bonds where they are needed most. Edge defects can also be seen where blocks have been damaged through poor handling and low strength. Dried blocks in the background look very grey indicating high cement content probably necessary for adequate strength.

Apart from improving the understanding of cement use and implementing better quality control in production there are advancements that can be made in the production technology as well. A study conducted by (Gooding & Thomas, 1995), as part of an Overseas Development Agency report, calculated that using more expensive high-pressure compression machinery to make blocks was not as economically attractive as adding more cement and using a low-pressure machine for the estimated life of each machine.

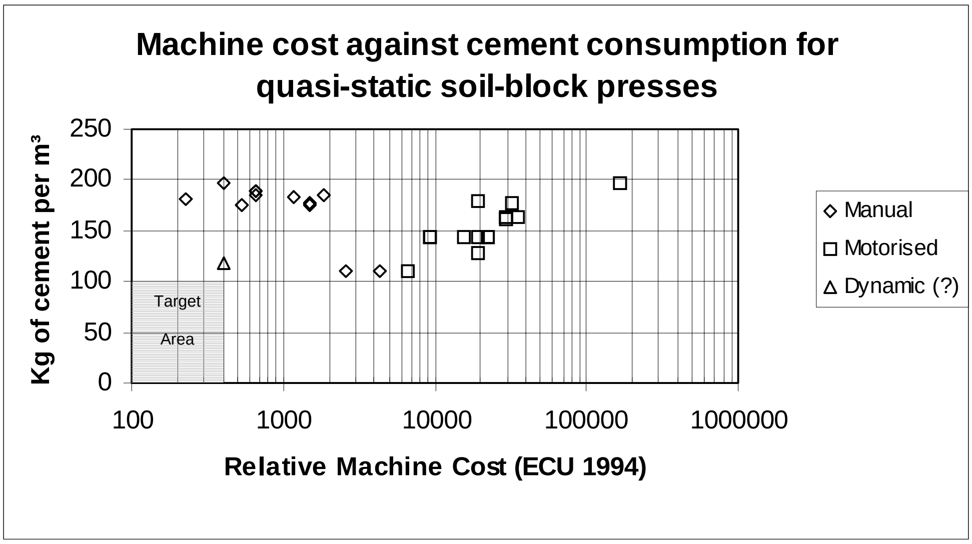
Terms used for different moulding pressures as described in (Houben & Guillaud, 1994):

Very low pressure	1 – 2	MPa
Low pressure	2 – 4	MPa
Average pressure	4 – 6	MPa
High pressure	6 – 10	MPa
Hyperpressure	10 – 20	MPa
Megapressure	20 – 40	MPa

Improvements in methods of compaction would greatly improve the characteristics of the finished CSSB, both immediate green strength and long term strength as well as reducing the porosity and permeability of the material. It could also facilitate in the further cost reduction of the material making the CSSB building material available to a wider group of people and more attractive and desirable for dwelling construction. From the graph shown in Figure 2.12 an increase in moulding pressure from 2 MPa to 10 MPa can double the wet compressive strength. Alternatively such an increase in pressure could successfully reduce the cement content from 11% to 7% without a significant loss in wet compressive strength. However, the argument that increasing the applied pressure can enable one to reduce the cement content without any loss in overall performance is not as attractive as it sounds. The high-pressure compaction ensures that more material is required for a block of given size and whilst some savings can be made in expensive additives such as cement the increased density offsets this advantage slightly.

The graph below is a combination of interpreted data from (Gooding, 1993) and (Houben et al., 1994), (who complied a CSSB machine catalogue with specifications and approximate prices). The graph shows the relative machine cost compared with the approximate cement usage per cubic metre of walling material. Gooding calculated the relationship between cement content, applied pressure and wet compressive strength. From this data it was possible to suggest the required amount of cement to achieve a certain block performance. This block performance has been taken as a 7-day W.C.S. of 2MPa and using the projected *density* that the CSSB press could achieve a cement requirement was calculated. It clearly shows the area of where a machine is necessary to fill the low-machine cost and low-cement usage area. Data points in the upper region of the graph represent machines that use low-pressure lever systems whilst points in the lower region include machines with a hydraulic compression mechanism delivering high-pressure. Motorisation does not necessarily procure greater applied pressure but usually results in higher machine cost and faster rates of production.

Figure 2.15 – Comparison of machine cost and cement requirement



From this graph it can also be shown where dynamic compaction could feature and that it may be able to provide the low-cost and low-cement block machine that is currently unavailable. Research into dynamic compaction is in its infancy, but significant discoveries have already been made. The next section of this chapter will describe the research conducted to date and to identify gaps where further investigation is required.

From the above it is now possible to summarise some of the different aspects of compaction that we would like to see improved or included in stabilised soil block manufacture.

- Higher density blocks exhibit greater strength and increase the effectiveness of any chemical stabiliser added to the block.

- Reduced porosity and permeability of the material hindering the ingress of water and slowing the rate of deterioration in humid conditions.
- Increased green strength to reduce breakages during early handling and to enable stacking immediately after forming.
- Compaction achieved via a mechanism that is easy to manufacture and to maintain using tolerable levels of materials, equipment and skills.

It is hoped that dynamic compaction may be able to deliver on some of these criteria and the following section will give details of experiments carried out to determine the potential of full-size dynamically compacted blocks for use in the humid tropics.

2.4 Dynamic compaction research

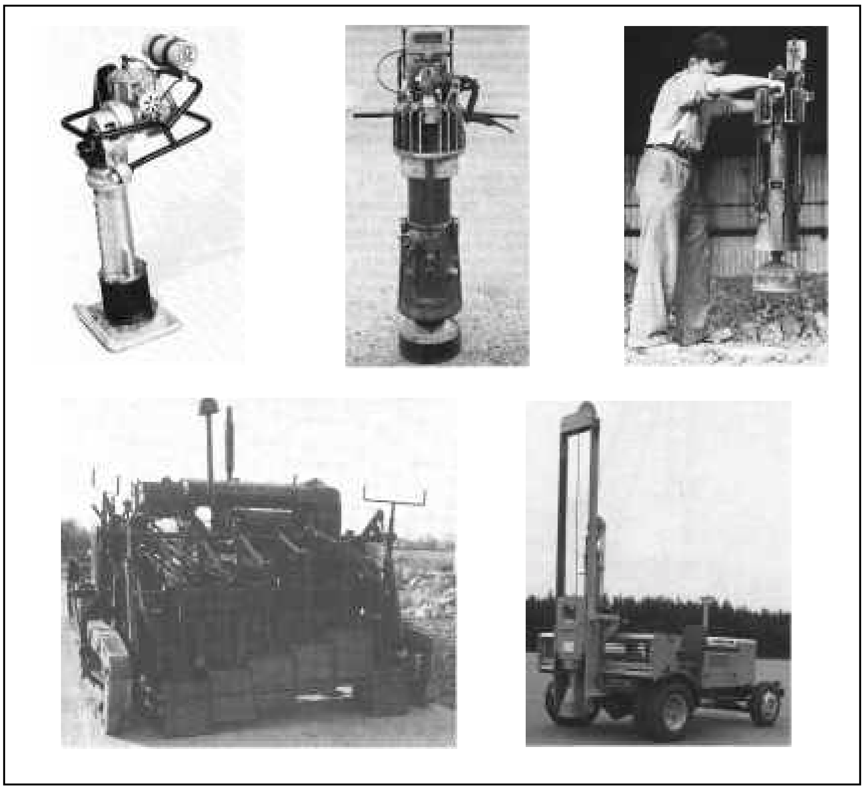
It has been found that the information on dynamic compaction of stabilised soil blocks is very scarce. Up till now the author is only aware of two pieces of work that cover this topic, and only one of which he has been able to access. There are however, other publications that deal with the subject of soil compaction using impact, both from a theoretical and practical viewpoint. This section will review the conceptual research on impact compaction carried out by the Transport Research Laboratory identifying both theoretical models proposed and experimental results of significance. Following this an overview of the optimisation experiments conducted by Gooding will be presented identifying areas of further research required. Finally the application of impact compaction to soil blocks as carried out by Montgomery (the author) will be reviewed.

2.4.1 Conceptual Research

The majority of the research into dynamic compaction of soils has been conducted by the civil engineering industry with a view to improving soil consolidation, (both efficiency and depth) for subsequent placement of a structure. Road compaction is one of the areas that has particularly focused on this technique with the application of vibrating rollers and vibrating sheep's foot rollers, (Parsons, 1992), (Ingles & Metcalf, 1972), (Hausmann, 1990). These techniques do not really apply to the confined soil compaction that would be experienced with CSSB manufacture by impact. However, research has also been conducted on dropping weight compactors, vibro-tampers, power rammers and single and multi-weight dropping machines, (Parsons, 1992). Such research is of greater interest despite the compaction taking place in the unconfined rather than confined state.

The figure below contains photographs of some of the "impact" compacting equipment available in the civil engineering industry. Each deliver a known quantity of energy over a known surface area and would be given an "energy transfer per unit area" rating of between 4.3 – 120kJ/m² depending on the model and settings of compactor.

Figure 2.16 – Civil engineering compaction equipment using impact



Theoretical and experimental analysis of the impact method was also conducted to try and determine the most effective machine for ground compaction and to improve the understanding of impact compaction. A test rig was developed and tests were conducted on a variety of soils by dropping a known mass through a known height and comparing the compaction with the 2.5 kg standard compaction test for the same soil. Measurement of the impact pressures experienced by the soil was also carried out using piezoelectric gauges buried at 150mm and 300mm beneath the surface.

Parsons discovered that for an approximately constant energy transfer per blow a smaller mass lifter to a higher height delivered a higher pressure than a larger mass lifted from a smaller height. Furthermore the pressure experienced at 300mm below the surface was approximately half the pressure experienced at 150mm below the surface. This suggests a linear decay in pressure from the surface to some point beneath the surface dependent on the energy transferred in the blow and soil characteristics. The application of more than one blow demonstrated that higher degrees of compaction could be attained for the same optimum moisture content as used in the 2.5kg standard compaction test.

Theoretical analysis of the compaction of the soil by impact included mechanical characteristics of the soil (plastic deformation on impact), the impactor velocity, mass and impactor surface area. From these a theoretical pressure applied to the soil could be calculated. The main variable of interest in this analysis is the dynamic modulus of deformation, which can only be found by measuring the deformation of the soil after an impact has been delivered. The analysis suggests that this variable will be constant for the type and moisture content of the soil. Whist this may be true for soil in the unconfined state it almost certainly is not the case with soil confined in a mould.

Another text (Scott & Pearce, 1975) suggested that impact compaction can be modelled as a highly damped spring with characteristics that depend on the Young's Modulus, Dilation Velocity, Poisson's Ratio, and Elastic Limit of the soil. They give an equation that links these characteristics to the rate of deceleration of a moving mass in order to model the stress and movement at the impact surface for an unconfined mass of soil. They investigate the effect of unsaturated and saturated soils monitoring the elastic properties, surface deflection and stress concentrations. They also suggest a model for a one-dimensional situation that may be analogous to dynamic compaction within a constrained mould. However the assumptions made for the development of the model are not clear so application to the specific case of dynamic compaction of soil blocks has not been possible.

Throughout the texts that deal with soil consolidation there seems to be a lack of explanation or understanding of what is actually happening during compaction. Each text can describe the characteristics before and after the consolidation has taken place but they don't attempt to explain the process. Soil is a complex material and consolidation takes place at the microscopic level of particle placement and interface with other particles. This is a very difficult area to model, but it would be beneficial to discover some of the prevalent mechanisms that occur during compaction whether by impact or otherwise.

2.4.2 Compaction optimisation of confined soil samples

Research in the previous sub-section concentrated on the compaction of un-confined soils. This sub-section covers in some depth the research conducted by (Gooding, 1993). It is the sole text that has put forward a systematic approach to determining the most effective method of compacting soil confined in a mould. It has also been helpful in experimental design for this Ph.D. and it's summary helps to explain some of the parameters selected for dynamic compaction of CSSB. Although Gooding thoroughly investigated the dynamic compacting process, he did not stabilise any of the dynamically compacted samples with cement. The characteristics and effectiveness of the combined processes was not looked into.

Before dynamic compaction was investigated, he looked into the process of quasi-static compaction (i.e. slow-squeezing). His research included varying the cement content, the applied pressure, mould taper, double and single sided compaction, pressure cycling and mould wall roughness, (some of these results have already been shown in earlier sections). Throughout his tests he used a fabricated soil called '*soil-A*' with a constant moisture content of 8%. His analysis of *soil-A* is included in Appendix A.

A relationship between compression pressure, 7-day wet compressive strength and cement content was developed and a model suggested estimating the wet compressive strength of a sample with known cement content and applied pressure. This model was based on actual experimental results taken from tests carried out using a range of pressures and cement contents. A small cylindrical mould specified in BS1924 was used for all of these tests. All the cylinders had their wet compressive strength tested after seven-day curing and subsequent soaking for 16 hours.

Gooding investigated the efficiency of impact compaction using *soil-A* without any cement present. The compressive strength had to be estimated from the achieved density compared with compressed samples. The wet compressive strength of dynamically compacted soil samples could not be measured, as the compacted samples would break apart when immersed in water. Each sample received the same energy but by different impact arrangements and the achieved density was recorded. Density was calculated by measuring the final cylinder height (±0.05mm) and mass (±0.1g) on ejection from the mould. Each cylinder received a constant 279 J/kg and the mass of each cylinder was kept at around 1.66 kg. Other factors such as the number of blows and impactor momentum were varied to find any optimum parameters for this technique.

Each sample received one of 1, 2, 4, 8, 16, 32 or 64 blows. The optimum number of blows (number that yielded the greatest density) was found to be at 16 blows, but it was also noted that only a 3-4% reduction in compaction efficiency occurred when this was varied from 8 to 32 blows for each of the different masses.

Different momentum transfer was also explored for the same energy transfer. Smaller impactor masses were lifted higher and larger impactor masses were dropped from a lower height. Three different masses were used in the experiments on the samples (23.35kg, 35.00kg and 46.80kg) and it was noted that the bigger masses dropping at slower speeds were more effective. Yet, the 23.35kg mass and the 35.00kg mass were only 0.4% and 0.2% less efficient respectively at the 16 blow configuration than the 46.80kg mass.

It was discovered that the method of dynamic compaction was more effective in consolidation than quasi-static compression for the same total energy transfer into the material. The selection of 279J/kg was taken from the energy required to quasi-statically compress *soil-A* to 9.7MPa. It was possible to achieve the same density through impact compaction with the application of only 25-50% of the energy necessary with quasi-static compression.

The experiments conducted by Gooding to optimise dynamic compaction for the same energy transfer is very interesting and helpful for machine design, but it does not indicate the levels of compaction that could be possible with additional energy. Nor does it indicate the best method of applying a certain amount of energy if a limited number of blows are to be applied. Are earlier blows more effective than latter ones? Does the impact energy need to increase as material becomes more compacted? The research also does not include the very necessary ingredient of compaction with cement present. Impact compaction of cement is not practised in the concrete industry, so the combination of these two elements needs to be experimentally assessed.

The only example of buildings made from dynamically compacted material was surveyed by Gooding as part of a survey of CSSB structures in several countries (Gooding & Thomas, 1995). He compares them with other structures in the area, constructed using similar appropriate techniques, with some interesting observations. The building made from dynamically compacted low-cement (6%) material, using a manual impact machine, had been standing for 10 years and was still in excellent condition. Other buildings in the same area made from CSSB that had been made using expensive motorised compression machinery and 9% cement were already deteriorating after only two years. Block production costs using the dynamic machine were 25% and 40% less than “sandcrete” and cement blocks respectively and were almost half the cost of CSSB made using the motorised compression machines. Such significant savings in cost and improvements in performance certainly warrant further investigation.

2.4.3 Application of impact to block compaction

As part of an undergraduate degree programme the author undertook a project labelled "Design and realisation of a test rig to research the production of full size dynamically compacted soil-cement blocks"(Montgomery, 1997). This project was completed in 1997 and achieved the following results. A full-size dynamic compaction test rig was designed and manufactured. The design chosen was suited to the level of technology available in developing countries. Several blocks were produced and their densities and surface penetration resistance was measured. Two blocks were stabilised using cement, but these were not used in the experimentation as they were only intended to be demonstrator blocks.

The theoretical formulae suggested in (Parsons, 1992) were applied to the results attained during the dynamic compaction of full-sized blocks in 1997. The table below shows the increase in energy that was delivered by the impactor as the soil block was compacted. It also indicates the total transfer of energy into the block after a certain number of blows.

<u>Table 2.3 – Summary of results of dynamic compaction conducted in 1997</u>

Impactor stroke (m)	0.1364	0.1571	0.1661	0.1748	0.1814	0.1866	0.1913
Energy(J) / blow		55.5	58.7	61.7	64.0	65.9	67.5
Energy increase		7.3	3.2	3.1	2.3	1.9	1.6
Energy transferred	0 blows	1 blow	2 blows	4 blows	8 blows	16 blows	32 blows
after blows (J)	0	55.5	104	221	468	980	2035

Between the initial resting-place of the impactor and the resting-place after one blow there is a distance of (0.1571 – 0.1364) = 0.0207m. This is the deformation of the soil during impact. The velocity of the impactor prior to impact can be assumed to be $V = \sqrt{2gh} = \sqrt{2 \times 9.81 \times 0.1364}$ = 1.64 m/s …etc.

Below is a table with the rest of the calculations for multiple blows during a compaction cycle using the above formulae.

Table 2.4 – Analysis of forces from dynamic compaction conducted in 1997

	1 blow	**2 blows**	**(3 blows)**	**(4 blows)**	**(8 blows)**	**(16 blows)**	**(32 blows)**
Velocity prior to final impact (m/s)	1.64	1.76	1.81	1.83	1.88	1.91	1.94
Stopping distance (m)	0.0207	0.0090	0.0043	0.0044	0.0016	0.0006	0.0003
Mean deceleration (m/s²)	64.6	171	375	384	1070	2800	6380
Calculated stopping time (s)	0.025	0.010	0.005	0.005	0.0018	0.0007	0.0003
Pressure generated (MPa)	0.057	0.152	0.332	0.341	0.948	2.488	5.656
Dynamic modulus of deformation	2.8E+6	1.7E+7	7.6E+7	7.8E+7	5.7E+8	3.8E+09	1.9E+10
Mean force in tonnes (final impact)	0.233	0.616	1.35	1.38	3.85	10.1	23.0

Note: The velocities and stopping distances for the blow numbers in brackets have been linearly estimated from compaction data for multiple blows. These figures are probably accurate to ±10% and can only show the continued trend.

Two things are immediately obvious from the table of results above. Firstly, the dramatic increase in force that is applied during impact between the first blow and much later ones. Secondly, the dynamic modulus of deformation for a soil compacted in a confined manner increases as it becomes compacted. Therefore the characteristics and behaviour of the soil will change during the compaction process. This will make accurate modelling the compaction significantly more difficult than an unconfined soil with a constant dynamic modulus of deformation.

Gooding quasi-statically compressed a block to 9.7MPa and noted that it achieved a bulk density of 2038kg/m^3. This compaction pressure equated to a transfer of 279J/kg. By comparison, Montgomery dynamically compacted a full size block to a bulk density of 2040kg/m^3 by applying 32 blows to it from a 36kg impactor. This block received a total of 2035J from the falling impactor. For a 10kg block this equates to approximately 204J/kg, some 26% less energy required than the quasi-statically compressed block, which is a significant saving. This research indicated that the savings in energy that Gooding had found could be extrapolated onto full size blocks and therefore warranted further research.

Montgomery also did not stabilise any of the full size dynamically compacted blocks as these were trials to test the feasibility of full size compaction. Consequently there are not any known characteristics of the produced blocks apart from a handful of penetrometer tests done on the freshly de-moulded blocks. These give little indication of the core strength and only sought to establish the level of uniformity of density throughout the block.

2.5 Chapter summary

The assessment of different building materials at the beginning of the chapter helps to focus on the more appropriate materials that can be employed in developing countries. For environmental reasons clamp-fired brick and ‘sandcrete’ blocks not sustainable in the long term. Aerated cement blocks and kiln-fired blocks are large-scale industrial processes that require high levels of technology and are unsuitable for local block manufacturing techniques. The only remaining material with any immediate promise is the compressed and stabilised soil block, which has a reasonable following in areas where the soil is suitable and fired-brick is scarce. However, this material is still inferior in performance to more advanced materials and needs to be improved to gain greater acceptance.

The potential for appropriate technology to help in this area has been seen already with the development of manual block presses in the 1950's (Cinva-Ram block press). It is hoped that technology may have a further role to play in improving the block manufacturing techniques still further. The project will be directed to provide a sustainable solution that is technologically appropriate and provides a significant improvement over existing processes.

An attractive option for cost reduction is the evidence that the cement content can be reduced if the material density is increased. However, current technologies that deliver increased material density are prohibitively expensive and are not economically attractive. There is evidence that an alternative method of compaction through the application of a dynamic blow may provide high levels of densification without the prohibitive machine complexity and cost. Earlier research into this area has indicated that not only is dynamic compaction a possibility for soil block production but also potentially is more energy efficient in compaction than quasi-static compression.

To date there has been very little experimentation into the application of a dynamic blow to compact a soil block. If the method does indeed have significant advantages over the more expensive hydraulically assisted high-pressure quasi-static compression then this warrants further research into this area. Research will require the designing and development of a suitable experimental rig with a view to low levels of complexity and cost if it is to be transferred for use in developing countries. The academic understanding of dynamic compaction also seems to be highly limited and any research into this area should seek to explain some of the dominant mechanisms at work during the impact blow. This will require the close analysis of the impact blow and possible model generation to describe the actions taking place within the material throughout the compaction process.

3 Preliminary experiments with stabilised soil

The previous two chapters identified the growing need for low-cost housing and the potential for dynamic compaction of CSSB to provide a low-cost environmentally sustainable alternative to existing walling materials. This chapter now focuses on the findings of early experiments conducted to improve the understanding of quasi-statically compressed stabilised soil and the processes involved in its production. Throughout these experiments several independent variables have been selectively altered and the effect on one or several dependent variables has been noted. These findings have aided the process of parameter selection for later tests to be conducted on *dynamic* compaction of stabilised soil, dealt with in the following chapter.

These preliminary experiments were conducted for several reasons. Firstly to reduce the large number of independent variables to a manageable number. Secondly to identify main relationships not covered in the literature. Thirdly to select (for those independent variables not held constant) what experimental values to use. And fourthly to assess experimental variability and hence select suitable sample sizes.

3.1 Summary of input variables and output measures

Many parameters can be varied in the production of stabilised soil. Careful experimental design will therefore be needed to minimise the number of experiments to assess key characteristics and relationships. For the moment we will omit mentioning the variables associated with dynamic compaction as they feature in the next chapter. The ranges of input variables, as shown below, were largely determined by practical constraints.

- Moisture content – taken as a percentage of solid material (range 2-10%)
- Compression pressure – usually recorded in MPa (range 4-20MPa)
- Mould wall thickness (range 0.5-32mm)
- Size and shape of sample (tall and short cylinders and blocks)

Other input variables kept constant except where explicitly stated otherwise are:

- Soil type – including particle size distribution (soil-B)
- Cement content (5% by weight)
- Mould wall surface finish (machined to approximately IT10)
- Delay before compaction (5-10min)
- Curing period and conditions (100% humidity for 7 days)
- Ambient temperature and humidity (20°C and 35% relative humidity)

The output measures used to monitor the process are as follows:

- De-moulding force
- Projected Dry Density (P.D.D.)
- Wet Compressive Strength (W.C.S.)
- Non-destructive tests

These output measures give the basis for determining any trends from the results and will be used to identify the relationships between variables of interest.

3.1.1 Input variables

From the literature review there seems to be a number of relationships that need clarifying. For example, what is really happening with the water in the compacted sample? How does the quantity of water present at compaction affect the consolidation combined with the cement curing to achieve strength? Varying the water content during the tests may help us to determine what are the dominant mechanisms in action.

Moisture Content (M.C.) **–** The previous chapter mentioned the problem of selecting the moisture content. The soil literature suggests one thing and the concrete literature another. The only solution is to explore different moisture contents to see what effects they have on both the achieved density and the final strength. Previous experiments using *Soil-A* investigated a range of moisture contents up to 10% (at which the samples became unmanageable). Consequently the moisture content used during the experiments ranged from 2 to 10%. 6% was deemed a good compromise between the different factors in achieving density and necessary strength, and was used as a normal value. Selected values for moisture content 2, 3, 4, 5, 6, 7, 8, 9 and 10%.

Compression pressure **–** This can vary from very-low-pressure 1MPa to hyperpressure 20MPa. The majority of experiments employed 10MPa, but other pressures were looked at to discover trends within the material. Selected values for pressure 4, 6, 8, 10, 12 and 20MPa.

Mould-wall thickness **–** The relative stiffness of the mould wall to restrain the pressures applied might have some effect on the degree of possible consolidation for a given energy transfer or pressure applied. The effect that this has on the compaction characteristics has not been explored previously. The mould wall thickness is of greater importance with dynamic compaction. It is believed that the forces applied to the mould walls during compaction are smaller and of much shorter duration than those occurring during quasi-static compression. Moulds from 0.5mm up to 32mm thick were used. Clearly the extra cost for much thicker moulds is a significant consideration and using the thinner moulds is much more attractive. Selected values for mould wall thickness 0.5, 2, 8 and 32mm.

Size and shape **–** For research purposes it is inconvenient and expensive to manufacture full-size blocks to check each variable and characteristic. Indeed, previous dynamic compaction research had been carried out on 100mm diameter cylinders as opposed to blocks for this very reason. Similarly, after a few initial experiments on full-size blocks (290 × 140 × 90 mm), most experiments were conducted on small short cylinders (∅54.4mm, approximately 45mm high). These cylinders were easier to manufacture, cure and test, than the full-size blocks that would also be produced later in the research. Extrapolation of findings from small cylinders to full-size blocks is not straightforward, however the *ranking* of properties at one scale is likely to be the same as the ranking at a different scale.

Unless specifically stated the input variables below have been kept constant throughout the experiments. The selection of those constant values is discussed each in turn:

***Soil type* –** In the field this can vary considerably and it is known that some soils are more suitable than others for the production of CSSB. In the previous research conducted by Gooding a suitable soil was mixed from builders sand and kaolin clay. Some of this original soil (*Soil-A*) was still available in small quantities and consequently was used for a few initial experiments. Later on in the research a different soil using similar ingredients had to be mixed and this was called soil-B and was used for the remainder of the experiments. The analysis of Soil-A and B can be found in Appendix A.

Cement content – Cement is usually the dominant cost in CSSB production, so the reduction of its quantity is very desirable. The relationship between cement content and compressive strength has been well researched in the past so it is not necessary to investigate it further here. How much cement is necessary depends on three factors, the clay content of the soil used, the degree of compaction during moulding and the required wet compressive strength of the finished block. Previous stabilised soil research (Rigassi, 1995) has indicated that cement contents below 2 or 3% will not actually enhance the wet compressive strength or improve stabilisation. Consequently 5% by weight has been selected as the smallest amount of cement practical to employ for CSSB and has been used in the vast majority of the experiments.

***Mould surface finish* –** Throughout the tests the moulds had a machined surface finish to an approximate tolerance grade of IT10.

Delay before compaction **–** As soon as moisture is added to the dry soil/cement mixture the cement reacts chemically with the water. Any delay between adding the water and the material compaction should therefore be kept as short as possible. For research purposes any variation in the delay period between mixing and compaction should be minimised as this might have an effect on final properties. Typically the compaction occurs between 5-10 minutes after mixing of water into the mix. The order of production within a batch may also have an effect on the final sample characteristics and while this is not large it is a factor that requires addressing. Indeed it would be useful to know whether a significant loss-of-strength penalty is incurred when a period as long as say 20 minutes elapses between mixing a batch and making the final sample.

Curing period **–** The vast majority of the tests conducted on stabilised soil had the soil curing for 6 days in a 100% humidity environment (samples placed in sealed bags containing water-saturated air). This was then followed by soaking for a further 24hours giving a total curing time of 7 days. This was a suitable period as many texts gave data for cement properties at 7 days and enabled reasonably quick feedback of results from tests.

Ambient temperature and humidity **–** The experiments have been carried out under laboratory conditions, typically at 20°C and with a low relative humidity.

3.1.2 Output measures

The list below describes the set of different measures used for assessing the finished material after stabilisation and consolidation. Each measure was not carried out on every experiment, as this was often either impractical or impossible. For example the wet compressive strength of a compacted sample cannot be found if the sample has no cement. These measures have been the key method of identifying any relationships between input variables.

- De-moulding force – measured using the compression rig
- Projected Dry Density – calculated from measured bulk density
- Wet Compressive Strength – measured using the compression rig
- Non-destructive tests – penetration resistance and indentation size

De-moulding force – After compacting a CSSB in a mould it must be successfully removed from the mould without damage. The majority of small tests done using a compression machine involved a straight-sided mould and the compacted sample was pushed up from the bottom. Where possible, this ejection force was measured.

Projected Dry Density (P.D.D.) – The *dry density* is calculated from the dry mass of the solids divided by the volume of the material. Since we know the dry mass of the material prior to mixing and compaction we can calculate the P.D.D. of the material upon ejection. The P.D.D. gives an indication of the level of consolidation that has occurred irrespective of the water present in the sample. The bulk density measure includes the mass of the water in the density calculation and therefore yields a higher value for density that can be misleading, especially where different moisture contents have been investigated. (Dry densities between 1900 and 2000 kg/m³ are considered to be excellent for CSSB manufacture (Houben & Guillaud, 1989), (International Labour Office, 1987).).

Wet Compressive Strength **–** Existing low-cement CSSB manufactured by low-pressure compaction have compressive strengths adequate for the majority of low-rise structures *provided that* water penetration is kept to a low level. However, when saturation of such CSSB has occurred it has often proved to be too harsh for the material to withstand whilst maintaining a load: surface flaking (spalling) or even collapse has followed. Wet compressive strength is measured by placing a cured and water-saturated sample between the jaws of a compression machine. Then slowly applying a force to the sample recording the maximum force sustained. Wet compressive strengths of over 2MPa are considered to be excellent for CSSB (Houben & Guillaud, 1989).

Non-destructive tests **–** Some of the samples produced had tests performed on them to indicate characteristics such as the 'green' strength of the material. These tests were also conducted to try and develop surrogates for determining characteristics that could only be found otherwise by destroying the sample. A penetration test was used to determine the green strength of a formed block. This involves pushing a rod a specified distance into the surface of the block and recording the force required (usually done using a penetrometer). The green strength of the block will not depend on its cement content, as the cement particles will not have had time to hydrate and add any strength to the material. Another test, the indentation test, was also developed specifically for the stabilised soil material as the penetrometer test proved unsuccessful in many circumstances. This test was developed towards the end of the research so unfortunately the number of tests conducted is relatively small.

3.2 Experiments employing full-size blocks of Soil-A

A number of full-size blocks were produced early on in the project using *Soil-A* (the predecessor to *soil-B*) to learn about the interaction of variables using the quasi-static compression technique. A Brepak earth block press was available for block production that could deliver pressures of up to 10MPa and this was used to compress blocks with different moisture contents to 10MPa.

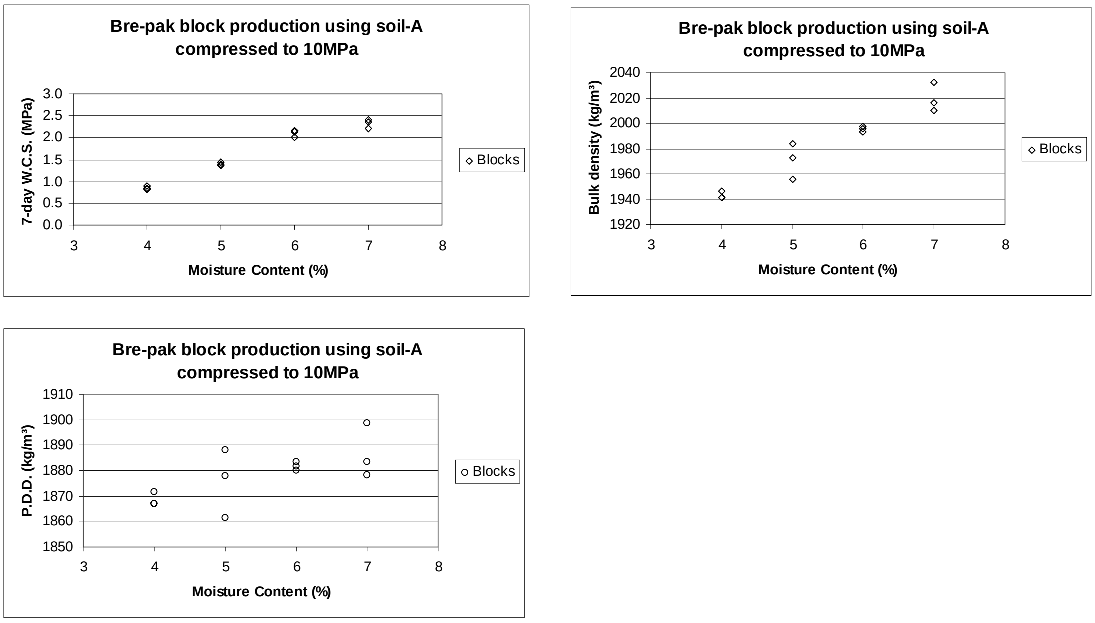
3.2.1 The effects of moisture on strength

The data included in Appendix G shows the measured results from 12 stabilised blocks that were produced using the Bre-pak machine using *soil-A* at four different moisture contents (4, 5, 6, 7% by weight) and a constant compression pressure of 10MPa. Due to an error in the mix calculation the cement content was 5.2% instead of the intended 5% which is a small error considering the variability of the material as a whole. The processing time for the production of each block was approximately 15 minutes to include dry and wet mixing of the material, compression and ejection.

Figure 3.1 below shows the variation in the 7-day W.C.S., the bulk density and the P.D.D. with moisture content. Increasing the moisture content from 4% to 7% delivers over 100% increase in strength yet only a 4% increase in bulk density and less than

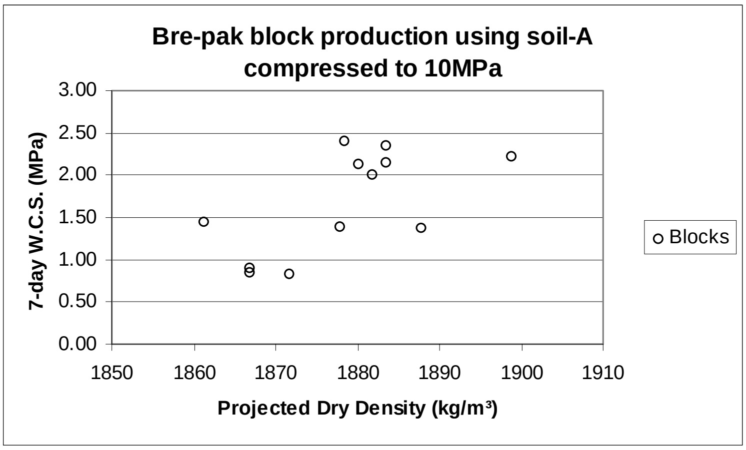
1% increase in P.D.D. These results would seem to suggest that a higher water/cement ratio is in fact not as deleterious as originally anticipated. In fact the contrary seems to be the case.

Figure 3.1 – Strength, bulk density and P.D.D. variation with moisture content



The graph indicates that there may be optimum moisture content for *strength*, as it would appear that the graph is levelling off above 7%. This could correspond to the optimum moisture content for *density* as described in the soil literature, but the shape of the graphs showing density do not confirm that this is the case. However, we can see that the increase in strength is in some way connected to in the increase in density. Comparing strength and density on the same graph (shown in Figure 3.2) demonstrates the possible relationship that exists between the two output measures. Unfortunately, there is quite a bit of spread in the data presented and this makes it difficult to see any relationship clearly.

Figure 3.2 – Strength against P.D.D. for full-size blocks

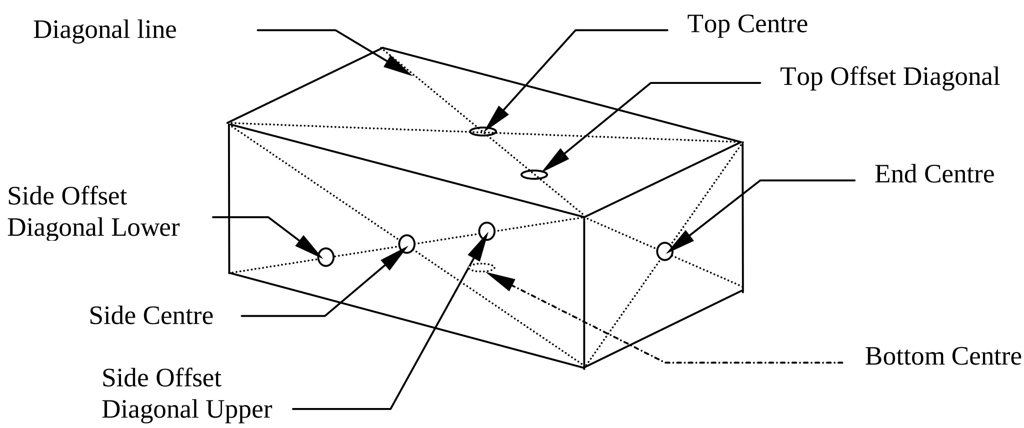


We believe that a denser material has a greater compressive strength, but we cannot conclude that 1% increase in P.D.D. alone can result in 100% increase in W.C.S. The extra water seems to be enhancing the strength of the material, a phenomenon that contradicts the cement literature. It is possible that additional water is permitting a better curing of the cement. However, when considering that the water/cement ratios for 4% and 7% moisture are about 0.8 and 1.4 respectively, these values are much higher than the recommended guidelines for concrete manufacture. Clearly the water content of the mix has a major effect on subsequent block properties.

3.2.2 Moisture content effects on penetration resistance

The results discussed in the previous subsection suggest that pushing the water content even higher than 7% would be advantageous to final strength. However blocks with a very high water content have so little green strength that they become unmanageable. The excessive quantity of water does not permit sufficient cohesion between particles and the blocks regularly break during ejection and subsequent handling. Consequently a compromise has to be made. CSSB texts mention the problem of handling but fail to give any guidelines for production except by using trial and error. To address this question of green strength another set of seven blocks made from *soil-A* were produced in the Brepak machine. *soil-A* was used without cement, compressed to 10MPa and with a range of moisture contents from 2% to 8.7%. Immediately upon demoulding, a soil penetrometer was used to measure the penetration resistance up to a maximum pressure of 0.45MPa. Penetration sites were chosen on the surfaces of the block to determine if there was significant variation in surface strength in different portions of the block. Figure 3.3 below illustrates the penetrometer sites used.

Figure 3.3 – Penetrometer sites on a finished block



Some of the blocks with very low moisture content were impossible to penetrate successfully, whilst blocks with higher moisture permitted easy penetration. The data for these blocks is presented in Appendix G and a summary of the data is shown below in Table 3.1.

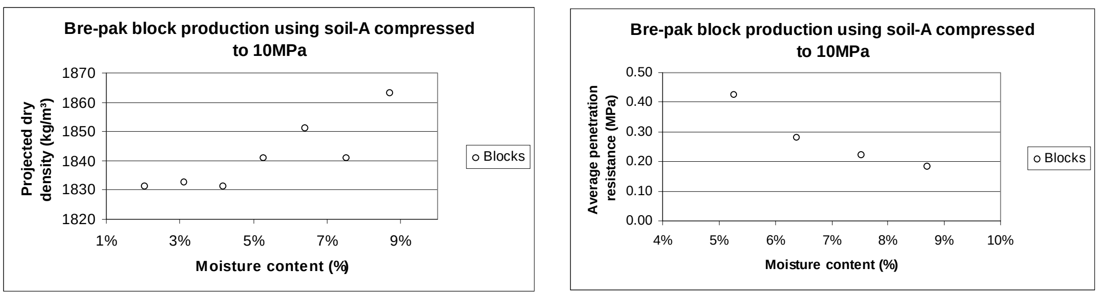
Table 3.1 – Penetrometer results from Brepak blocks

Moisture Content	%	2.0	3.1	4.2	5.3	6.4	7.5	8.7
Projected Dry Density	kg/m³	1831	1833	1831	1841	1851	1841	1863
Bulk Density	kg/m³	1857	1878	1896	1926	1958	1968	2013
Penetrometer Pressure (P_p) Average	MPa	N/A	N/A	N/A	0.43	0.28	0.22	0.18
(P_p) Standard Deviation	MPa	N/A	N/A	N/A	0.02	0.04	0.03	0.03
(P_p) Coefficient of variation	%	N/A	N/A	N/A	4.8	14.2	12.1	15.3

Note: N/A represents a reading that was off the scale of the penetrometer, (i.e. > 0.45MPa).

This data has been represented graphically in Figure 3.4 below concentrating on the effect of moisture on P.D.D. and penetration resistance. The relationship of increasing P.D.D. with moisture content that eluded us in the earlier experiment can be more clearly seen here, probably because the experiments covers a wider range of moisture contents. It is interesting to see that the penetrometer average plotted against the moisture content demonstrates that higher moisture levels yield lower penetration resistance and hence lower green strength. The fall in green strength, despite rise in P.D.D., suggests that the dominant mechanism controlling the green strength is the cohesion between particles and the amount of water that surrounds them rather than particle intimacy.

Figure 3.4 – P.D.D. and penetration resistance variation with moisture content.



Both of the sets of results investigated so far have demonstrated that there is some relationship between density and strength. As the density is an easily measurable quantity and can be calculated immediately after block manufacture it is an attractive measure both for research purposes and for quality control in block production. As the production thus far has concentrated on the production of blocks at a single compaction pressure and therefore constant energy transfer there is little variation in the achieved density. In order to suggest a strength density relationship it is necessary to explore a wider range of densities and corresponding strengths. Exploration of this is not very practical using either the Brepak machine or full-size blocks due to the problems with the machine and the high material cost of making lots of blocks. Consequently a smaller scale test needs to be applied for further analysis of this phenomenon.

3.3 Experiments employing small cylinders of soil-B

Further tests were conducted at a different scale and with a slightly different material, but greater consistency could be assured with the new material and the smaller scale permitted much faster sample production. This scale of production also offered greater control and reliability than with the Brepak block press. Small cylinder production commenced with the development of a set of cylindrical moulds with different wall thickness including 0.5, 2, 8, and 32mm. All of the moulds had an internal diameter of 54.4mm and produced samples around 45mm high with a dry soil mass of 200g (±0.5g). This was selected as a suitably small quantity that could be dumped into the mould without the need for tamping. Also, 200g was a round number for easier calculation of water mass to achieve desired M.C. Furthermore a sample of this size could also be easily manufactured by dynamic compaction using a similar rig and the same moulds, as the next chapter will explain more fully.

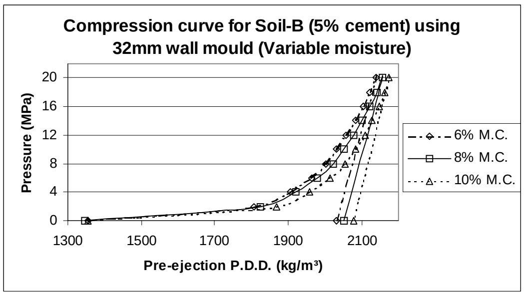
A new soil had to be developed at this stage to accommodate the future tests that were to take place on small scale as well as full-size blocks. This material consisted of builder's sand and kaolin clay and was supposed to be similar to the original soil-A. The sand material was oven dried to 105°C and sieved down to 5mm prior to mixing with the kaolin in the ratio of four parts sand to one part kaolin. As the majority of the tests conducted required stabilisation, cement (5% by weight of the total mix) was also added to the dry mix.

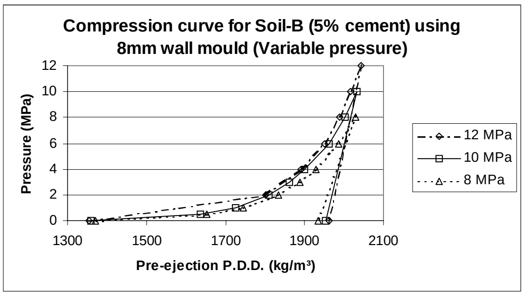
3.3.1 Pressure density relationship

Three separate investigations were carried out to determine the relationship between applied pressure and achieved density. The development of this relationship specific to soil-B was necessary in order to assess the effectiveness of dynamic compaction against a suitable standard. As dynamic tests were also going to be conducted at small scale this relationship would provide a good means of comparison between the two different compaction techniques. The first investigation involved the compression of three samples at three different moisture contents, 6, 8, 10%, and monitoring the density within the mould during the compression cycle up to 20MPa. The second investigation produced compression curves for three sets of three samples at 6% M.C. soil-B compressed to 8, 10 and 12MPa. The third investigation produced five sets of three samples at each of the following pressures 4, 6, 8, 10 and 12MPa and subsequently cured and crushed them to discover their compressive strengths. The data for all three investigations can be found in Appendix F.

The figure below shows a summary of the results of the first and second investigation measuring the P.D.D of the samples during the compression cycle. The left hand graph clearly shows that the further increase in moisture content to 10% increases the level of compaction achieved above 8% and 6% levels, which is consistent with the relationship suggested by the soil literature. It also indicates the elasticity of the material when the compression pressure is reduced to zero. Each curve represents the average of three sets of samples taken for each moisture content. On the right side is the graph showing the data from the second investigation displaying the compression curves for 8, 10, 12MPa and their respective elastic restitution for the single moisture content of 6%.

Figure 3.5 – Pressure density relationship for soil-B

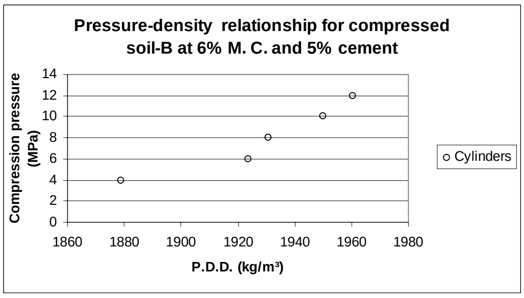


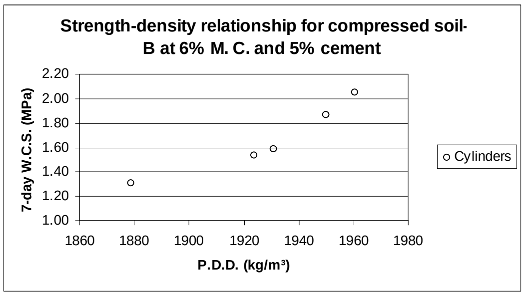


From these graphs we see that the P.D.D. that can be achieved by 10MPa pressure will be around 1950kg/m³ for the 6% M.C. condition. This will be taken as the target for the dynamic compaction tests to confirm the potential of dynamic compaction providing the necessary degree of consolidation. The moisture content of 6% was selected for these tests, as it seemed to be a good compromise between final strength and green strength that could be used on the full-size tests later.

Figure 3.6 below displays the data for the third set of samples produced under different compression pressures. Each point represents the average of three points of data and this most easily demonstrates the general trend of increasing pressure leading to increased P.D.D. and subsequent 7-day W.C.S. It was noticed that the variation within the batches was quite high and this led to questions regarding moisture loss or decreased workability over time in each batch.

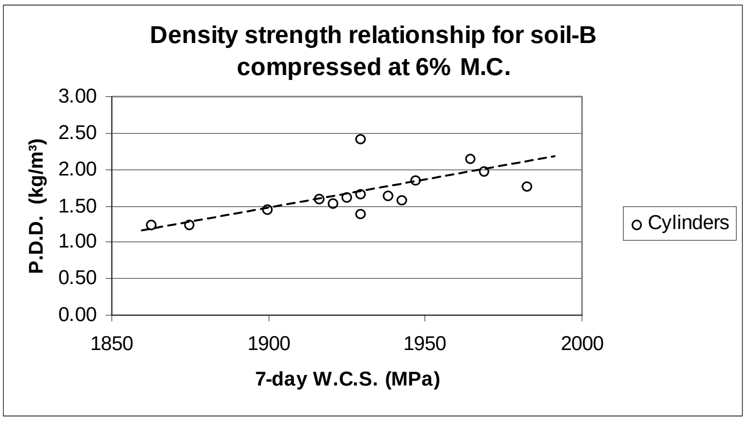
Figure 3.6 – Results of compression tests on soil-B at 6% M.C.





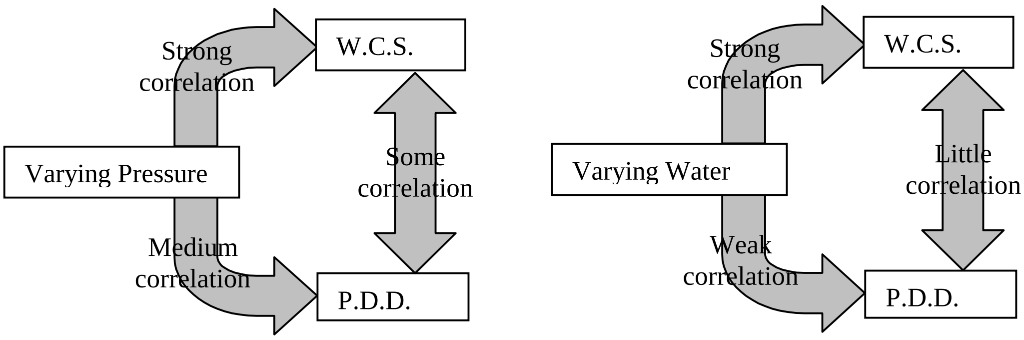
It has been hinted at already that there may be some connection between the achieved density and the 7-day wet compressive strength. The data collected from the third investigation can be presented to demonstrate this phenomenon. Unfortunately the variation in the strength is quite high for each P.D.D., which indicates that if a relationship is developed between strength and P.D.D. it might not be very accurate. Statistical analysis of the variation is required to provide a relationship with any degree of certainty. The figure below shows the general trend that increasing the density has a significant increase in the strength. It indicates that a 5% increase in density yields an increase in strength of approximately 50%. Such a relationship would suggest any small increase in density would be greatly advantageous.

Figure 3.7 – Strength density relationship for soil-B at 6% M.C.



The figure below diagrammatically represents some of the inter-relationships that can now be suggested from the experiments conducted.

Figure 3.8 – Inter-relationships of pressure and water content with outputs



3.3.2 Assessment of de-moulding forces

The third investigation above also provided the opportunity to investigate the forces required ejecting different density samples from a mould. Analysis of de-moulding forces has been omitted from all of the CSSB texts and is investigated here to help with machine design. The forces necessary to eject a full-size block from a mould are often of significant magnitude to warrant a separate ejection mechanism and these tests will indicate possible forces necessary. The small size of these cylinders will have a significant effect on the magnitude of the ejection force compared to a full-size block, but it is hoped that the relative size of the forces for different levels of compression will be representative. This study may also indicate whether or not any difference in the de-moulding force exists between quasi-statically compressed samples and dynamically compacted samples. Data from these squeezed cylinders will be used later with data from impacted cylinders.

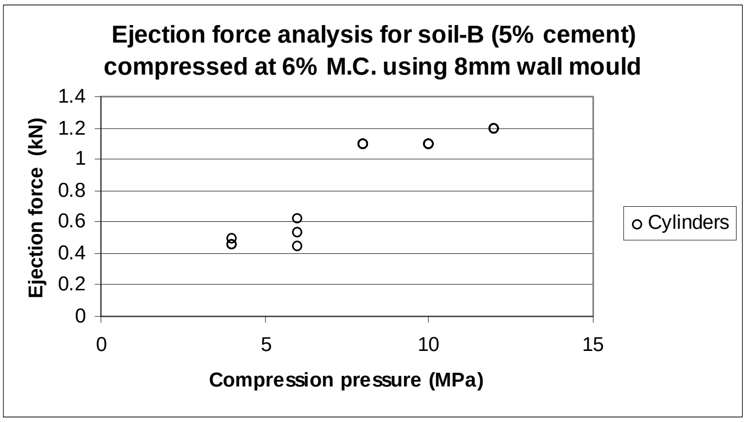
The table below is a summary of the data from the third investigation described in the previous subsection. Three samples were produced at each pressure and the averages of the vales are shown. Full results can be found in Appendix F.

Table 3.2 – Summary of data for compressed small cylinders at different pressures

Pressure	MPa	4	6	8	10	12
Energy Transfer	J	54	70	83	97	111
Ejection force	kN	0.5	0.5	1.1	1.1	1.2
Ejected height	mm	45.8	44.7	44.6	44.1	43.9
Bulk Density	kg/m³	1992	2039	2047	2067	2078
P.D.D.	kg/m³	1879	1924	1931	1950	1960
7-day W.C.S.	MPa	1.31	1.53	1.59	1.87	2.05

The graphical view of the variation in ejection force with compression pressure can be seen in the figure below. It demonstrates that ejection force is roughly proportional to moulding pressures. It should be noted that many of the data points are overlapping each other in the graph, hence less than 15 points are visible. If we assume that the ejection force is a function of both the compression pressure and the mould wall area in contact with the sample, then we may be able to suggest a formula to determine the ejection force required for full-size blocks.

Figure 3.9 – Ejection force analysis for different compression forces



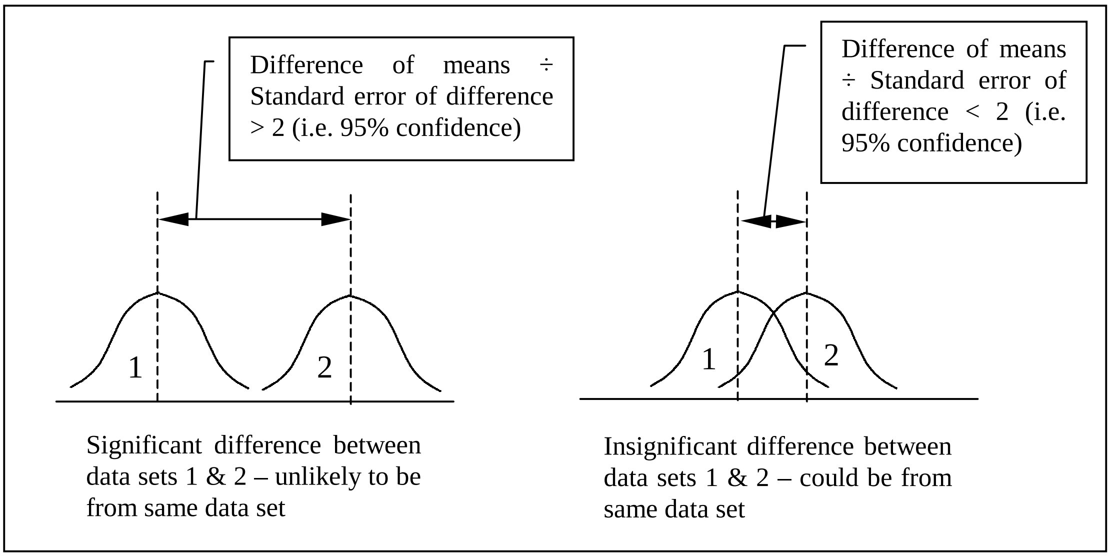
The mould wall area for a small cylinder is 0.0076m². This equates to an ejection shear stress (force per unit area of mould wall area) of approximately 145kPa. A standard block is 0.29 × 0.14 × 0.09m and hence has a mould wall area of 0.0774m². This yields a *projected* ejection force of 12.5kN for a full size block with 6% M.C. compressed to 10MPa. The actual (measured) ejection force for a block compacted to 9.7MPa using soil-B at 6% M.C. was between 15-20kN. These results are of the same order of magnitude and will be useful for machine design.

3.3.3 Strength variation in cylindrical samples

In order to accurately assess interrelationships between variables we need to know the inherent variability of the material in use. Earlier experiments indicated significant variation in the ejected densities of the cylinders produced using quasi-static compression. This density variation has an effect on the wet compressive strength of the material. We do not know what the variation in the strength within the material is for different samples all compressed to similar densities. To discover the actual variation in the material strength and to discover if there is a statistically significant difference between the first of the batch and the third in the batch a further set of small cylinders were produced.

A total of 18 samples were all produced by compressing soil-B with 6% M.C. to 10MPa. The batch size was three, and each of the first, second and third within each batch were averaged together investigating the Projected Dry Density, 7-day Wet Compressive Strength and Ejection Force. If a large difference exists between the first and the third in the batch, a statistical test can be applied to these results to determine whether or not the results could have come from the same data set. If the test shows that they do not come from the same data set then it can be assumed that there is a significant difference in the two sets of data. The diagram below illustrates the statistical theory of this test.

Figure 3.10 – Statistical test for difference between two data sets



The data presented in Appendix F has been analysed below in Table 3.3a, b and c using the standard error of difference test. The test used deems that a significant difference between sets of data exists if the difference between the two sample means is greater than 2 times the standard error of difference (giving a 95% level of confidence for a normal distribution).

Knowing that strength is highly sensitive to changes in density it is not surprising to find that the coefficient of variation of strength is an order of magnitude larger than the coefficient of variation for density. The data also highlights the large variation that is experienced in the ejection force required to eject compressed samples from the mould. This will partly explain the difference between the extrapolated ejection force from small cylinders and the measured ejection force for full-size blocks.

Table 3.3a – P.D.D. variation in cylindrical samples compressed to 10MPa

Order in batch	Units	First	Second	Third
Average of P.D.D. of samples (a)	kg/m³	1950	1938	1934
Estimate of population S.D. from samples	kg/m³	9	13	8
Coefficient of variation	%	0.4	0.7	0.4
Standard error of (a)	kg/m³	3.5	5.2	3.4

Difference of means (1st & 3rd)	kg/m³	15.3	
Standard error of difference (1st & 3rd)	kg/m³	4.9	Sample
DoM/ SED	kg/m³	3.1	size
Significance (Normal distribution)	%	99.8	n = 6 (×3)

Table 3.3b – 7-day W.C.S. variation in cylindrical samples compressed to 10MPa

Order in batch	Units	First	Second	Third
Average of 7-day W.C.S. of samples (a)	MPa	1.76	1.61	1.63
Estimate of population S.D. from samples	MPa	0.09	0.13	0.11
Coefficient of variation	%	5.3	7.8	7.0
Standard error of (a)	MPa	0.04	0.05	0.05

Difference of means (1st & 3rd)	MPa	0.14	
Standard error of difference (1st & 3rd)	MPa	0.06	Sample
DoM/ SED	MPa	2.27	size
Significance (Normal distribution)	%	97.7	n = 6 (×3)

Table 3.3c – Ejection Force variation in cylindrical samples compressed to 10MPa

Order in batch	Units	First	Second	Third
Average of Ejection force of samples (a)	kN	0.95	1.07	1.13
Estimate of population S.D. from samples	kN	0.18	0.14	0.09
Coefficient of variation	%	18.9	13.4	8.0
Standard error of (a)	kN	0.07	0.06	0.04

Difference of means (1st & 3rd)	kN	0.18	
Standard error of difference (1st & 3rd)	kN	0.08	Sample
DoM/ SED	kN	2.17	size
Significance (Normal distribution)	%	97.0	n = 6 (×3)

These results give us the needed information about the inherent variability of the material. Density variation is of the order of 1% throughout a batch, whilst strength variation is around 10%. This demonstrates adequate control of the production process and gives us confidence in making assertions with data sets that differ significantly more than experienced here. A more worrying trend in the above data is every characteristic of interest has a significant difference between the First and Third members of each batch. This clearly suggests that something out of our control is happening to the sample during the 5-10 minutes between the first and the third sample in the batch. Despite the small variation in these characteristics of interest, a larger variation would be more typical during a normal production regime. Whilst a variation of 1% on density during these strict laboratory tests is tolerable if this were to increase to say 5% in the field, a then a much more significant variation of around 50% of strength would result. For further laboratory tests a sample set of three should be sufficient for determining the characteristics of a sample with selected parameters.

3.4 Block characteristics that help to reduce construction costs

In the interests of reducing walling costs, a number of techniques have been assessed particularly with a view to incorporation into block manufacture using dynamic compaction. These subsections detail several methods of cost reduction available to the block manufacturer, some of which can also save costs in wall construction. Our desire is to find a method, or combination of methods, that significantly reduce the total cost of the building unit, which we hope to apply to the production of blocks made via dynamic compaction.

3.4.1 Material reduction

The raw material used in producing a block has a cost associated with it, both in terms of the actual material used and any associated transportation costs. Reducing the raw material will reduce the cost of the block, and placing indentations or perforations in a block can be an effective way of achieving this. In order to remove significant amounts of material from the centre regions of a block there must be sufficient block width to accommodate the voids left behind. Also the minimum material thickness needs to be carefully chosen so that the material does not become too weak to support the necessary loads. The drawback to including any perforations or voids in a block is that it increases the mould complexity and reduces the ease of block manufacture, particularly block ejection.

According the graph in Figure 2.12, a sample with 10% cement and compressed to 4MPa has a wet compressive strength of 3MPa. A standard block with dimensions $0.29 \times 0.14 \times 0.09$m and an approximate bulk density of 2060kg/m^3 would have a cement mass of around 0.7kg. If the compaction pressure was increased to 10MPa, then cement content could drop to 8% and still achieve the same 3MPa compressive strength. The block now has a bulk density of 2160kg/m^3 and would have cement mass of around 0.58kg present in it. A 150% increase in pressure results in only an 18% drop in cement content. This has already been shown to be a false economy in quasi-static compression because this extra moulding pressure seriously increases the machine cost and complexity.

If half of the material present in the block is removed then the cement mass would naturally drop to 0.28kg per block which is less than half the amount required for the block compressed to 4MPa. This material removal could be achieved by the inclusion of voids in the material, an already popular technique. The higher density of the material would yield sufficient strength for forming and handling and whilst the absolute load that the block could sustain would be less, the compressive strength would still be within the required limits. This option would not be possible with blocks of lower densities, as they would not be strong enough to have such large voids placed in them and still be strong enough for forming and handling.

3.4.2 Tall thin blocks

The ratio of a block’s height to width is its’ slenderness ratio (height/width), (Norton, 1997), (Keable, 1996). For most blocks this slenderness ratio is not more than 1 but with some more advanced materials at can be as high as 2. If the height of the block is large then this will reduce the number of blocks necessary to fill the same area of walling. In order to maximise the use of the material therefore we want to have a high slenderness ratio and a large surface area of the external face of the block. Requiring fewer blocks per square meter of walling also reduces the amount of mortar required between block courses. Increasing the slenderness ratio reduces the volume of material required per square meter of walling, whilst only increasing the block height makes reductions in the quantity of mortar that is required. Increased slenderness may be more difficult to achieve with CSSB than increasing the block height, so for the moment we will concentrate on this alone.

Throughout this project we wish to reduce the cement consumption of the walling as much as possible. It is possible to calculate the projected cement requirements of different walling strategies using blocks of different characteristics. One of the characteristics that can be adjusted in this study is the block height. This increases the amount of cement required in the material, mortar and render per block, but actually decreases the overall cement requirement per square metre. Although the decrease was quite small, if also applied to blocks with less cement in the material, laid with thinner mortar and without any render then significant combined savings can be made.

The application of tall thin blocks presents an issue of stability that needs to be considered. We can compare different walling materials and their structural modes of failure to consider the implications of tall thin blocks for walling. If house walls ‘fail’, it is usually by surface erosion, by overturning or by internal material changes like swelling. To prevent erosion we require adequate surface properties such as hardness or wet compressive strength that are unaffected by whether or not the building blocks are hollow. To prevent overturning we look first to architectural measures such as providing adequate foundations, connecting perpendicular walls or constraining the outward thrusts from the roof. However the block properties also affect a wall’s ability to resist horizontal forces applied to its top. Increasing both block mean density (ρ) and wall thickness (Block width b) are beneficial.

Although there are various overturning failure modes, almost all have a force threshold determined by ρb^2. For example the formation of a hinge at the wall bottom (assuming the mortar has no tensile strength) occurs when $F = \rho g b^2/2$ where F is the outward force per unit length of a wall. The table shown below compares different materials by this criteria. Note: employing hollow blocks instead of solid ones lowers F because it lowers the mean block density ρ.

Table 3.4 – Assessing the failure force for different blocks

Material	Wall Thickness (b)	Mean Density (ρ)	Failure Force (F)
	m	kg/m³	N/m
Single skin brick	0.105	1350	74
Double skin brick	0.220	1350	327
Solid cement block	0.150	2200	248
Hollow (50%) cement block	0.150	1100	124
Foamed cement block	0.140	480	47
Low-density solid CSSB	0.140	1700	167
High-density solid CSSB	0.140	2000	196
High-density hollow (30%) CSSB	0.140	1400	137

The above table illustrates why double skin brick and solid cement blocks are most favourable for taller structures as they have the highest failure force. For the purposes of low-rise dwellings this is not necessary and consequently a lower failure force can be accommodated. High-density solid CSSB is a good contender in terms of failure force, but if the walling material were made even thinner then it may not be quite so appropriate.

3.4.3 Cement rich skin

As an alternative to reducing the cement content of the block to low quantities, it may be possible to concentrate the cement in the area where it is needed most, i.e. the exterior surface. This cement rich layer would effectively be acting as a built in layer of render protecting the less stable material behind it from the elements. For example instead of having 5% cement throughout the block one could put 10% cement in the first 20mm and have the rest of the block stabilised with only 3% cement. Providing that the cement rich layer did not suffer from de-lamination from the rest of the block, this could reduce the cement demand for each block. Catastrophic de-lamination is reduced because the block contains cement and the courses of blocks are joined with a cement based mortar.

The production of such blocks with this cement rich layer greatly increases the complexity of the block production and construction process. A clear means of identification would be necessary to indicate which face of the block was cement rich, and furthermore the staff erecting the structure would need to be trained to lay the blocks in the correct manner. Homogenous blocks would also be necessary for the corners and any exposed edges, adding another type of building material to the construction. The calculations carried out on this type of construction shows that the saving in cement is only a modest 13%.

3.4.4 Summary of cost reduction methods

The table below summarises the different possible variants that can be accomplished with the CSSB and how each one performs with reference to the unmodified CSSB. By combining several of these variants into a single block the material can theoretically achieve a tolerable cement requirement, (less than 15kg/m^2), without excessive energy consumption. The tall, hollow, interlocking block as described below even uses less cement then the clamp fired bricks assessed in Table 2.1. As this is one of the more common and more wasteful methods of making satisfactory building materials, this confirms that this variant of CSSB is a real contender. The raw data for this comparison can be found in Appendix E.

Table 3.5 – Theoretical comparison of different CSSB variants

Material	**Dimensions** ($l \times b \times h$)	**Note**	**Energy**	**Cement**	**Suitability for production**	
					'Locally'	**On-site**
High-density CSSB	Mm		MJ/m^2	kg/m^2	Ranking	(1 = best)
Normal	290 × 140 × 90	1	290	18.7	2	1
Hollow	290 × 140 × 90	2	220	15.1	2	1
Cement-rich skin	290 × 140 × 90	3	270	16.3	1	2
Interlocking	297 × 140 × 97	4	270	15.4	2	1
Tall	290 × 140 × 90	5	280	17.6	2	1
Rendered	290 × 140 × 140	6	300	19.3	2	1
Tall, Hollow, Interlocking	297 × 140 × 147	7	190	11.0	2	1

Notes

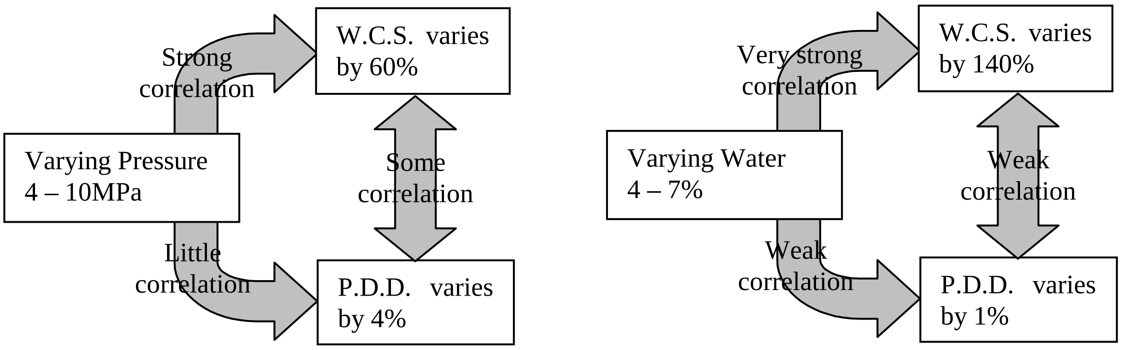
1. High-density (2000kg/m^3) solid blocks manufactured on-site from local soil/cement mix (5% cement), laid with 10 mm of soil/cement mortar (20% cement) and no render, (Cement transported 100km).
2. As 1. but with 30% material remove from the block core.
3. As 1. but with 10% cement in first 20mm of exterior block surface and 3% in the body of the block.
4. As 1. but constructed with thin mortar only 3mm thick.
5. As 1. but with increased block height to 140mm to reduce mortar per square metre.
6. As 1. but with 15mm render on a block with only 3% cement in the body of the block.
7. As 1. but with a combination of tall, hollow and interlocking arrangements.

Apart from the improvements that can be offered by increased material compaction, there are modifications that can be made to the shape and size of the CSSB to minimise material costs. The addition of perforations in the block could reduce the material cost by as much as 30%. Furthermore the improvement of dimensional tolerances of the block could promote the use of thinner mortar between block courses. The application of taller blocks would also reduce the number of courses that need mortaring. A combination of these block features has indicated significant saving in cement and shows the most promise for providing lower-cost walling.

3.5 Chapter summary

Throughout this chapter experiments on stabilised soil have been conducted to assist the understanding of CSSB characteristics and production. The variables that exist in the process have been identified and possible relationships that may exist between variables and output measures of interest have been suggested. Experimentation had indicated that the moisture content of the sample has a large effect on the achieved density but also on the achieved strength. The strength seems to be directly related to the density achieved for given moisture content. The general trend that an increase in density of 10% yields a 100% increase in strength has been suggested.

Figure 3.11 – Revised inter-relationships between pressure and water with outputs



Investigations on small cylinders have given an improved understanding of the pressure-density relationship, which will be very useful for comparative tests on dynamic compaction. The aim during future tests will be to achieve a projected dry density of at least 1950kg/m³ as this is representative of compression to 10MPa. These tests have also indicated the ejection forces required to de-mould a sample and that it is roughly related to the compaction pressure and mould wall area. It will be interesting to see if the process of dynamic compaction yields significantly different values for the ejection force at similar achieved sample densities.

A more concerning discovery has been the significant variation in the achieved density and strength of samples produced in the same batch a short time apart. The variation in density is only around 1% but this in turn results in a 10% variation in strength. It would be excellent if variation during regular block manufacture could be kept as low as this; however, this is highly unlikely because of the more strict production methods using during research. It does indicate that for the sample size selected, the variation achieved is on the lower limit of practical significance and therefore the sample size of three is acceptable for future tests. The reason for this variation has yet to be determined; it may be linked again to moisture, as it seems unlikely that the cement would be providing any resistance to the consolidation process after such a short period of time. For research purposes full-size block production typically has a batch size of 1 block per batch so this problem should not plague later tests, but it will need to be considered during future batch tests and for block production generally.

After conducing the experiments described in this chapter we feel more confident about working with the soil in question. We have gained a better understanding of its performance under compression and its characteristics at different achieved densities. Armed with this knowledge we can now proceed to investigate the application of *dynamic* compaction and assess its performance against quasi-static.

4 Dynamic compaction of stabilised soil

The experiments described in the previous chapter concentrated on tests using stabilised soil. These tests have enhanced our understanding of how soil (specifically soil-B) behaves during constrained consolidation. This chapter extends the investigation to include dynamic compaction of this soil, initially as small cylinders and then as full-size blocks. This will be a lengthy chapter, as the majority of the experimental results from the Ph.D. will be presented here. After briefly discussing the reasons for selected methods of experimental practice and outlining some new variables of specific interest to dynamic compaction, the relationships that exist within dynamic compaction will be presented and explored. This will then be followed by results taken from full-size tests and finally a comparison of the dynamic compaction process with the quasi-static compression process will be made.

4.1 Experimental design

4.1.1 Sample size selection

The experiment conducted earlier on small cylinders, quasi-statically compressing them to 10MPa (described in subsection 3.3.3), indicates that for a sample set of six (the first member of a batch of three taken from six separate batches) the coefficient of variation is around 0.5% of the density. This variation in turn results in a variation of 5% in strength, which is a tolerable variability for experimental purposes. It demonstrates quite a high degree of repeatability within the material and the process of production used. The variability test will need to be repeated for the new production method of dynamic compaction of both small cylinders and full-size blocks to confirm a similar variation. As discussed below, the variation was similar and a sample size of three was selected for small cylinder production. The sample size necessary for full-size block production will be examined experimentally as well.

The variability of the dynamic compaction process for small cylinders was investigated by making 18 cylinders of soil-B stabilised with 5% cement. Each cylinder received 16 blows from a 5kg impactor being dropped from 0.2m. The cylinders were produced with a batch size of three and the respective first, second and third members of each batch formed 3 samples each of six members. The ejection force was measured, the P.D.D. of each cylinder was calculated upon ejection and the 7-day wet compressive strength was measured for each cylinder. The results in the tables 4.1a, b and c below show the variation experienced with this method of production for three output measures.

Table 4.1a – P.D.D. variation in dynamically compacted cylindrical samples

Order in batch	Units	First	Second	Third
Average of P.D.D. of samples (a)	kg/m^3	2022	2010	1998
Estimate of population S.D. from samples	kg/m^3	9	5	5
Coefficient of variation	%	0.5	0.3	0.2
Standard error of (a)	kg/m^3	3.80	2.10	1.95

Difference of means (1st & 3rd)	kg/m^3	23.50	
Standard error of difference (1st & 3rd)	kg/m^3	4.28	Sample
DoM/ SED	kg/m^3	5.49	size
Significance (Normal distribution)	%	>99.9%	n = 6 (×3)

Conclusion: There is a statistically significant drop in density of 1.1% between the First and Third members of each batch.

Table 4.1b – 7-day W.C.S. variation in dynamically compacted cylindrical samples

Order in batch	Units	First	Second	Third
Average of 7-day W.C.S. of samples (a)	MPa	2.38	2.23	2.12
Estimate of population S.D. from samples	MPa	0.14	0.15	0.18
Coefficient of variation	%	6.1	6.5	8.3
Standard error of (a)	MPa	0.06	0.06	0.07

Difference of means (1st & 3rd)	MPa	0.25	
Standard error of difference (1st & 3rd)	MPa	0.09	Sample
DoM/ SED	MPa	2.72	size
Significance (Normal distribution)	%	99.3	n = 6 (×3)

Conclusion: There is a statistically significant drop in W.C.S. of 11% between the First and Third members of each batch.

Table 4.1c – Ejection force variation in dynamically compacted cylindrical samples

Order in batch	Units	First	Second	Third
Average of Ejection force of samples (a)	kN	1.33	1.35	1.36
Estimate of population S.D. from samples	kN	0.15	0.10	0.12
Coefficient of variation	%	11.6	7.4	9.0
Standard error of (a)	kN	0.06	0.04	0.05

Difference of means (1st & 3rd)	kN	0.03	
Standard error of difference (1st & 3rd)	kN	0.08	Sample
DoM/ SED	kN	0.34	size
Significance (Normal distribution)	%	26.6	n = 6 (×3)

Conclusion: There is no statistically significant difference in ejection force between the First and Third members of each batch.

From the results above it is now possible to suggest that the process of dynamic compaction does not add any further variation to the P.D.D. or the 7-day W.C.S. of small cylinders than did the quasi-static compression process. A variation of about 1% in density across the batch still exists and approximately 10% variation in 7-day

W.C.S. still applies. Greater comparison between the dynamic compaction method and the quasi-static method will be done later in this chapter.

For practical reasons we wished to make several (*n*) specimens from each batch. Increasing *n* will reduce the variance of the sample mean about the population mean, which is good. Unfortunately it will also introduce a bias. If we choose a sample size of *n*, we reduce the Coefficient of variation of our estimate of the population mean by a factor of $\sqrt{n}$, which is good. Unfortunately to get *n* samples from a single batch entails the passage of time, so that the last member of the batch has a longer time delay before compaction than the first member. This variation in production time will therefore introduce a new source of variation in P.D.D. We hoped that 3 would be a sufficient sample size n. From the table above we see that:

(a) with $n = 3$, the Coefficient of variation is $< 0.5 \div \sqrt{3} = 0.3\%$.

(b) with $n = 3$, due to increased production time, the average will be biased downwards by typically 0.5%, (varying with the speed of production)

Such a small variation is at the lower limit of practical significance and consequently we can continue to use a sample size of three, $n = 3$.

This analysis confirms that using a sample size of three would be acceptable experimental design for investigation of small cylindrical samples, but no assumptions can be made with full-size blocks as yet. In order to check the variability of producing full-size blocks, a set of five blocks was produced by dropping a 36.8kg impactor approximately 300mm onto the surface of soil-B (0% cement). Only eight blows were applied during the production of each block, resulting in a relatively low P.D.D. The results in the table below show the average of the measured block height and the calculated P.D.D. for five blocks. It clearly indicates that the variation of the process of dynamic compaction is still of the order of 0.5%. As these blocks were not made with cement it is not possible to determine the variation of the 7-day W.C.S., but this is assumed to have the same relationship with density as seen before.

Table 4.2 – Variation in P.D.D. for full-size blocks

Block Number	No. of blows	Block Height mm	Block P.D.D. kg/m³
1	8	113.4	1738
2	8	112.7	1748
3	8	113.6	1735
4	8	112.6	1750
5	8	112.2	1756
Standard Deviation			9
Coefficient of variation			0.5%
Coefficient of variation of mean of 3			0.3%

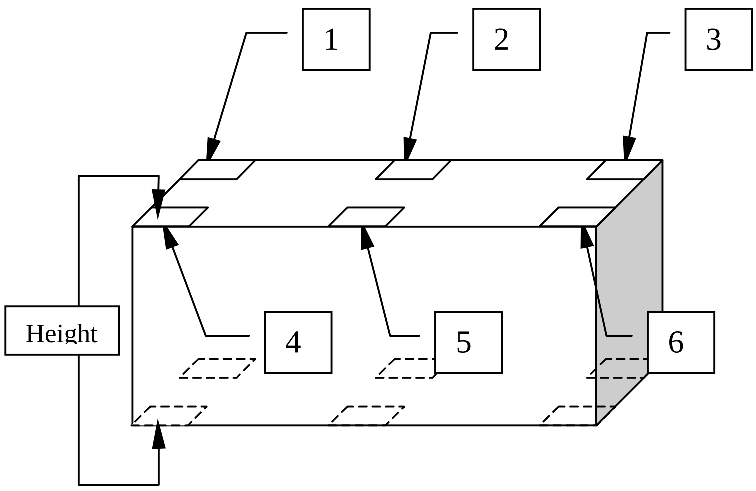
From these tests it is now possible to say with greater assurance that the inherent variation of the consolidation of soil-B results in a variation of less than 1% of P.D.D. and less than 10% of 7-day wet compressive strength. Consequently experimentation can now begin to look for characteristics within dynamic compaction that yield changes in results greater than normal variation. These changes will give indications to relationships between input variables and help to improve our understanding of the process.

4.1.2 Primary measure of performance is P.D.D.

In the majority of the tests presented in the following sections, the Projected Dry Density (P.D.D.) has been used as a primary measure of performance to monitor sample and block production. This measure has already been indicated as a suitable measure for compaction and possible strength prediction. The P.D.D. of the compacted material is used to monitor the relative levels of compaction between samples or blocks produced using different production variables. This immediate measure has been very useful in determining the dominant variables without having to wait for feedback data from the wet compressive strength test.

It should be noted at this time that the method of measuring the samples needs to take into account the possible variation in the shape of the sample. Dynamically compacted small cylinders typically exhibited a small slope on their top surface. The cylinder diameter remained constant throughout the tests but the height had to be averaged from the highest and lowest point on the cylinder circumference. The variation in height was usually only between ±0.3 - ±0.6mm, but this is significant in a such a small sample. Full-size blocks exhibited this same phenomenon and consequently the block height had to be measured at six points on the surface of the block.

Figure 4.1 – Six block height measuring points



4.2 Variables of dynamic compaction

The process of dynamic compaction presents not only new challenges in the method of application but also new variables that will need to be assessed for significance and subsequently optimised. The impactor mass, drop height and number of blows have all been investigated before but these parameters need to be extended to cover both small cylinder and full-size block production.

Impactor mass **–** Previous research conducted on dynamic compaction of soil indicated that a larger impactor mass was generally better than a smaller one. The limitations therefore imposed on the impactor design are ones of practicality, safety and cost. For the small cylinder production the following range of impactor masses were used: 2.5kg, 5kg and 10kg. However for full-size block production a range of larger impactor masses was used during the tests including 36.8kg, 46.8kg and 60.0kg.

Drop height **–** Apart from the practical limitations and safety issues of lifting a large heavy mass through a large height, there is evidence to suggest that a large drop height is undesirable for confined soil compaction (see subsection 2.4.2.) This was seen with samples produced by the application of one or two blows with high momentum. The effect of this high momentum transfer apparently resulted in a shock wave rebounding off the foundation and shattering the sample. It was suggested (Montgomery, 1997) that impactor velocities of over 2m/s should be avoided for this reason. Consequently the drop height was initially set to 200mm, but this was later increased up to 400mm (equivalent to 2.8m/s) without any adverse effects being noted.

Number of blows – After the imposed limitations of impactor mass and drop height, the number of necessary blows comes down to a trade-off between energy transfer and production time. Energy transfer is necessary to achieve the required consolidation, but the application of a large number of blows is time consuming. Previous research indicated that an optimum was between 8 and 32 blows. It was hoped that after initial trials at large numbers of blows a bit of balancing between drop height and the impactor mass this number could be reduced to less than 16.

From the variables listed above it is possible to state one further variable that is of interest to us, energy transfer. Calculation of the energy transfer using dynamic compaction is a trivial exercise involving the impactor mass, drop height and the number of blows. The total energy transferred into a sample takes the form of the following: $E_T = mg\sum_{i=1}^{n} h_i$ (1)

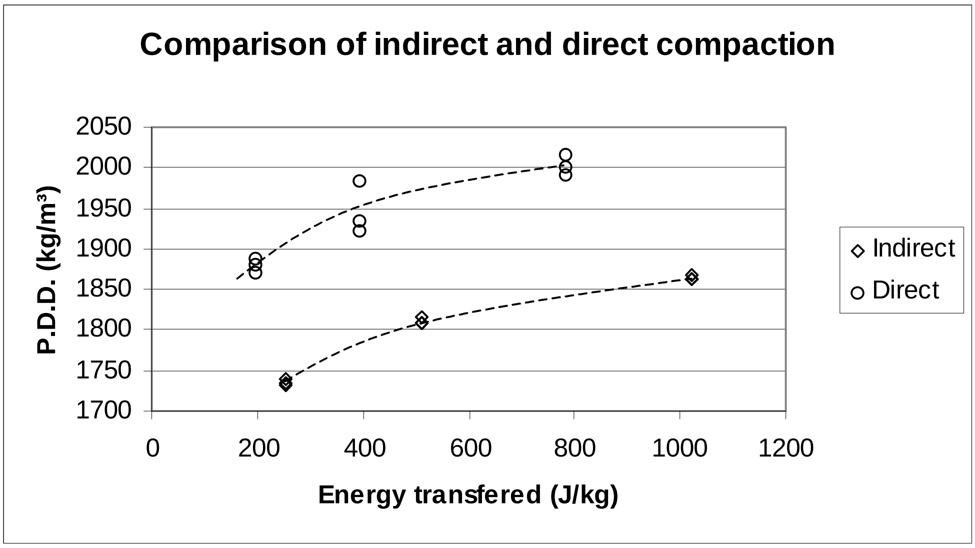
Where m is the impactor mass, g is the universal gravitational constant (9.81), h_i is the drop height for the i^{th} blow and n is the number of blows applied. If the point from which the mass is dropped is fixed relative to the foundations then the actual distance the impactor falls will be dependent on the blow number. Later blows will have a larger drop height than earlier blows, because of the significant consolidation that is achieved. This variation in the drop height will be considered and included in calculations where appropriate and experimentally possible.

The following subsections deal with some more interesting and significant aspects of dynamic compaction that would be of great interest to the machine designer. Aspects such as direct and indirect impact, mould wall thickness and soundness of the machine foundation have been explored and reported here. These are other variables to the production process that are kept constant once their significance has been determined and a suitable parameter selected.

4.2.1 Indirect and direct impact

It would simplify machine design if a billet could be placed between a falling impactor and the block it is compacting (‘indirect’ blows). However, early on in the Ph.D. it was noted that there is a noticable difference between samples compacted through direct and indirect blows. Experiments conducted on small cylinders indicated that the use of an intermediary billet of steel (mass of 2.5kg) between the sample surface and the falling impactor yielded significantly lower levels of compaction. Data from two sets of experiments is presented below in Figure 4.2. It clearly shows that indirect compaction produces much less compaction for the same energy transfer. All the samples were compacted using a 2.5kg impactor falling through either 0.2m (direct) or 0.26m (indirect) onto 200g samples with 6% moisture. Either 8, 16 or 32 blows were applied and the total energy transfer was calculated and converted to energy per unit mass. The graph indicates that the two methods of compaction differ by almost 10% on density, constituting a practically significant difference between the two methods. These results suggest that up to 50% of the strength could be lost if indirect compaction was chosen.

Figure 4.2 – Analysis of direct and indirect compaction



The numerical results for the above experiments can be found in Appendix F. From these results it was concluded that the additional complexities of direct impact compaction were justifiable given the significant improvement in achieved consolidation. Consequently all future experiments were conducted using the direct impact method.

4.2.2 Foundational effects

The process of dynamic compaction relies on the availability of a firm surface onto which compaction can occur. The firmness of the foundation affects the effectiveness of the dynamic blow applied simply by virtue of energy dissipation. A firmer foundation will not yield as much under a dynamic blow and will therefore permit greater compaction energy to be transferred into the sample. A single experiment was conducted to determine the penalty of soft foundations on the block density. Softer foundations were produced by having the full-size block mould placed on top of a 20mm steel plate separated from the 100 tonne strong floor by washers in each corner.

The firmer foundation was achieved by applying a layer of dental paste between the mould and the strong floor to ensure maximum surface area for contact and therefore greatest strength.

It was no surprise that the block compacted on the soft foundation performed worse than the block compacted on the firm foundation. The significance of the stiffness of the foundation was high, but not as high as expected. Two samples compacted with a 46.8kg impactor falling through approximately 200mm for 24 blows yielded densities of 1739 and 1811kg/m^3 for the soft and firm foundation respectively. This represents a variation of about 4% suggesting that the different foundations have a significant effect on the level of consolidation. It was assumed that larger drop heights would further reduce the potential density and consequently the firm foundation was selected as the best option for experimental research.

4.2.3 Delay between impacts

Another variable that dynamic compaction offers is the duration between consecutive blows applied to the sample. As yet it is not fully known what exactly is happening during material consolidation and even less is known about impact compaction mechanisms. This gap in our understanding led us to conduct a test to determine whether or not a time delay between consecutive blows has a significant effect on the level of consolidation achieved. The experiment would also help to explain some of the dominant mechanisms acting during impact compaction.

The variability within such an experiment was deemed to be higher than normal and consequently to have a greater confidence in the achieved data five blocks were made for each arrangement as opposed to one or three. The first arrangement involved dropping a 36.8kg impactor through 300mm directly onto the surface of the material at the rate of one blow per minute to give a total of eight blows. The second arrangement applied the same type of blow only with the blow rate set to one blow every two seconds. It was hoped that the thirty-fold reduction in compaction time would indicate if any difference existed between the two samples above and beyond the inherent variation of the samples themselves. In order to ensure than any cementitous action did not interfere with the experiment soil-B was used without any cement.

Although the densities of the blocks are quite low it is interesting to see that this time delay has an effect on the level of compaction achieved. Blocks produced fast yielded densities between 1735 and 1756kg/m^3, whilst blocks with the 1 minute delay produced blocks with densities between 1764 and 1769kg/m^3. Not only are the densities higher but they are also in a smaller range, indicating a higher degree of repeatability.

In order to prove the significance of these results a series of statistical tests were performed on the data. A summary of these tests and their results are shown in the table below.

Table 4.3 – Statistical variation in full-size block production

Blow Type	Number of samples	Average of dry density	S.D. of dry Density	Coeff of Variation of density	Standard Error
2sec/blow	5	1745	9	0.52%	4.05
1min/blow	5	1766	2	0.12%	0.92

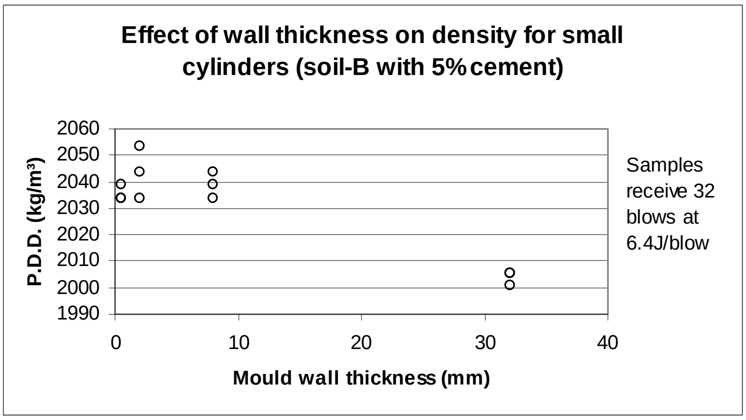
Applying the Standard error difference test to this data yields a Difference of Means ÷ Standard error difference = 5.00 which equates to a statistical significance of near unity. Whether or not the process of delaying the time between blows has a practically significant effect on the density achieved is a different matter. The improvement yielded from delaying compaction only equates to an increase of 1.2% in the density achieved. A variation of ±0.5% can be considered to be unimportant which suggests that this finding would only just be considered important and therefore incorporated into production. However, the requirement of a delay between impacts would increase production time to unacceptable levels and consequently fast compaction has been used throughout the experiments. This data suggests a further mechanism is at work during dynamic compaction and indicates what might be happening during the impact blow. This area will be discussed in more depth in chapter 5.

4.2.4 Mould wall thickness

The thickness of the walls of the mould needed for dynamic compaction is of greater concern to the machine designer than to the block producer. Clearly the use of thinner moulds is financially attractive, as they require less material for production and easier methods for mould fabrication. In order to determine if there exists any difference between different mould wall thickness a set of different moulds was created for some tests.

A set of twelve samples was produced by indirect dynamic compaction. The intermediary billet was the same mass as the impactor to try and maximise the momentum transfer. This was done mainly for practical reasons, as it would have been very difficult to ensure an accurate enough free fall for an impactor to go into a mould with 0.5mm wall thickness. The sacrifice made in achievable density is a tolerable quantity for this type of test. The samples were produced with a 2.5kg impactor falling though 0.26m delivering approximately 6.4J per blow. A total of 32 blows were applied to each sample and the projected dry density achieved during compaction was in the region of 2000-2055kg/m³. The graph in the figure below shows the density achieved by the samples relative to the thickness of the mould wall. It is encouraging to see that the very thick walls of 32mm do not provide the highest levels of compaction. It is also good that the other three moulds used are clustered together at the higher density indicating a low sensitivity over the range of smaller mould walls.

Figure 4.3 – Mould wall thickness experiment results



During the experiment the strain on the walls of the 0.5mm and 2mm wall thickness mould was measured. It was hoped that the level of compaction achieved using the

0.5mm wall mould would cause yield in the steel and therefore test the mould to failure. Interestingly the maximum strain experienced by either mould was only 40 micro-strain, a small fraction of the typical 1200 micro-strain that it takes for steel to reach its yield point. This is very encouraging, as it means that the forces sustained on the mould during dynamic compaction are only a small faction of the forces applied during quasi-static compression. The ramifications of this finding will be discussed in greater depth later in the thesis.

4.3 Small cylinder production via dynamic compaction

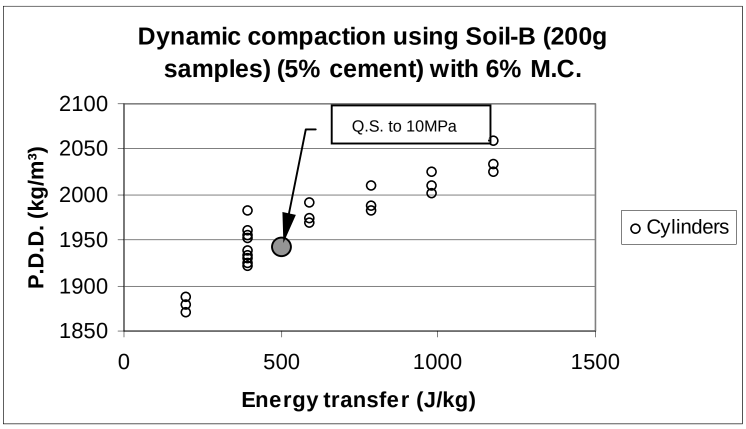
Further production of small cylinders is necessary to clarify the effect of new variables on the output measures. These tests have already been done via quasi-static compression, so for completeness they should be repeated at the same scale and with the same parameters using dynamic compaction. We are also aiming to achieve particular material characteristics (i.e. adequate material consolidation and compressive strength) and further tests applying different energy and momentum should indicate the relative performance of dynamic compaction more persuasively. The general aim is to achieve the same equivalent density as 10MPa would achieve (around 1950kg/m^3) and also to maximise the 7-day wet compressive strength (ideally over 2MPa). The results that these tests give us will also dictate some of the tests carried out on the full size blocks in the next section.

4.3.1 Energy and momentum transfer

In order to get the desired density several different combinations of energy and momentum transfer were explored. A variety of impactor masses, drop heights and numbers of blows were used to manufacture samples in batches of three. All of these samples had a constant material (soil-B) and moisture content (6%).

The graph below shows the results of a series of different tests using a 2.5 or 5kg impactor falling through 0.2m a number of times. The number of blows applied included 8, 12, 16, 20 and 24 and the P.D.D. was calculated from the ejected sample height. It can be seen from this data that the energy transferred into the sample has a direct effect on the P.D.D. However, it is also apparent that a larger amount of energy per kg is being applied at this scale compared to quasi-statically compressed full-size blocks. (Gooding estimated that 280J/kg was equal to the energy consumed during quasi-static compression of a soil block to 10MPa.) This data shows that 400J/kg is necessary to *dynamically* compact the material to 1950kg/m³ at this scale. However this is less than the 500J/kg required for *quasi-static* compression at the same scale.

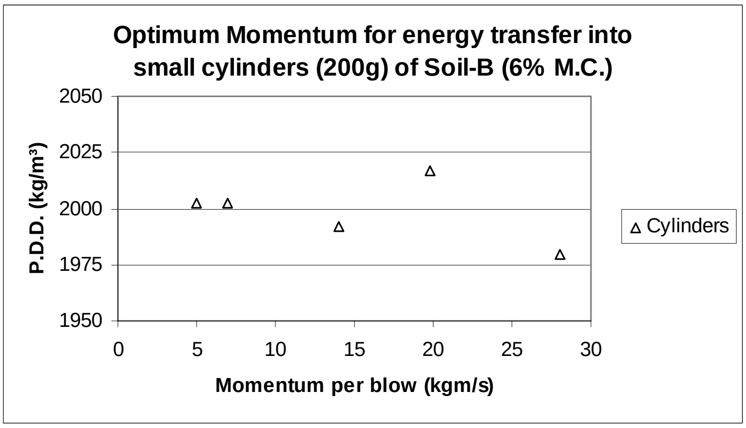
Figure 4.4 – Energy density relationship for small cylinders of soil-B



The data around the 400J/kg area indicates that for the same energy transfer a significant range of densities can result. This is because the relationship between energy transfer and density is not directly proportional. The same amount of energy can be applied to a sample in different ways, many of which will be far from the optimum configuration. The optimum momentum transfer was investigated by Gooding and determined that a low-velocity high-momentum blow is more effective. We can clarify this assertion at this scale by investigating a range of different momentum arrangements.

Another set of samples was produced using the combination of 2.5, 5 and 10kg impactors, 100, 200 and 400mm drop heights and 4, 8, 16 and 32 blows. Five combinations of these variables yield different momentum transfers for each blow applied, yet the same total energy. These results can be seen in the figure below. The data is presented more clearly by observing the averages of the P.D.D. from each sample set. The graph does not indicate a definite optimum as expected, but it does indicate that the graph is relatively flat (variation ±1%) over the region of interest.

Figure 4.5 – Optimisation of momentum transfer for small cylinders

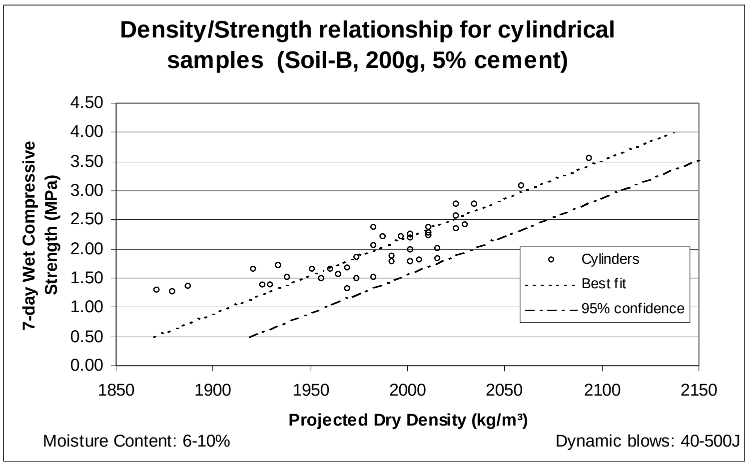


We want to be able to use this information to suggest a suitable momentum transfer for full-size blocks. But these results do not present any clear guidance and extrapolation of the data without subsequent confirmation is unacceptable. Therefore the impactor mass, drop height and number of blows used for full-size blocks will have to be determined by other factors such as practicality, cost and speed.

4.3.2 Strength vs. density

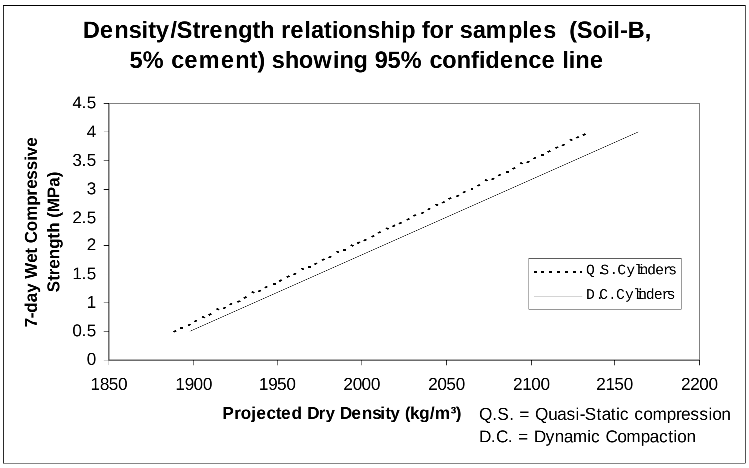
Previous experiments indicated that there is an empirical relationship between the achieved density of the sample and the strength that the sample achieves. For the limited condition of using soil-B mixed with 5% cement it is possible to define a relationship between strength and density. The graph below is a summary of cylindrical samples produced by dynamic compaction for different moisture contents and energy transfers. It indicates a linear relationship over the range of interest. We can propose the following relationship between density and strength with a 95% confidence: $\sigma_{7-day-wet} = \frac{\rho_{P.D.D.} - 1880}{77 \times 10^{-6}}$. Therefore we can say with 95% confidence that a sample with a projected dry density of 2000kg/m³ will have 7-day wet compressive strength of more than 1.6MPa. (The accuracy of this relationship is probably only limited to the range of data used to create it.) For the purposes of this investigation it shows the region of greatest interest, samples that exhibit wet compressive strengths between 1.5 and 3MPa, considered “Good” to “Excellent” by the CSSB literature.

Figure 4.6 – Density strength relationship for dynamically compacted cylinders



If the data from these small cylinders is added to the data received from the quasi-statically compressed samples then we have a bigger data set for analysis. Rather than present all the raw data together, we will only display the calculated 95% confidence lines from the cylindrical samples produced by dynamic compaction and quasi-static compression. These two lines are plotted on the graph in the figure below and it is clear that they are very similar. This gives more weight to the proposal that the strength can be calculated from the known density, a very attractive finding.

Figure 4.7 – 95% Confidence lines for density/strength relationship for cylinders



This data gives us a benchmark for the production of full-size blocks. We would aim to produce blocks with similar densities and see if their 7-day strengths lie in the same region. If they do, then assessment of the material characteristics of small cylinders can be assumed to be transferable to full-size blocks with a degree of confidence. The next section commences the extension of experimental investigation to include the production of full-size dynamically compacted stabilised soil blocks.

4.4 Full-size block production via dynamic compaction

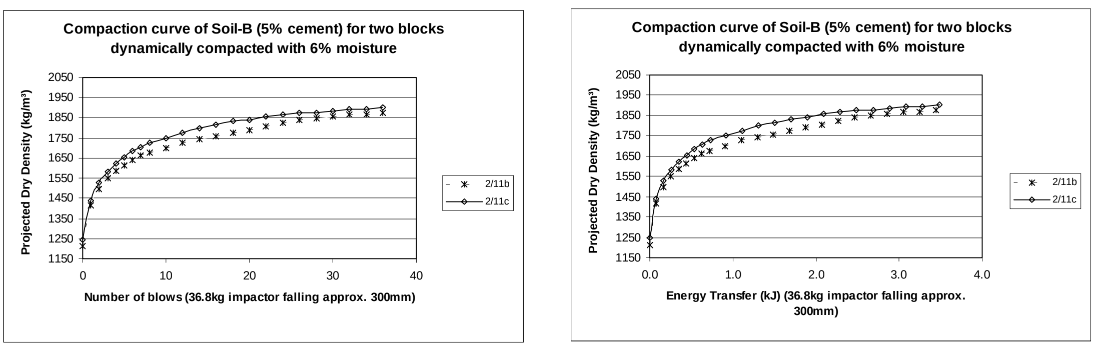
A total of 22 full-size blocks were produced using soil-B by dynamic compaction, four of them were compacted without cement. Different moisture contents were investigated, the compaction curve for the dynamic process was also recorded, and the finished blocks were cut into 100mm cubes for compression testing after 7-day curing. There are two main motivations for the development of a dynamic compaction rig capable of producing full-size blocks. The first is to continue research into the production of full-size blocks, as confirmed possible by (Montgomery, 1997), and the second is to advance the development of a suitable machine for block making. Chapter 6 will discuss the Test Rig design in more depth, but the results of the block production generated from the Test Rig will be presented in this section.

4.4.1 Compaction curve for dynamic compaction

In order to maximise the data collected from the production of each block, the block height was measured remotely after every, or every other, blow. These results enable us to plot a compaction curve for each block, monitoring the P.D.D. as well as the energy necessary to achieve it. The advantage of this system is that it offers a large number of data points for analysis that permits density estimations from known energy transfers. This information will be useful in comparing the results both between full-size and cylindrical samples and between dynamically compacted and quasi-statically compressed samples.

The remote measurement method uses a ruler guide on the impactor, so that after each blow the relative position of the impactor can be measured to ±0.5mm. Once the block is compacted, ejected and measured then this relative measurement can be used to calculate the in-situ block height during the compaction sequence. Such a compaction curve can be see in the figure below. Two blocks labelled “2/11b” and “2/11c” have received 36 blows from a 36.8kg impactor falling through approximately 300mm. The graph clearly shows the similarity between two blocks compacted by the same method. They do not follow exactly the same compaction curve, but that is expected, as there is a small degree of variation in the process. The graph on the left shows the density against the number of blows applied, whilst the graph on the right shows the density against the energy transferred.

Figure 4.8 – Compaction curves for blocks at 6% moisture

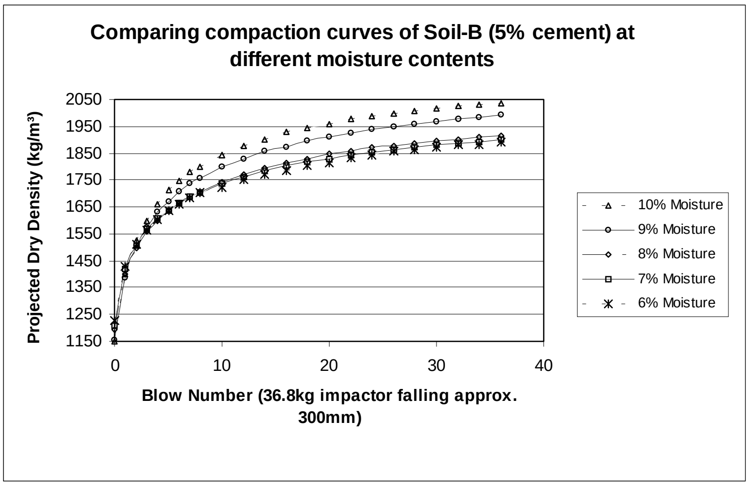


A disappointing outcome of these results is that the final P.D.D. of the blocks did not quite get to 1950kg/m^3 as was hoped. Furthermore, the quantity of energy transferred into the block (3.5kJ) was much higher than the equivalent energy to compress a similar block to 10MPa, (280J/kg × 8kg = 2.2kJ). This seems to contradict Gooding's findings of dynamic being more effective than quasi-static compression. Before we jump to any conclusions it would be good to investigate other moisture contents and other impactor arrangements to see if a comparable block can be made by this method using similar amounts of energy.

4.4.2 Different moisture contents

Experiments at small scale indicated that the moisture content has a significant effect on the P.D.D. for the same energy transfer, therefore the investigation of other moisture contents may yield more effective compaction. A total of thirteen blocks were produced each receiving 36 blows from a 36.8kg impactor falling approximately 300mm. Five different moisture contents, 6, 7, 8, 9 and 10% were explored and two or three blocks were made at each moisture content. These tests also provide significantly more data to help determine the inherent variation of the process. The graph below presents the average compaction curve for each moisture used. It is clear that the moisture content has a significant effect on the effectiveness of the compaction, as an increase in moisture content from 6 to 10% increases the density by about 8%.

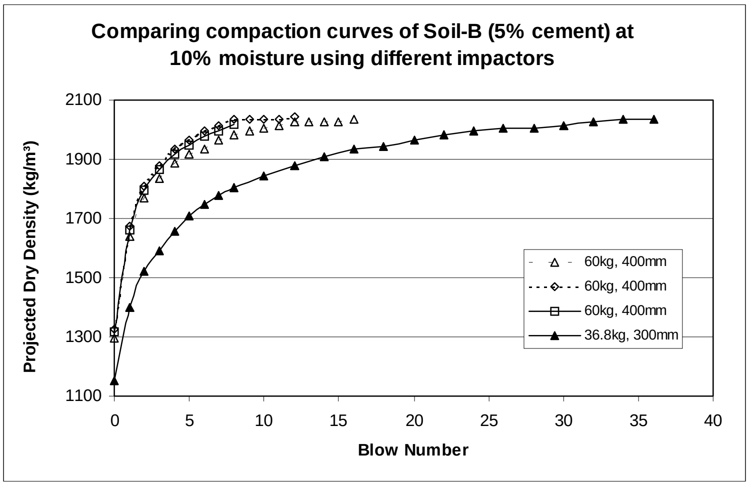
Figure 4.9 – Compaction curves of blocks at different moisture contents

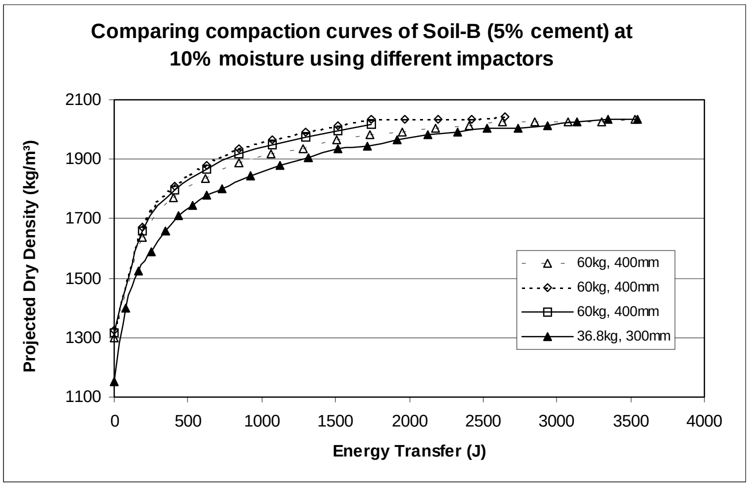


The differences between the curves on the graph are quite distinct confirming our expectations that increasing the water content would yield higher levels of consolidation. However, our motivation for selecting 6% was to achieve good handling characteristics of the finished block. Pushing the moisture content up to 10% reduces this handling strength, but fortunately this was offset due to the increased achieved density and did not present a problem.

It can be seen that the 10% moisture content line crosses the 1950kg/m³ line after about 18 blows. This lower number of blows is much more attractive as it takes less time to apply. What now needs to be determined is whether or not a block can be produced using perhaps a heavier impactor lifted through a slightly larger distance to achieve 1950kg/m³ with a more tolerable 16 blows or less. The graph in the figure below shows the compaction curve for a set of blocks compacted with a 60kg impactor falling from 400mm. Only a single block for each is used to display the compaction curve against the blow number or the total energy transferred. The top graph shows the different rates of compaction as each blow is applied, illustrating the significant difference between different impactors and drop heights. Whilst the lower graph shows the total energy transfer by either impactor arrangement and shows that the higher mass and greater lift height only slightly improves the compaction effectiveness (i.e. consolidation per unit energy transferred).

Figure 4.10 – Compaction curves for different impactors





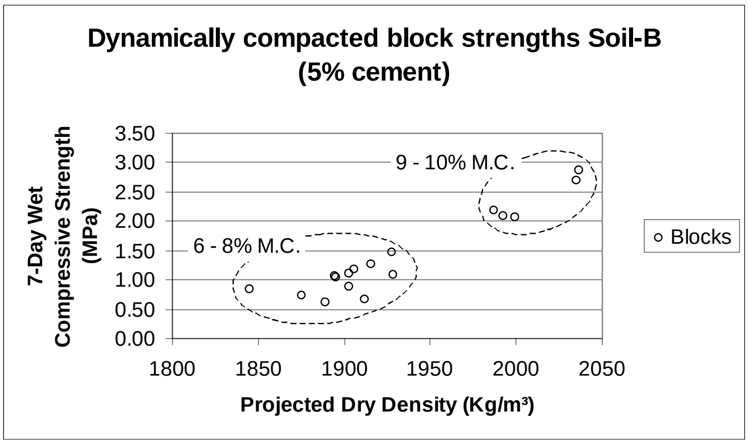
It is pleasing to see that the level of density achieved by these methods is well above the desired 1950kg/m³. It is even more encouraging that a small improvement in effectiveness is achieved if a larger impactor is dropped from a greater height. Our original concern about raising the impactor height was not found to be justified, as the ejected blocks showed no signs of shattering or de-lamination from the increased velocity impacts.

4.4.3 Block characteristics

Being able to achieve a block density of over 1950kg/m³ is only part of the necessary requirements for adequate block production. We already believe that such a density will give a material compressive strength that is enviable among CSSB, but we need to establish the actual strength of these blocks. Other characteristics of the material and the production effects will also be explored in this subsection.

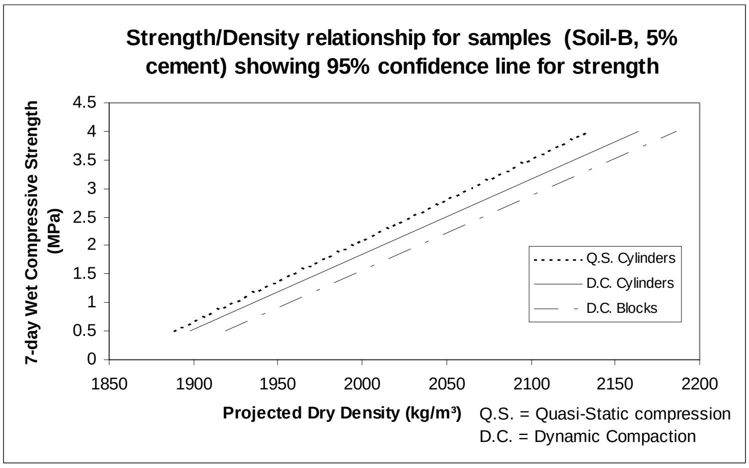
The graph in the figure below summarises two important and related output measures, namely density and strength. Most of the full-size blocks that were made with cement were cured for 6 days and then cut into 100mm cubes before spending 24 hours in water prior to getting crushed. The results of these tests can be seen below. In many cases a block was cut to form two 100mm cubes thus doubling the compressive strength data for that particular block. The graph clearly shows a significant difference between the two ranges of moisture content used. Just as blocks made with 9-10% water had much higher densities, their strengths are also much higher than blocks made with less water. This again demonstrates the need for careful control of the water present in the soil mix to maximise the achieved density and subsequent strength.

Figure 4.11 – Strength results for dynamically compacted blocks



It is not clear from this graph whether or not the relationship between density and strength is similar for these blocks as the small cylinders. But we now have the data to plot the lines of 95% confidence for small cylinders compacted quasi-statically and dynamically as well as full-size dynamically compacted blocks, (see below).

Figure 4.12 – 95% Confidence lines for density/strength relationship for soil-B



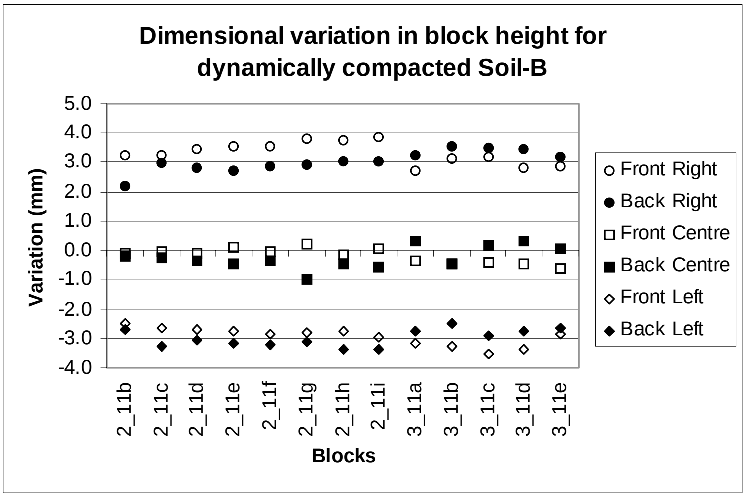
The figure above shows the three density/strength lines and it is encouraging to see that their gradients are very similar. It is disappointing that to achieve the same compressive strength dynamically compacted blocks need to be about 30kg/m³ denser than the quasi-statically compressed cylinders. This is an increase of slightly more than 1%, which cannot be considered normal variation in the material or processes but could be considered as an effect of changing scale. Indeed larger samples typically have lower strengths than smaller ones.

Another measure that has been considered with these results is the dimensional variation of the compacted blocks. The small cylinders exhibited a small variation in their length due to the impactor falling at a slight angle. The same was true for the full-size blocks. We need to be able to confirm that any dimensional variation (other than consistent and in-built variation from the dynamic compaction process, which could be eradicated later in the design) is less than ±2mm to comply with block standards found in (Centre for the Development of Industry, 1998).

We already have a set of blocks that have had their height measured accurately at six points that we can use to determine the height variation of the compacted blocks. The figure below shows a graph that has height data taken from 13 blocks made in the same production cycle over two consecutive days. The method of height measurement was the same for each block and the relative location of the front, back, left, right etc. was the same for each block.

The data is plotted not as absolute values of block height but as a variance from the average height for each block. The pattern displayed within the data clearly indicates that the impactor was not falling parallel with the base of the machine. The left-hand side of the impactor was falling lower than the right-hand side.

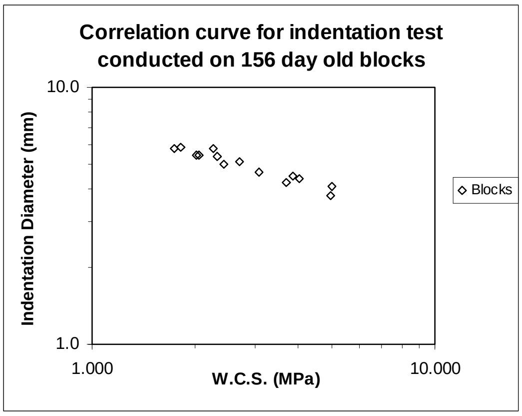
Figure 4.13 – Height variation of dynamically compacted blocks



The above data also confirms that the actual variation across the surface of the block is less than ±2mm once the variation from the incorrectly aligned impactor has been removed. This is an acceptable variation and complies with the block standards. Unfortunately such a large variation would not be tolerable in interlocking blocks unless a different method of block orientation was used.

Measuring the green strength of the finished blocks was not possible using the standard soil penetrometer. The device is not designed to work on such a compact material and the majority of blocks made by dynamic compaction were too dense to get a reading. Consequently another test was developed to monitor the green strength of the block specifically for stabilised soil material. It involves dropping a 1kg mass onto an indentation pin and measuring the resulting diameter of the indentation. It was hoped that this measure would be a non-destructive test that could indicate future block strength as well as level of achieved densification. The results shown here are taken from air dried samples cut from the above 13 blocks after 156 days. The data indicates a definite connection between the diameter of the indentation and the wet compressive strength of the block. The relationship is most easily seen when the values are plotted on log-log axis as shown below. This is a very exciting finding and one that would be well worth exploring further during field trials.

Figure 4.14 – Indentation results from tests on cured blocks



4.4.4 Block variants

The CSSB variants summarised in subsection 3.4.4. indicated that hollow blocks, cement-rich skin blocks and interlocking blocks provided the most significant savings in cement. We could not investigate interlocking blocks, as this would have necessitated mould redesign and further rig development. However, we were able to produce some hollow blocks and some cement-rich skin blocks for testing and analysis.

We produced hollow blocks by reducing the total dry material in the block from 8kg to 6.5kg and adding a pair of wooden frogs (0.0011m^3 volume) to the mould. The soil was mixed with 5% cement and 10% water and carefully placed into the mould in three separate charges to ensure better material placement around the frogs. This was further enhanced by manual prodding of the soil mixture around the frogs prior to compaction. Blocks were made using a 60kg impactor dropped twice from 0.2m and a further 6 times from 0.4m, delivering approximately 1.65kJ.

The finished blocks were ejected with great care, but they still suffered from minor crack defects. The blocks were measured and put to cure for 6 days before having their 7-day W.C.S. measured. The results of the four blocks in question are listed in the table below.

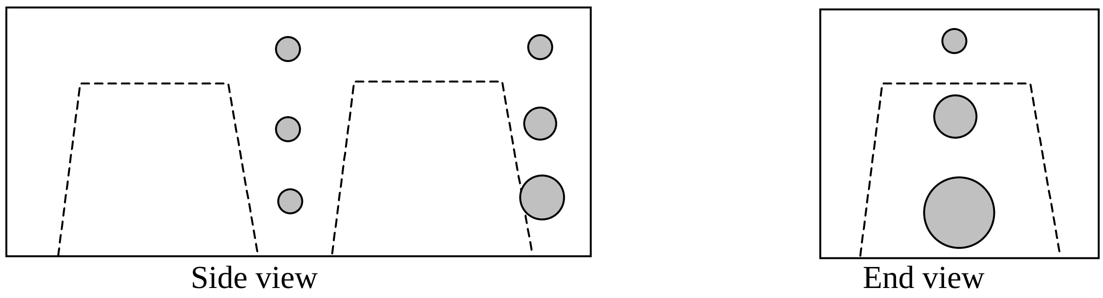
Table 4.4 – Characteristics of hollow blocks

Block Label		4/4a	4/4b	4/4c	4/4d
Average block height	mm	103.0	103.4	103.3	103.0
Standard deviation of heights	mm	0.3	0.4	0.4	0.4
Coefficient of variation of heights	%	0.29	0.38	0.36	0.36
Average P.D.D. of block (including voids)	kg/m³	1554	1548	1551	1555
Average P.D.D. of material	kg/m³	2108	2098	2102	2111
7-day block compressive strength	kN	12.69	12.17	14.18	13.41
7-day W.C.S. of block	MPa	0.31	0.30	0.35	0.33
Minimum 7-day W.C.S. of material	MPa	0.59	0.57	0.66	0.63

In order to draw meaningful conclusions from these results we will need to assess hollow blocks slightly differently than homogenous blocks. The average P.D.D. of the material takes into account the density variation that exists between the top surface and the bottom of the flanges around the central voids. The minimum 7-day W.C.S. of material indicates the compressive strength calculated using the reduced surface area for loading, making it comparable to the W.C.S. of homogenous blocks.

The hollow blocks had a very high average P.D.D. yet exhibited a very low W.C.S. We can suggest that this was because of the very high slenderness ratio of the flange (2.5-3) and the low density of the material at the bottom of the flanges where high strength is needed most. An indentation test on the flanges confirmed that the density at the bottom of the flange was smaller than at the top surface. The diagram below illustrates the results of the indentation test on the different regions of the hollow block. They clearly indicate a rapid change in density and strength in the thinnest part of the flanges.

Figure 4.15 – Sketch of indentation results on flanges of hollow blocks



The indentation tests illustrate the problem with poor material placement and non-uniform consolidation. Whilst we can demonstrate that it is possible to produce hollow blocks using dynamic compaction, these results show a massive (70%) loss in block strength for only a 20% saving in cement. Reducing the cement content from 5% to 4% would have only reduced the strength by around 20-30% and the same savings would have been realised with lower mould complexity and faster production time. The hollow block technique would require further improvement to become an acceptable alternative.

Two cement-rich skin blocks were also produced by dynamic compaction. The production technique for these blocks required the cement rich layer to be placed and spread out in the mould manually. This layer received a single low energy blow after which the rest of the material was added and the total mix compacted together. We wanted the cement-rich skin to be 5mm thick, so approximately 10mm of un-compacted material was placed in the mould. The cement rich layer had a cement content of 10% and the remaining soil had the usual 5% cement. The material was compacted using a 60kg impactor dropped twice from 0.2m and a further 6 times from 0.4m, delivering approximately 1.65kJ.

Upon ejection the blocks were measured and then cured for 6 days. After curing they were air dried for one day and multiple indentation tests were carried out on the block surfaces. Only a small difference in the indentation tests could be noticed between the cement-rich side and the other sides of the block. This was enough; however, it was much easier to visually identify the cement rich layer on the block. The achieved P.D.D. of the two blocks were 1974 and 1965kg/m^3, quite acceptable for the energy transfer and comparable with 10MPa quasi-static compression.

The added complexity of manually placing the cement-rich layer in the mould would make this technique impractical during normal block production. We also do not know the performance of this variant to be able to compare it accurately with other CSSB. Further research would be necessary to determine if this variant would provide the benefits that we want without adding significantly to the machine complexity or the block production time.

It was noticed during the production of these block variants that each block exhibited a dimensional variation of less than ±0.5mm over the top surface, significantly lower than the ±3.0mm experienced on previous blocks. All of the block variants enjoyed special care in the material placement prior to compaction, which could be the reason for the improved dimensional tolerance. Such a small variation would be acceptable for interlocking block manufacture if it could be sustained during normal production. Interlocking blocks were suggested to be another good method of reducing the cement material for walling. Therefore, this finding justifies further research into improved material placement for incorporation into dynamic compaction.

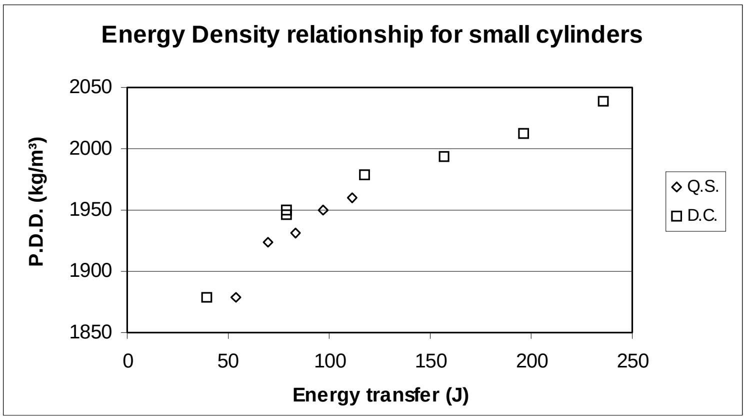
4.5 Comparison of dynamic and quasi-static consolidation

This section summarises the data collected from experiments conducted using the two different compaction methods. It aims to clarify the comparison of the effectiveness of the two methods using several measures of interest, namely P.D.D., energy transfer and block ejection force.

4.5.1 Achieved density for same energy transfer

Using the data that has been collected on small cylindrical samples compacted by quasi-static and dynamic methods we can compare the two methods of compaction. The graph in the figure below shows averages of sample sets of cylinders of the same soil compacted by quasi-static (Q.S.) and dynamic compaction (D.C.). It confirms the original premise that dynamic compaction is somewhat more effective at material consolidation than quasi-static compression.

Figure 4.16 – Comparison of energy transfer for small cylinder production



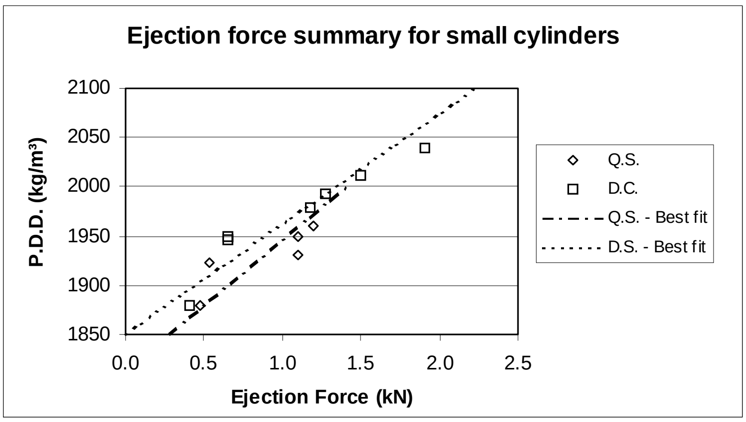
The graph plots data from 4-12MPa pressure and cylinders compacted with 8-24 blows and indicates the greater potential for compaction with dynamic over quasi-static. The application of extra blows delivers extra densification without the need for any machine modification, whereas significant machine modification would be necessary to increase a machine press from 4 to 12MPa or higher.

Early production of full-size blocks was conducted away from the optimum moisture content and this significantly reduced the achieved density for the energy transferred. Later block production indicated that a block could be manufactured with similar P.D.D. as a block compressed with 10MPa pressure using less than 1.7kJ of energy. This represents energy saving of about 20% over quasi-static compression (2.2kJ using soil-A). This also compares favourably to the estimated energy consumption of a 2MPa manual block press requiring 1.5kJ per block and very favourably with the 10MPa manual hydraulically-assisted press requiring 2.9kJ per block. We have therefore confirmed the original premise that dynamic compaction is more energy efficient in material consolidation than quasi-static, providing suitable production parameters are chosen and maintained.

4.5.2 Ejection force

With the results collected from the small samples it is also possible to analyse whether or not dynamic compaction offers any reduction in the ejection force of the compacted samples. This is interesting for the machine designers, as they will have to develop a system to provide the necessary force for block ejection. We have already established that the ejection force at small scale can be extrapolated to full-size blocks, so any findings at this scale can be assumed to apply at full-scale as well. The graph in the figure below shows a summary of the small cylindrical samples and their ejection force plotted against the compacted density. Due to the large variation in the numerical results between supposedly similar tests it is difficult to justify any practical difference between the two sets of data below. It was hoped that dynamic compaction would yield a small reduction in the ejection force for similar density samples and it is possible to see this marginal difference by applying lines of best fit to the data.

Figure 4.17 – Comparing ejection force for dynamic and quasi-static compaction



Such a small difference between the two methods of compaction does not present any real advantage of using dynamic compaction over quasi-static. We believe that much greater advantages can be found in reduced machine complexity as the next subsection explains.

4.5.3 Machine cost and complexity

We have established that dynamic compaction provides slightly more effective compaction for the same energy transfer, but does not improve other criteria such as ejection force. Are there any other advantages dynamic compaction can offer over quasi-static? The simple answer comes from the process by which compaction takes place. Quasi-static compression transmits between 30-70% (Gooding, 1993) of the load applied to the top of the block onto the sides of the mould. The overall machine must also be able to withstand over 100% of the maximum applied load without yield, deformation or failure. These requirements result in significant mechanical structures being applied for safe and reliable machine operation. Whilst low-pressure can be applied by a long lever or cam mechanism, high-pressure requires an additional hydraulic circuit. Hydraulic circuits are expensive and require maintenance for longevity and are typically inappropriate for low-cost applications in developing countries.

Experimental evidence has shown that high-pressure equivalent densities can be achieved by dynamic compaction with very thin walled moulds without any sign of yield or significant strain. This leads us to believe that the dynamic compaction process can be applied to full-size block making to produce high-pressure equivalent densities without the need for thick walled moulds or the complex hydraulic circuit. Removing these features from the machine design represents a large reduction in machine cost. The capital investment required for a dynamic machine would therefore be much less than a comparable quasi-static press, thus making it available to a wider market and more attractive for investment.

4.5.4 Other beneficial block characteristics

During the production of CSSB and their variants by impact compaction, several beneficial block characteristics were noticed. These may be of limited value but present some interesting features of dynamic compaction that have not been recorded during quasi-static compression. Gooding suggested that dynamic compaction delivered more uniform compaction, and this phenomenon has also been seen in the production of the full-size blocks through the indentation tests.

Another two features that have been identified as beneficial are to do with the surface characteristics of the block. During the cutting of the full-size blocks into cubes for compression strength tests it was noticed that the exterior skin of the block was harder than the core. This was further confirmed with the indentation test. The removal of the block from the ‘splitable’ mould does not cause the usual scraping and wiping effect normally experienced with ejection from quasi-static presses. The process of releasing the mould from around the block (something that is not appropriate with quasi-static compression) is delivering a visibly superior block surface. The combination of improved surface finish and increased surface hardness gives the finished block slightly better resistance to environmental attack and abrasion, truly a beneficial by-product of dynamic compaction.

4.6 Chapter summary

The results presented in this chapter have been very encouraging. We have also been assured that the experimental data collected is sufficiently accurate and repeatable to draw sensible conclusions from them. The inherent variation experienced during tests on quasi-static samples is very similar to the variation in impact compaction, from which we can conclude that the dynamic compaction process does not add any further variation. This small variation (±0.5%) is also present during full-size block manufacture.

We have established that dynamic compaction provides some 20% more energy efficient consolidation than quasi-static for each scale investigated. During block production small deviations (±2%) from the optimum moisture content will require additional energy to achieve desired consolidation. Compaction to 10MPa pressure-equivalent densities has been successfully achieved and many samples achieved even higher densities with additional blows. After choosing appropriate production parameters block P.D.D. was frequently over 2000kg/m^3, and exhibited 7-day W.C.S. of over 2MPa. From these findings we also re-calculated the relationship between P.D.D. and the W.C.S. for full-size blocks and found it to be only slightly different to the relationships discovered with the small cylinders.

The transfer of energy into the block via the falling impactor has a number of variables associated with it. Optimisation of the impactor mass, drop height and number of blows applied was investigated experimentally on small cylindrical samples. This indicated that the momentum transfer was not a critical parameter for compaction across the range investigated. This was assumed to be the same for full-size blocks as well and consequently the total energy transfer was monitored more carefully. Different impactor arrangements used during block manufacture indicated that a good solution was to use a 60kg impactor falling from 0.4m between 8 to 16 times.

Other aspects related to dynamic compaction were also investigated with some interesting findings. The practice of indirect compaction, (via intermediary billet) greatly reduces the potential for consolidation and should be avoided if possible. The losses of around 10% on density would result in unacceptably high strength losses of as much as 50%. The stiffness of the machine foundations was also found to have a practically significant effect on the final block properties. Increasing the delay between impacts was found to have a statistically significant effect on the blocks, but fortunately this was of little practical significance and any extra delay would have increased the production time unacceptably.

Our understanding of block manufacture by dynamic compaction has been greatly enhanced and will provide valuable guidance for machine design. It was discovered that thinner walled moulds are not only acceptable for dynamic compaction, but also yield slightly better consolidation compared with very thick walled moulds. A two-part thin walled mould was successfully implemented during full-size block production and overcame the problem of block ejection. Poor block tolerances were caused by a combination of poor impactor constraints and poor mould filling. These issues may need to be further assessed for inclusion into an appropriate machine design. Block variants were also successfully produced using impact, but the increased production time and poorer block characteristics recommend further research and improvements to the process.

The numerical data collected during these experiments have also given us an idea of the processes taking place during dynamic compaction. We now know the compaction curves for the material at different moisture contents and different impactor arrangements. This information can help us to suggest the mechanisms of impact. We believe that impact generates significant forces that cause consolidation, but the magnitude of these forces is still not known. Interesting findings concerning mould wall thickness lead us to believe that other mechanisms are acting during the compaction that are different to quasi-static compression. Closer inspection of the point of impact, its duration and effects now needs to be carried out.

5 Impact Mechanism

The literature review indicated that little was known about impact compaction and even less about impact compaction of confined soil. The results of experiments conducted in the previous chapter help us to make certain assumptions about the mechanisms involved in compacting soil by impact. The experiments described in this chapter have been conducted to investigate some of the fundamental mechanisms acting during an impact blow. Our motivation for conducting these experiments is twofold, firstly to improve understanding of the process of impact compaction and secondly to assess the magnitude of the forces delivered during impact and thereby assist with machine design. This chapter will be split into two sections. The first section develops a series of models for the compaction process and for impactor motion. The second section describes experiments conducted to measure certain features of dynamic compaction and assesses the models against the experimental evidence.

5.1 Models of compaction

The soils literature adequately describes the effects of soil compaction of soil without actually explaining any of the mechanisms that take place. Soil consolidation is achieved by bringing particles closer together and as a result driving out some gaseous material, and under extreme conditions, some liquid material as well. This section will develop models for three separate areas of interest; the force distribution during compaction, particle-particle interaction, and air/water dismissal. Eight conceptual models are presented below:

1. Force distribution during compaction,
    - Shock wave propagation to bottom and back
    - Compression transfer to top and sides
2. Particle-particle interaction
    - Sliding of particles past each other
    - Knocking off of asperities
    - Plastic deformation of lumps of clay
3. Air/water dismissal
    - Expulsion of air in the short duration of impact
    - Diffusion of air into water
    - Pressurisation of air in core followed by its slow diffusion out

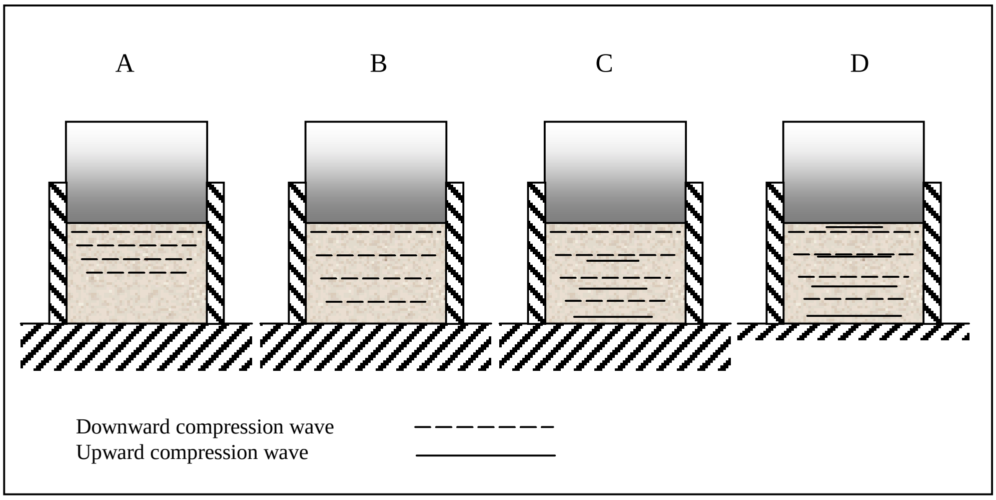
5.1.1 Force distribution during compaction,

It is already known that during quasi-static compression the sides and the bottom of the block mould feel only a fraction of the pressure exerted onto the top of the block, (typically 30-70%). We assume that the same is true for dynamic compaction, because the medium for the force transfer is the same, namely the soil. Whatever force is applied to the top will be felt in some measure on the sides and the base, but how the force is distributed during impact is still not understood.

Shock wave propagation to bottom and back – Experiments conducted using soft foundations indicated the phenomenon of shock wave propagation through the material during an impact blow. This was seen most clearly when de-lamination occurs, (i.e. when upper layers of the block have an internal tensile force applied that exceeds cohesive forces and material separation results). Such a phenomenon suggests that a shock wave impulse formed by the falling impactor is travelling through the soil.

The diagram in the figure below shows four different possibilities for compaction wave propagation through material confined in a mould. The horizontal lines in the material represent the wave front of the compaction at a succession of times. This wave reaches the bottom of the mould in C and D and reflects back up as a rarefaction wave upward through the material, depicted by the solid lines.

<u>Figure 5.1 – Compression wave propagation during compaction</u>



A – Has very low impact energy (similar to vibration) and only compacts the upper layers.

B – Has impact blows of higher energy that deliver shock waves as far as the bottom of the mould.

C – Has even higher energy impact blows and causes a reflection wave to bounce off solid foundations and travel back up some distance into the material.

D – Has elastic foundations and higher energy impact blows that causes a significant reflection wave travelling all the way back up the material causing de-lamination at the top of the block.

We can determine the speed of these compression waves if we know the speed of sound through the material. Sound waves travel through a material at a rate determined by the bulk modulus (G) and the material density (ρ) and using the following formula:

$$c = \sqrt{\frac{G}{\rho}} \qquad (2)$$

We can estimate the density of the material during compaction from the height of the block, but we need a method of determining the value of G for the material as well. Whichever method is used to determine G and the resulting speed of sound through the material should be verified experimentally in some way. The concrete industry uses a Pundit tester to estimate concrete strength from the speed of sound passing through it. Such a test would be acceptable to determine the speed of sound through the compacted material.

Compression transfer to top and sides – This model is similar to quasi-static compression model. It suggests that the force applied to the top surface of the material from the impact blow is transferred through the material along slip planes. If this force is large enough, the particles resisting the force give way and move closer together. Similarly to quasi-static the maximum forces are felt on the top surface and they reduce significantly as one progresses through the material and towards the bottom (since the growing friction of the vertical forces has been transferred to the mould sides). This model would suggest that compaction force would be smaller at the bottom of the block and therefore display a lower achieved density. This material characteristic is well known in quasi-static compaction, but has not been noticed during previous or current dynamic compaction research.

5.1.2 Particle-particle interaction

Sliding of particles past each other – Typical soils consist of large particles surrounded with smaller ones. The very smallest particles are clay, which have a flat plate-like structure with water molecules bonded to these plates. In the presence of additional water and/or force these plates will slip past one another. As the force is applied to the material (via impact or squeeze) these clay particles slid past one another enabling the larger particles (that they are coating) to move into a closer arrangement. This model seems to most accurately explain the effect of better consolidation from the addition of extra water as seen in both the soils literature and the dynamic compaction experiments.

The dynamic viscosity (μ) of water and air at 25°C are 0.001 Ns $m^{-2}$and 1.853×10^{-5} Ns m^{-2}. From this we can calculate a maximum likely shear stress (τ) for these fluids assuming a 1m/s velocity change (*du*) over 0.1mm (*dy*) using:

$$\tau = \mu \frac{du}{dy} \tag{3}$$

This yields a shear stress of 10Pa for water and 0.2Pa for air. These stresses are tiny compared with the shear strength of the solid component. Therefore we can conclude that any slipping is due to the very small shear forces between particles surrounded by air and water rather than shear planes through solid material.

Knocking off of asperities **–** This model may apply if the presence of clay or moisture is too small for sliding to occur between particles and the forces applied are large enough for material fracture. Sharp points on the particles may break off during the application of forces exerted by the impact blow. The load path through the material will be predominantly through point contacts between particles and these may crush or break under force. As compaction continues the number of point contacts increase until the force applied is sufficiently resisted without any further crushing and hence consolidation ceases.

If we assume the compressive strength of the rock particles is approximately 500MPa, this is significantly higher than the mean pressures that we are expected from dynamic compaction over the total surface of the block. If we were achieving a mean pressure of 10MPa there would need to be a 50 to 1 stress concentration for localised crushing to occur. Considering the wide range of particle sizes and close packing of them together, this seems highly unlikely and therefore we can assume that the forces are not physically affecting the solid particles.

Plastic deformation of lumps of clay **–** Clay will exist in the form of closely packed lumps that have not been broken down into smaller pieces during the soil mixing process. In the presence of water these lumps are quite soft and will deform under applied force. Initially these lumps may hold harder particles further apart but as the forces applied during compaction are exerted these lumps will deform and permit closer arrangement of the harder particles. These lumps of clay will have a shear stress dependent on the quantity of water present in them, but much lower than the solid rock material present throughout the soil.

5.1.3 Air/water dismissal

Expulsion of air in the short duration of impact **–** The soil comprises of three phases, solid, liquid and gas. For the applied pressures it can be assumed that the solid and liquid phases are incompressible compared to the gaseous phase. During the first blow the air volume reduces to approximately 65% of its initial volume. If the air was originally at 1 bar (or 0.1MPa) and the volume is reduced by 35% then (assuming no air loss or temperature change) the new air pressure is 0.15MPa. Substituting an adiabatic assumption for an isothermal one raises this pressure to 0.18MPa. During the first impact blow a small amount of dust is usually ejected from the mould along with the expelled air. We believe that this air loss constitutes a significant proportion of the volume reduction experienced by the block during the impact blow. It is possible that some of the air does not escape during the impact time and becomes trapped and compressed within the block. The above values indicate that the increase in pressure would be very small assuming most of the air escapes during the impact blow.

Diffusion of air into water **–** As pressure is applied to the mixture and particle-particle intimacy increases the mixture of air and water and pressure could cause some of the air to diffuse into the water. What then happens to the air when the pressure is removed is open for debate. The solid particles could keep the pressure on the overall matrix and keep the air dissolved in the water, placing the material into tension. Alternatively the air could slowly diffuse out of the water and out of the material long after compaction has been completed. The quantity of air that could be diffused into the water depends on the amount of water present, the applied pressure, the temperature and the amount of air already dissolved in the water. Whilst we cannot rule out the possibility that air may diffuse into the water we can suggest that the effects will be small for a number of reasons. There will only be a small amount of air under pressure throughout the block and the pressure applied to the air will also be small. We have already established that if none of the air escaped then the air pressure within the soil would increase to less than 0.2MPa. We also know that some of the air does escape so it is even more unlikely that air would be diffusing into the water within the soil during initial blows. During latter blows the pressure might be higher, but the volumetric change in the air is even smaller so the effects will still be limited.

Pressurisation of air in core with slow diffusion out **–** We believe that at some stage during the compaction air is being trapped within the block and becoming pressurised during further consolidation. At the end of compaction this trapped air will be at a greater pressure than the local atmospheric conditions. This could suggest that the air pressure in the pores increases during each blow and pressure equalisation occurs some period later as the high-pressure core slowly diffuses out of the material between blows. It could also suggest that as this pore pressure is increasing it could marginally hinder further consolidation until the pressure has equalised with the atmosphere.

5.1.4 Theoretical models for impactor trajectory during impact

The process of the dynamic compaction is assumed to include a combination of elastic, plastic and possibly viscous effects. In order to anticipate what sort of compaction was dominant in the different stages of compaction, a set of basic equations of motion was derived. These describe the motion for the different effects taken one at a time, (plastic, elastic and viscous) from which a possible position trace could be generated. Below are the theoretical derivations of plastic, elastic and viscous models of impactor retardation.

Plastic deformation **–** constant retardation a = -*k*

$$x = -\frac{kt^2}{2} + at + b = pt^2 + qt + r \qquad ①$$

$$\text{and} \quad v = \frac{dx}{dt} = 2pt + q \qquad ②$$

(i) at $t = 0$, $x = 0$, inserting into ① gives $r = 0$ ③

(ii) at $t = 0$, $\frac{dx}{dt} = v_0$, inserting into ② gives $q = v_0$ ④

(iii) at $v = 0$, $t = T$, inserting into ② gives $p = \frac{-v_0}{2T}$ ⑤

(iv) However at maximum indentation ($v = 0$), $x = X$,

inserting ③④⑤ into ① giving: $\therefore X = \frac{1}{2} v_0 T$ (4)

Elastic deformation – retardation is proportional to penetration $\therefore m\ddot{x} = -kx$

$x = A\cos\omega t + B\sin\omega t$ ①

where $\omega = \sqrt{\dfrac{k}{m}}$ is constant

and $v = \dfrac{dx}{dt} = -A\omega\sin\omega t + B\omega\cos\omega t$ ②

(i) at $t = 0$, $x = 0$, inserting into ① gives $A = 0$ ③

(ii) at $t = 0$, $v = v_0$, inserting into ② gives $B = \dfrac{v_0}{\omega}$ ④ $\therefore x = \dfrac{v_0}{\omega}\sin\omega t$

(iii) at $v = v_0\cos\omega t = 0$, $t = T$, giving $\omega = \dfrac{\pi}{2T}$ ⑤

(iv) However, at maximum indentation (v = 0), x = X, inserting ③ ④ ⑤ into ① gives: $X = \dfrac{v_0}{\omega}\sin\omega T$ (5)

Viscous deformation – retardation is proportional to velocity $a = -cv$

$x = P + Qe^{-ct}$ ① and $\dfrac{dx}{dt} = -Qce^{-ct}$ ②

(i) at $t = 0$, $x = 0$, inserting into ① gives $Q = -P$ and $\therefore x = P(1 - e^{-ct})$ ③

(ii) at $t = 0$, $\dfrac{dx}{dt} = v_0$, inserting into ② gives $Qc = -v_0$ ④ $\therefore P = \dfrac{v_0}{c}$ and $\therefore c = \dfrac{v_0}{P}$

(iii) at maximum indentation (v = 0), t = T, $\therefore X = P$ and x = X inserting into ③ gives $x = X(1 - e^{-\frac{v_0}{X}t})$ ⑤

(iv) Relationship between T and X is $T >> \dfrac{X}{v_0}$

1. If pure plastic deformation occurs then the relationship between T and X is

$$T = \frac{2X}{v_0}$$

2. If pure elastic deformation occurs then the relationship between T and X is

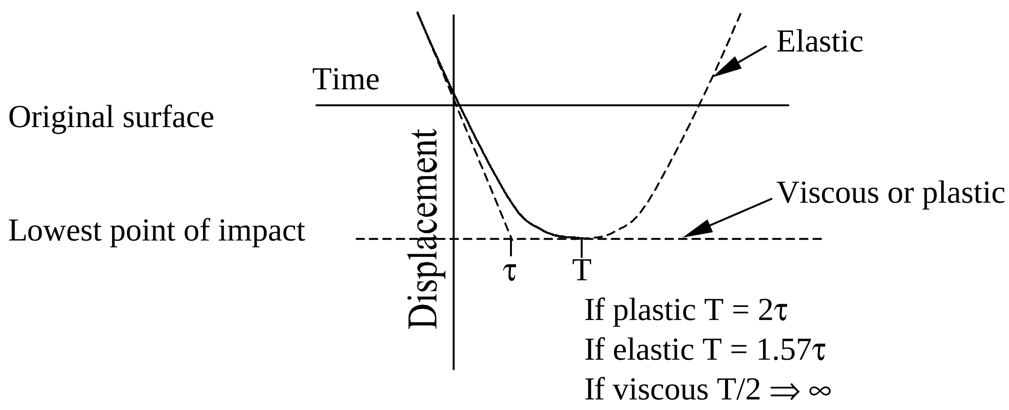
$$T = \frac{\pi}{2}\frac{X}{v_0}$$

3. If pure viscous retardation is dominant then the relationship between T and X is

$$T >> \frac{X}{v_0}$$

The graph below shows the possible displacement/time trace for the falling impactor as contact with the surface is made and compaction of the material results. The solid curved line represents the trace of the falling impactor whilst the dashed line indicates the original velocity at impact extended to cross the lowest point of impact to determine τ.

<u>Figure 5.2 – Theoretical displacement analysis of impactor</u>



It was hoped that if an actual trace of the motion of the impactor could be gained then this trace could be analysed with reference to the theoretical traces. This would help to determine the dominant effect in the different stages of compaction and perhaps lead to a deeper understanding of dynamic compaction. If we can analyse the position of the impactor accurately then we should also be able to analyse the position of the top surface of the block during the compressive part of the impact.

5.2 Experimental measurement of impact

Earlier in the project it was assumed one real advantage of impact compaction was that the forces delivered to the block were smaller than with high-pressure quasi-static compression. It was of both academic interest and economic interest to determine whether this assumption was correct or not. Academic because the actions of an impact blow onto the surface of a confined soil sample had yet to be analysed. Economic, because lower forces justify the use of less material in the mould design and general machine structure.

Before attempting to accurately monitor the position of the impactor during a series of impact blows, we estimated certain characteristics from known equations of energy and motion. From experiments conducted earlier we can estimate the actual impactor drop height from relative impactor positions before and after each blow. This data gives the distance travelled and the deformation achieved by the applied blow. The impact sequence can be divided up into a series of sections:

The impactor lift: h_i = height lifted to impactor stop,

i = blow number

During impactor free-fall: $v^2 = 2gh_i$,

$E = 1/2mv^2$,

$a = g\ (9.81 m/s^2)$

Contact with material surface: $F_C = ma$, (where 'a' is <u>not</u> constant)

Retardation of impactor and compaction of material: $v = 0$,

$u^2 = 2gh_i$

Elastic restitution of material: $e = \frac{v_2^{'} - v_1^{'}}{v_1 - v_2}$ where v_2 and $v'_2 = 0$

Impactor bounce: $E_b = mgb_i$,

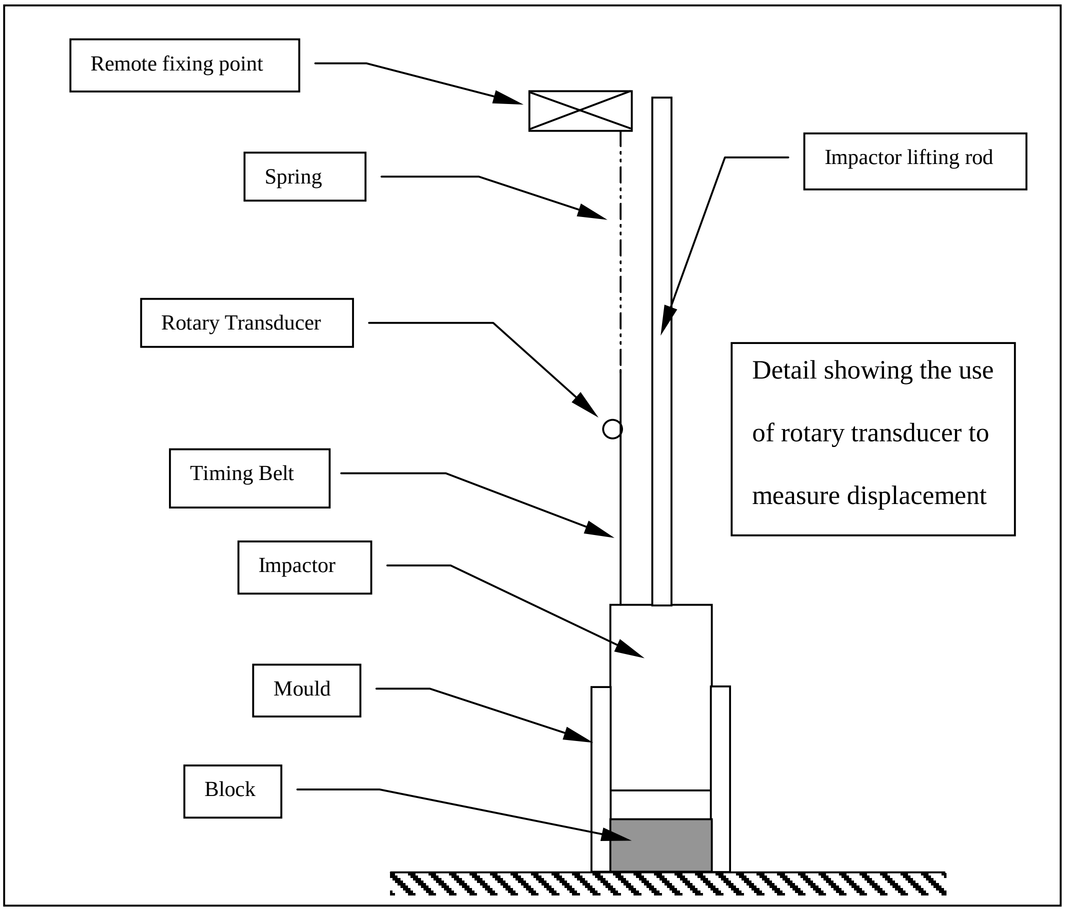
b_i = impactor bounce height

Experimentation was conducted on dynamic compaction of soil-B with 6.5% moisture and using a 36.8kg impactor falling through approximately 200mm. Therefore the impactor velocity at impact would be approximately 2m/s and the impact energy would be around 74J.

Several different methods could be used for monitoring the dynamic blow. Remote measurement could be used via a laser or sonic pulse monitoring absolute position. Alternatively a mechanical device or sensor that was attached to the impactor in some way could also monitor the location (e.g. via a rotary transducer) or acceleration (via an accelerometer) of the impactor. For practical and economic reasons it was decided to use a rotary transducer and monitor the relative position of the impactor during the compaction cycle. An accelerometer was also used, but it was only rated up to 25g and consequently could only be used to indicate the point of maximum acceleration rather than measure the magnitude of it.

The rotary transducer was connected to a toothed wheel and mounted onto the impactor guide. A timing belt passing over the toothed wheel had one end connected to the impactor base and the other end connected to a series of springs before being fixed to a remote part of the rig. This arrangement enabled the timing belt to move freely up and down past the rotary transducer whilst also staying sufficiently taught to accurately measure displacement both on the upward and downward strokes of the impactor. A diagram of the arrangement is shown below. The tension in this timing belt is negligible compared with the weight of the impactor.

Figure 5.3 – Diagram of sensor position for impactor analysis



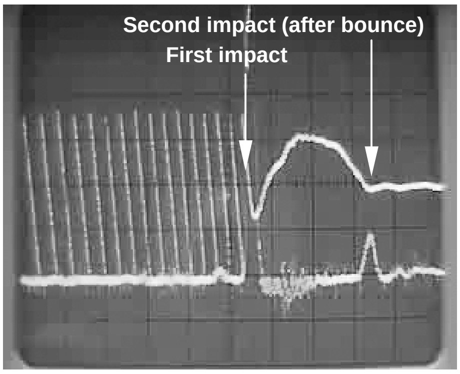
A digital decoder was connected to the rotary transducer to give an 8-bit number output from the transducer and this was intended to be transmitted directly into the parallel port of a computer. Unfortunately serious interfacing problems were encountered in capturing the digital data from the rotary transducer. After many weeks of trying, this method was shelved and another system of analogue analysis using a storage oscilloscope and digital video camera was implemented instead.

The resulting video was then analysed frame by frame to determine the actual position of the impactor relative to the impactor's starting position on the oscilloscope trace. This procedure was incredibly laborious and generated results of only passable numerical accuracy. Consequently it was only carried out on traces taken from the 1st and 38th blow delivered to a sample of soil. Traces from other blows were only analysed at the impact point determining the amount of compaction achieved, the impactor bounce height and the elastic displacement of the material.

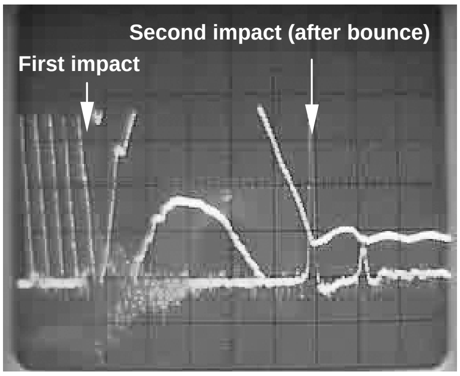
5.2.1 Impactor position, velocity and acceleration

Below are snapshots taken from the compaction video of two separate blows delivered to a test sample. On the left of the snapshot one can see the descent of the impactor indicated by the almost vertical lines running from the top to the bottom of the screen. Eventually contact is made with the sample and the impactor comes to a stop before bouncing backwards, (indicated by the hump). The flat line on the right side of the snapshot gives the final resting position of the impactor.

Figure 5.4 – Signal traces from rotary transducer and accelerometer during impact



5th Blow

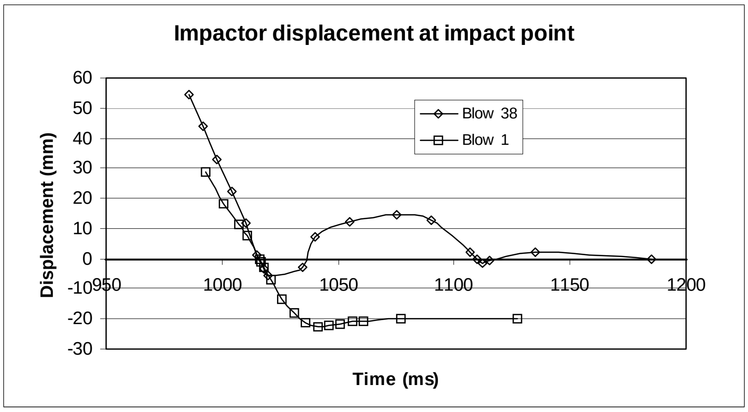


29th Blow

Spikes on the second line on the snapshots (the accelerometer trace) indicate the peak acceleration at impact both for the initial impact and for the subsequent bounce(s). It is clear that during the 5th blow the accelerometer is experiencing significant shock from the impact blow well in excess of its working range of 0.5 volt (vertical scale 1 div = 0.5V = 250m/s^2). The shock becomes even more significant during latter blows. The rotary transducer was calibrated for displacement using a different system than the recording system and this resulted in an error of a factor of two throughout the data recorded. Once the error factor was found, then the data could be adjusted and the corrected results are reasonably close to the theoretical calculations for the experiments, hence making them of satisfactory accuracy for experimental interpretation.

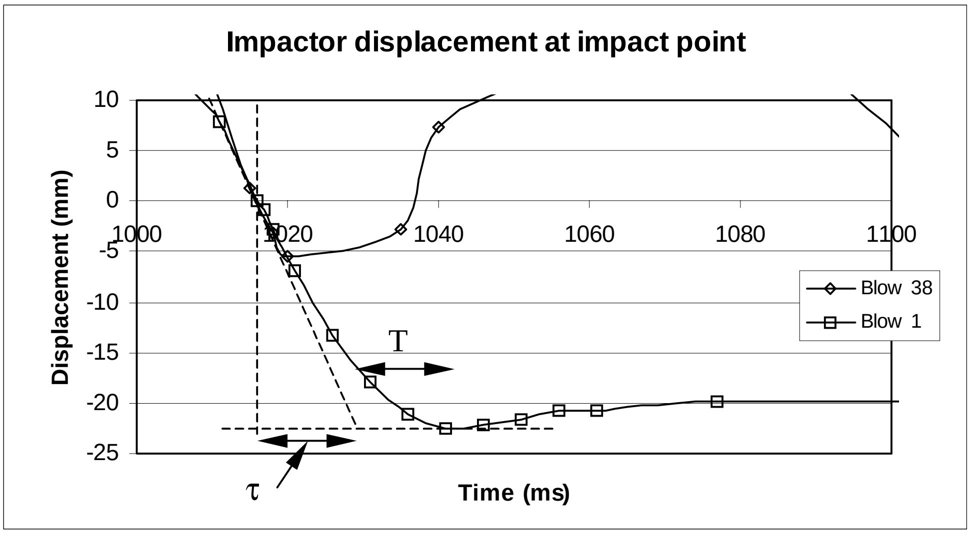
The following data comes from another block that was manufactured by impact compaction with careful analysis of the video traces received from the experimental equipment described above. The results of the two position traces from the 1st and 38th blow can be seen in the graph below. For both of the traces the approximate soil level prior to the impact has been indicated as zero displacement and from this the level of compaction or the elastic deformation can be identified. The data presented has also been time shifted to superimpose the two traces so that they coincide at their respective point of impact. The results demonstrate the significant difference between the initial blow applied and the much later blow where densification per blow has become very small.

Figure 5.5 – Graphical representation of impactor displacement



We can take a closer look at the impact region of the graph and analyse it with respect to the models generated earlier. The graph in the figure below shows the graphical interpretation of the trace. It is unfortunate that these numerical results do not seem to correspond very well with the theoretical models for impactor motion. This can be seen with the initial blow and where the impactor initially comes to rest more than 20mm below the original surface. The theoretical data suggested that for a completely plastic material deformation $T = 2\tau$ and if completely elastic $T = 1.57\tau$. The dashed lines drawn onto the graph indicate the theoretical lines for determining T with respect to τ. By inspection we can see that $T < 2\tau$, but also less than 1.57τ.

Figure 5.6 – Close up of impact point



The graphical trace indicates at least three things. One, the curvature of the graph increases with displacement, (favours elastic over viscous interpretation). Two, $T<1.57\tau$ indicates some plastic or viscous action. Three, Rebound is small indicating plastic or viscous action. During the 38th blow the evidence is also a little confusing. Latter blows would be expected to be predominantly elastic as any plastic deformation has reduced to almost zero. However, the above graph indicates that for the 38th blow $T \varphi \tau$. Perhaps these results could be interpreted in the following way. Initial blows are predominantly plastic (T is large) but as the sample becomes more compacted the plastic element decays and an elastic component becomes noticeable (T is smaller). This does in fact correspond with the theoretical data as 2τ (plastic) is larger than 1.57τ (elastic). Perhaps the numerical accuracy of the data received doesn't warrant any deeper or further analysis than this.

The changes that occur to the compaction process can also be monitored from the data received from the compaction video. The figure below shows the impactor displacement for three separate stages of an impact. The plotted variables are:

Compaction – permanent change in block height from the applied blow

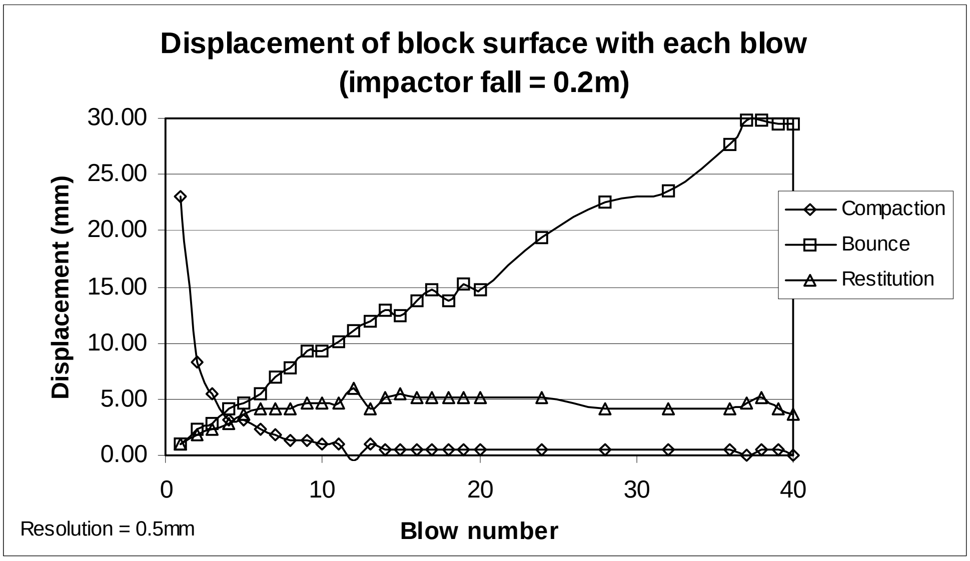
Restitution – assumed elastic deformation of block during impact, defined as the recoil height of block surface from maximum indentation to final steady state

Bounce – height of impactor bounce after impact relative to final block surface (assumes negligible compaction from subsequent bounce impact)

The graph shows some very interesting phenomenon that has not been seen before. We can see that the compaction graph complies with the pattern of dynamic compaction as seen already, which gives us assurance that the measurement method is working OK. However, what we haven't seen before is the elastic region of the impact and the impactor bounce height relative to the block surface. It seems that there is a limit to the elastic deformation of the material, possibly dependent mould stiffness rather than the block material itself. This limit of elastic deformation stabilises at about 5mm after the first few blows. It should also be remembered that the impactor energy is almost constant, especially after the first few blows.

Another more striking feature of this data is that the bounce height of the impactor increases linearly after each blow and only seems to level off during the last few blows. This result combined with the almost constant elastic deformation of the block gives us a good clue about what is happening within the material. As the material becomes compacted the elastic restitution increases with each blow. Therefore subsequent blows achieve a greater bounce height than the previous blows do. The elastic restitution is dependent on the materials that come into contact and their respective velocities. Generally the elastic restitution is related to the material hardness, and we believe that the block is becoming harder with compaction and therefore this fits in with the data quite well.

Figure 5.7 – Impactor position analysis for complete block production



Up until the 20th blow a marginal compaction could be measured for each blow. After this the compaction is measured after every four blows to ensure a measurable compaction within the resolution of the equipment. It can be seen that the increase in compaction per blow drops to virtually zero after 20 blows, yet the bounce height seems to rise steadily. This phenomenon suggests that the elasticity of the material is increasing with each blow.

5.2.2 Changing material properties during compaction

We already know that the density of the material changes during compaction, but we now have good reason to believe that the overall material stiffness is also changing during compaction. This seems plausible as the material is increasing its resistance to further consolidation with similar energy blows and the impactor rebound height is increasing. We know the mechanical properties for the air and water and their respective fractions during the compaction sequence. We can also assume a certain overall stiffness of the material from the elastic restitution and rebound height of the impactor.

We can apply a parallel stiffness model or a series stiffness model to the composite material of air, water and rock. If we select the parallel stiffness model the stiffness of the material will be dominated by the rock, whilst the series stiffness model will be dominated by the air. We believe that the air within the block is playing a significant part in the compaction so we will apply the series stiffness model first.

If we replace stiffness by elastic bulk modulus we can estimate the elastic bulk modulus of the material using the following equation: $\frac{1}{G'} = \frac{\lambda}{G_a} + \frac{\gamma}{G_w} + \frac{1-\lambda-\gamma}{G_r}$

Where γ and λ are the volumetric fractions of water and air during a sequence of impact blows.

By using $c = \sqrt{\frac{G}{\rho}}$ and working backwards from the speed of sound through air and water and their densities we can estimate the values of G for air and water to be 0.13MPa and 2.0GPa respectively. We have a range of 20 to 150GPa for the Young's modulus (E) for rock taken from an Ashby diagram (Department of Engineering, 1996). Depending on the constraints this can be converted to bulk modulus using either $G = \frac{3E}{8}$ (tri-axial) or $G = \frac{E}{3}$ (uni-axial). This gives us a possible range for G of 6.7GPa to 56.3GPa. If we take a midrange value of 30GPa and calculate G' for the material at the beginning and end of consolidation we get a range of 0.28MPa to 0.78MPa. Knowing the final density as 1880kg/m³ we calculated the final speed of sound through the material as 20m/s, (far less than the speed of sound through air alone). We then tested the speed of sound though a similar freshly compressed block using a pundit tester and after calibrating the device a speed of 490m/s was recorded. This suggests that the speed of sound through the block is dominated by the speed of sound through the small fraction of air and the series model is inaccurate by at least an order of magnitude.

With the parallel model we assume that the material is stacked in parallel and calculate G' using $G' = G_a\lambda + G_w\gamma + G_r(1-\lambda-\gamma)$. This model yields a range for G' between 14GPa and 21GPa. If we again attempt confirmation by calculating the speed of sound through the material we get a maximum speed of 3342m/s, (grossly dominated by the speed of sound through the solid). From this we can assume that the mechanism by which sound travels through a composite material follows a model other than the parallel or series stiffness model. One would expect that the sound would travel through the solid material, hindered by the point contacts and interfaces with the water and air that surround them.

We can propose one further model that suggests that the total time through the material is the sum of the times through the solid, water and air taking the proportions rock, water and air as a fraction of the whole distance travelled. Taking the fractions of the total volume of the finished block and the speeds of sound through rock (estimated using G as 30GPa), water and air, using: $c' = c_a\lambda + c_w\gamma + c_r(1-\lambda-\gamma)$ we get an overall speed of 2620m/s. Unfortunately, as this model depends on the accuracy of G for the rock we cannot suggest that this model is superior than the parallel model despite yielding a better value for c'.

These investigations suggest that there is some link between the increasing consolidation affecting the block bulk modulus. The ranges for G' are numerically inadequate because they are based on a wide range of G for rock. Using the speed of sound through a *cured* block that was made yields a value for G' at around 10GPa. However, assessment of the Young's modulus on a freshly quasi-statically compacted block still constrained in the mould gives a value of 0.7GPa. Converting this to bulk modulus results in a rather low value of 2GPa, similar to that of water. The curing process will make the block stiffer and therefore increase the bulk modulus significantly so an initial value of 2GPa increasing to 10GPa is not unreasonable. Unfortunately we have not been able to establish a suitable model to determine the value of G' during the compaction procedure. All we can say is that as the material becomes compressed the rebound height increases and therefore the stiffness of the material also increases. Without an accurate value for G for the rock material we cannot verify our models or the results attained.

5.2.3 Maximum acceleration experienced by impactor

Early calculations conducted by (Montgomery, 1997) utilised an estimated stopping distance and time to estimate the acceleration and hence the force applied to the surface of the block. With the data that was collected from the rotary transducer a more accurate estimate could be made for the stopping distance. From the known impactor velocity prior to impact (calculated from the drop height) and the distance in which the impactor came to a stop the acceleration could be calculated.

By ignoring the horizontal scale on the traces (defining the time) and just looking at the height changes from the vertical scale it was determined that during latter blows the impactor came to a stop in 5mm from a velocity of 1.8m/s (drop height of 0.19m). By assuming ideal plastic deformation and hence constant rate of change of velocity we can determine the acceleration, (using $v^2 = u^2 + 2as$ where v = 0 u = 1.8m/s and s = 0.005m), giving a constant deceleration of 324m/s².

For *plastic* deformation model use: $v^2 = u^2 + 2ah$ which gives a value of a = <u>324m/s²</u>

If deceleration is proportional to penetration (i.e. *elastic*): $a = -kx = v\, dv/dx$

$k = (1.8/0.005)^2 = 129{,}600$

Max acceleration = $0.005k$ = <u>648m/s²</u>

If compaction follows *viscous* model then: $a = -cv = v\, dv/dx$

$c = V/X$, $a_{max} = cV = V^2/X$

Max acceleration = $1.8^2/0.005$ = <u>648m/s²</u>

Maximum acceleration during elastic or viscous deformation would be about twice as large at around <u>650m/s²</u>.

These values do not take into account the bounce achieved by the impactor, that drive the acceleration experienced by the impactor even higher. If the impactor bounces upwards with an initial velocity of 0.9m/s then the total change in velocity is 2.7m/s. If the compaction distance is 0.005m and the elastic restitution distance is 0.006m then s = 0.011m and the maximum constant acceleration is 331m/s^2. Again this value could be as much as twice as large for elastic or viscous effects.

5.2.4 Losses in the system

During very late blows no significant densification is achieved with each blow. It can therefore be assumed that no useful work is being done to the material and all the impact energy is being lost through a number of mechanisms.

Kinetic energy **–** energy restored to the impactor causing it to bounce and vibration energy dissipated through the floor and foundations

Heat energy **–** hysteresis losses from elastic displacement of material and mould

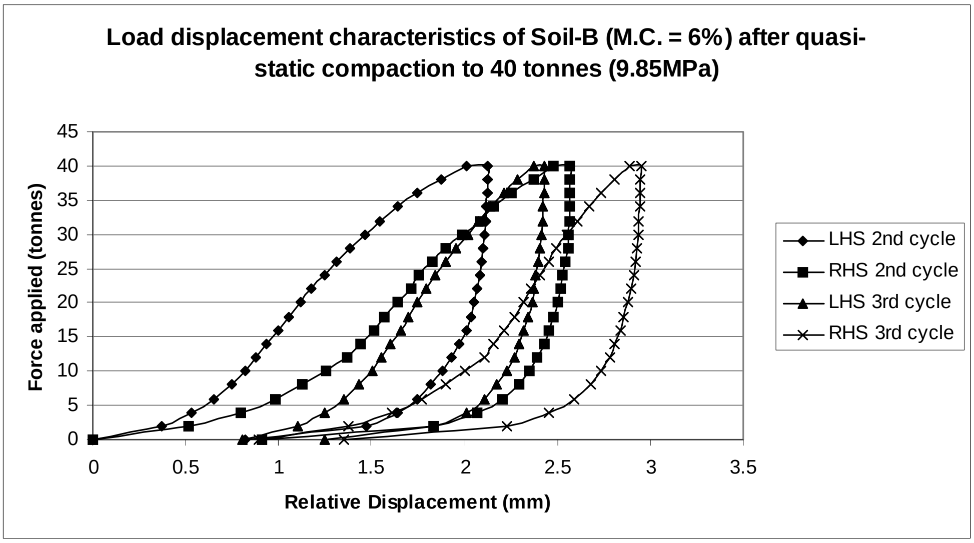
Sound **–** energy lost through the generation of noise

Unfortunately the only mechanism that we can easily measure is the kinetic energy restored to the impactor. The rebound height of the impactor after impact gives an indication of the elastic energy required generating a restoring force sufficient to make the impactor bounce. We can assume that the energy attained by the impactor using E_b=mgb_i is equal to the elastic energy delivered by the material into the impactor. Using initial bounce velocity of 0.9m/s yields a final bounce height of 0.04m requiring 15J of energy for a 36.8kg impactor. Initial impactor energy is around 75J so approximately 20% of the energy is lost during impactor bounce. The rest of the energy is lost through a combination of vibration dissipation, hysteresis and noise.

We have experimental evidence for the hysteresis losses incurred by quasi-static compression of full-size blocks from tests conducted on *soil-B*. The elastic deformation from impact compaction is higher than with quasi-static compression so it is reasonable to assume that the hysteresis losses would also be larger.

The data presented in the figure below is from the compression of a full-size block of soil-B compressed to 40 tonnes and then recompressed a second and third time monitoring the displacement of the two ends of the compression plate during the cycles. From this graph we can estimate the hysteresis losses experienced in the compression of the material to 40 tonnes. Calculation of the energy lost through hysteresis is the total energy input less the elastic restitution energy restored. For the block featured below the hysteresis losses were 16J for the second cycle and 31J for the third cycle. Please notice that the displacement achieved by the application of 40 tonnes is only about 2.5mm rather than the 5mm elastic displacement experienced during an impact blow.

Figure 5.8 – Hysteresis effects on compression of full-size blocks



These hysteresis losses are not insignificant and could apply to dynamic compaction as well. Furthermore the losses experienced by elastic deformation during dynamic compaction of twice that recorded by quasi-static suggests that the loss of 60J is not out of the question. Noise generation and machine vibration are other possible outlets for energy losses that we are unable to easily isolate and determine the magnitude of.

It is improbable that these losses are present during very early blows as very little elastic deformation occurs initially. From this we can suggest that the impact initially delivers most of the impactor energy into compaction and then as the elastic restitution element increases the energy begins to be lost through impactor bounce and hysteresis within the material.

5.2.5 Assessment of forces for machine design

Now that we have reasonably accurately measured the relative position of the impactor during an impact blow we can make some assumptions and try and extrapolate the values to give estimates for the forces applied. Three different models were proposed elastic, plastic or viscous, and the maximum possible accelerations were calculated for each model. We need to extrapolate these results to yield possible values for a larger impactor being dropped from a greater height.

In the plastic model the acceleration is constant and therefore the resistive force is also constant. We know the 36.8kg impactor when dropped from 200mm penetrated the block during a latter blow by 0.005m. This was not entirely plastic, but lets assume that it was for a moment. Lets also assume that the resistive force of the material does not change during the impact, (consolidation does not occur). The indentation distance now depends on the impactor energy divided by the resistive force $\Delta = \frac{mgh}{F_p}$. So increasing the impactor drop height by a factor of two and the mass from 36.8 to 60 will result in a larger indentation by a factor of 3.26, but the same maximum force.

In the elastic model the material acts like a spring, so the higher impactor energy will change the indentation Δ and therefore maximum force as well. Using the energy of the impactor: $mgh = k\frac{\Delta^2}{2}$ to give $\Delta = \sqrt{\frac{2mgh}{k}}$ and therefore, $F_{\max} = \sqrt{2mghk}$ such that only m and h are variable. Increasing the impactor mass and drop height now increases the indentation by a factor of √3.26 or 1.8. With greater indentation the force is also increased by a factor of 1.8.

In the viscous model the maximum force is dependent on the initial velocity of the impactor which has now been increased by a factor of √2. Calculating the increase in indentation is more complex because it depends on both the increase in velocity and the increase in impactor mass. Hence $\Delta = \frac{mv_0}{k}$ so if the mass increases by 1.63 and v_0 has increased by √2 giving a combined increase of 2.3.

The table below summarises these relationships and extrapolations and leads us to suggest that the range of possible forces that could be applied is between 12kN and 43.2kN. Crudely taking an average for the three models yields a force of 30kN. Converting this to a pressure on the top of the block yields only 0.74MPa.

Table 5.1 – Extrapolation of compaction forces

	Indentation depth Δ (mm)	Maximum Force (kN)
Plastic Model		
36.8kg falling 200mm	5 ‡	12 †
60.0kg falling 400mm	5 × 3.26 = 16.3	12 ×1 = 12
Elastic Model		
36.8kg falling 200mm	5‡	24†
60.0kg falling 400mm	5 × 1.8 = 9	24 × 1.8 = 43.2
Viscous Model		
36.8kg falling 200mm	5‡	24†
60.0kg falling 400mm	5 × 2.3 = 11.5	24 × √2 = 34

‡ - measured value
† - calculated value

We still believe that only a fraction of this force is felt at the mould sides. The literature suggests that the maximum force experienced on the sides of the mould is about 70% of the force applied to the top. We also need to take into account the change in the area that the force is applied to. The top plate is 0.0406m² and the area of the mould side that we wish to design has a width of 0.29m and a height of 0.1m giving a wall area of 0.029m². Assuming the pressure remains the same throughout the material, the force applied on the side F_s can be calculated using $F_t A_t = F_s A_s$ to be $1.4F_t$ which virtually cancels the 70% reduction suggested above. Consequently the force applied to the sides of the mould are approximately 30kN. It is now possible to use this figure to confirm the performance of the mould during the tests and apply the data to machine design.

6 Machine design and development

The motivation for research into dynamic compaction of soil blocks was the need to solve a specific practical problem. The pioneer in the dynamic compaction of soil blocks, A. Groth (1987), went on to make a machine and built houses in Botswana with the finished blocks. Gooding (1993) made significant improvements in the understanding of dynamic compaction and suggested some parameters for machine design. Montgomery (1997) took those parameters and proved the potential of the process in the laboratory for full-size blocks. The experiments conducted during this Ph.D. have further improved the understanding of dynamically compacted production of full-size blocks. The next stage was to develop a suitable prototype for field trials and dissemination. This chapter describes the design methodology, the application of experimental results to machine design, the modifications made to the mould and the way that prototype models of Lego® were used to aid design selection.

6.1 Approach to a production machine

The next stage in the design process was to draw up a set of specifications for the machine. The development of specifications can be a cyclic process with several iterations performed before a final set is chosen. The specifications outlined in this section derive from the understanding of the dynamic compaction process, the required block characteristics and the limitations of machine construction in developing countries. The specifications outlined here can be split up into three different sections. One section deals with the requirements for easy machine manufacture. The second section is concerned with machine operation and use. The final section confirms the desired characteristics of the blocks produced by the machine and discusses methods that achieve them.

The Test Rig that was used for the experiments reported in previous chapters, is different both in form and function from a production machine. The Test Rig required a greater degree of flexibility than is necessary for production. Moreover, machine productivity was not a major concern during experimentation. Whereas a production machine needs a high production rate and its design needs to facilitate that.

6.1.1 Design for ease of manufacture

Machine to have few moving parts **–** Soil can act as an abrasive if permitted to come between moving metallic parts of a machine. It is therefore important to design the machine with as few moving parts as possible. Those moving parts essential to its function should be located where the likelihood of soil contamination is small. The use of rolling element bearings should be avoided, as these will be especially prone to degradation from the presence of soil.

Simple to manufacture and maintain **–** Many developing countries already have a surplus of complex machinery that cannot readily be maintained. We do not want to be adding another machine to this category. The design of the machine should therefore take into consideration the level of technical competence and tooling availability in the area where it is to be used. Through personal experience and communication with other researchers in machine development, a basic level of skill and tooling has been identified. If the machine can be locally manufactured then the necessary maintenance and repair work could also be performed locally. The machine needs to be manufactured using basic power tools such as a manual metal arc (stick) welder and a hand held angle grinder. These two tools can be found in many manufacturing centres in developing countries. Processes such as milling, grinding and even drilling are less common and require more specialised tooling and operator training, hence they should be excluded from the machine production if possible.

Low-cost alternative to hydraulic block press – The aim was to meet the need for a low-cost machine that can produce a comparable block to that from a hydraulically assisted press. The removal of the hydraulic circuit, thick-sided moulds and heavy-duty bearings will dramatically reduce the overall cost of the machine. But this is only the start of potential cost reduction from machine design. Machine tolerances should be as large as possible to remove the need for some jigs and fixtures during machine production. Specific parts that need to be purchased, like hinges, should be kept to a minimum. Wherever possible parts should be manufactured on site reducing the costs and improving the potential for local maintenance. Greater emphasis needs to be placed on the function rather than the form of the machine.

6.1.2 Design for ease of use

Machine portability – We intend to reduce the transportation of soil and therefore promote an on-site building material production unit that utilises very local or on-site soil to make blocks. The machine will therefore need to be easily portable as a complete unit or at the very least it will need to be separable into different parts for moving from one site to another and reassembled with relative ease. This portability requirement of the machine should limit the weight of any single part of the machine to less than 100kg.

Low personnel requirements **–** Block production requires about 2.5kJ of human energy to compact each block. The rate of block production will therefore depend on the power output of the production team. The application of good ergonomic design and team rotation of the most arduous activities will help to maximise the productivity of the team. It is estimated that a team of three persons could operate the machine continuously to produce at least 60 blocks per hour (mean power output of 42W).

Safety and ease of use **–** Dynamic compaction uses a heavy mass that falls onto the top of the block in a mould. Falling masses present a significant hazard and this machine should include adequate protection for its users and for bystanders. Improved safety can be achieved through good working practices, training and built-in safety mechanisms.

6.1.3 Confirming the block specifications

Block size and shape **–** The standards outlined in (Centre for the Development of Industry, 1998) define 6 types of blocks based on the standard parallelepiped shape. Some include perforations, horizontal and vertical indentations. It is not possible to produce every type with a single machine, but some machines can produce several different types with minimal modifications. Such a capacity should be incorporated into the machine design. For example, a block mould can be modified to include a frog relatively easily. Additional features on the block make the machine marginally more complex, but can significantly improve the block characteristics and reduce material consumption.

High-density block production **–** Chapter 3 showed that the achieved density of the compacted block is closely related to its compressive strength. For the purposes of the machine design we have taken the requirement that the block has a wet compressive strength after 7-day curing of 2MPa. In order to achieve this strength a projected dry density of around 2000kg/m³ is necessary. From this density we can estimate other necessary parameters from the experimental results, such as the ejection force.

Versatility in material usage **–** One of the limitations of making blocks out of soil, is the specific range of soils that are should be used. Block compaction via impact can accommodate a wider range of soils, giving it greater site versatility. This can be further enhanced by machine versatility using different impactor arrangements and number of blows.

6.2 Interfacing dynamic compaction with machine design

Now we have a slightly better understanding of the process of dynamic compaction and the mechanisms involved during consolidation. From our experience in the production of small cylinders and full-size blocks we can reduce the ranges of certain machine design parameters. This section derives suitable parameter values from the experimental findings.

6.2.1 Optimisation of energy transfer

Previous research (Gooding, 1993) has already established that the application of neither one or two very high energy blows nor a very large number (e.g. >64) of low energy blows is as effective as a modest number of medium energy blows. This optimisation study was repeated during this research using the small cylinders. The results indicated that over the range investigated (4 to 32 blows), there was a little variation (<1%) in the achieved density and a modest variation (13%) in the resulting strength for the energy transferred.

The results presented in Table 6.1 are for small cylindrical samples that each received a total energy transfer of 157J via a range of impactor masses, drop heights and number of blows. The 7-day W.C.S. and the P.D.D. shown are the average of three samples produced at each arrangement.

Table 6.1 – Small cylinder production using constant energy transfer

Number of Blows	Mass of impactor kg	Drop height m	P.D.D. kg/m³	7-day W.C.S. MPa
4	10	0.4	1980	1.62
8	5	0.4	1992	1.73
8	10	0.2	2017	2.02
16	2.5	0.4	2003	2.26
16	5	0.2	1993	2.17
32	2.5	0.2	2003	1.93

From the table above we can see that for each arrangement the P.D.D. is within ±1% of the target of 2000kg/m³. As the inherent variability of the block density is also around 1%, we could conclude that 6 combinations of impactor mass, drop height and number of blows are equally satisfactory. However, the 7-day W.C.S. varies by slightly more than the inherent variability of 10%. The data indicates that the W.C.S. is highest between 8 to 16 blows and drops off at 4 or 32 blows. We will therefore limit the blow number (n) to between 8 to 16. Extrapolating these compaction parameters to produce a full-size block with a soil mass 40 times larger, needs to be done with care. Direct extrapolation of the energy used would suggest that about 6.3kJ of energy is required. However we know from Chapter 4 that only 1.8-2.4kJ is sufficient to adequately compact a full-size block to 2000kg/m³.

Other considerations need to be applied during extrapolation that affect the ergonomics and productivity of the proposed production machine. Higher values of n will reduce the productivity of the machine as each blow takes 2-3 seconds to apply. We would not want to be lifting a mass of over 80kg, even with some form of mechanical lever. We have found that a 60kg impactor worked sufficiently well during experiments on the Test Rig. The lifting height for the impactor needs to have an upper limit of 400mm otherwise the lifting mechanism becomes un-ergonomic. More complex lifting mechanisms add to the cost and size of the machine. Higher lift heights generate higher impact velocities that may also generate detrimental negative pressure rebound waves.

Using the above upper limits for impactor mass (M = 80kg) and drop height (h = 0.4m) and knowing the maximum energy (2.4kJ) required making a block with P.D.D. of 2000kg/m³, we can calculate the minimum number of necessary blows to be 8. If only 1.8kJ is required then this reduces to only 6 blows. From this we can select suitable design values for the machine to be M = 60kg, h = 0.4m, n = 8-10 to achieve a target of 2000kg/m³. However, if de-lamination occurs at 0.4m then the drop height can be reduced to say 0.3m without increasing the number of blows too significantly (n = 10-14). This gives the machine a degree of flexibility to cope with different circumstances without further modification.

6.2.2 Improved impactor constraint

During the experimentation it was noticed that the top surface of the block had an intolerably high variation in its slope. This was believed to be a result of a combination of the following: the impactor falling at a slight angle, the base plate of the impactor being incorrectly aligned with the base of the machine and poor placement of soil in the mould. Originally the impactor of the Test Rig was a solid block of reinforced concrete with a metal plate at the bottom welded to the reinforcing bar. Initial tests with this indicated that the concrete was suffering from fatigue and beginning to crack. This prompted the change to a cylindrical metal impactor, but in so doing the impactor alignment became more difficult.

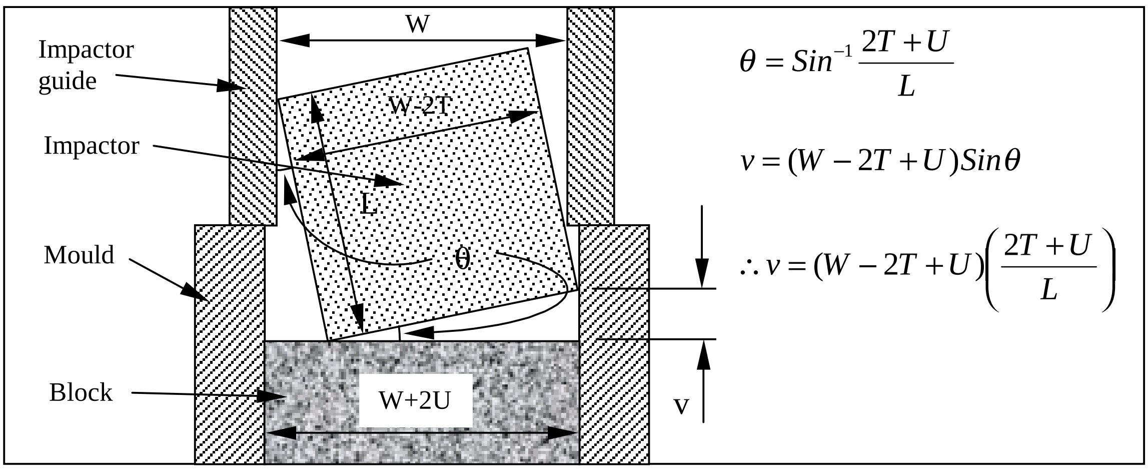
Part of the problem was that a rope pulley some 5 meters above the ground was the only lifting point for the impactor. Even with a 400mm linear bearing at approximately 2 meters off the ground it was virtually impossible to constrain a 60kg impactor successfully over a 400mm free fall. Better impactor constraints could have been implemented, but not without extensive rebuilding of the rig.

The original design for the impactor and impactor constraint could still be acceptable if the concrete was contained within a skin of steel. We need to determine the maximum permissible clearance between the impactor and the impactor guide so that the maximum angle at which the impactor can fall will not produce an unacceptable variation in the block height. This surface variation arises from the impactor rotating in the guide very slightly in both planes of constraint and thus is the sum of two components.

According to (Centre for the Development of Industry, 1998) the acceptable height variation across a block is ±2mm. The interior dimensions of the mould is 0.29m by 0.14m, and clearly to avoid any chance of the impactor hitting the mould sides, the interior dimensions of the *impactor guide* should be smaller than this, say no bigger than 0.288m by 0.138m. The impactor itself will be smaller still in order to give a clearance between the impactor guide and the falling impactor.

The variation (v_x) caused by rotation about the x-axis and (v_y) caused by rotation about the y-axis should sum to not more than 4mm. The diagram shown below indicates how these variations can be calculated using the length of the impactor (*L*), the width of the impactor guide (*W*), the tolerance (*T*) between the impactor and the impactor guide and the tolerance (U) between the impactor guide and the mould. If we take L as 500mm and W_x as 288mm for the x-axis constraint and W_y as 138mm for the y-axis constraint we can calculate the maximum acceptable tolerance to be 1.8mm.

Figure 6.1 – Impactor constraint diagram



These calculations suggest that the impactor should be about 1.8mm smaller on all sides than the impactor guide, i.e. 284.4mm by 134.4mm. For design purposes we will select 285 by 135mm as the dimensions of the impactor.

6.2.3 Ergonomics and productivity

Having now selected a suitable design of impactor we now wish to establish the method of energy transfer to the impactor to lift it to the desired height. The lifting mechanism may provide some mechanical advantage or may even be mechanised to reduce the human effort required. However the energy requirement remains constant and the more complex the mechanism the more potential there is for losses in the system. Ergonomic data {Gee, 1997} suggests that the aerobic energy output of a human is between 70-175 watts (W) for light intensity work, but this value would be lower in a hot environment.

We need to be applying a maximum of around 2.5kJ to each block and we wish to produce at least one block each minute. This equates to a man power requirement of 42W, easily within the range of a single person. However the force required lifting a 60kg mass is 600N, and the mass will be moved through 0.4m during approximately two seconds. This results in a power requirement during the lift phase of each impact cycle of approximately 120W. It is particularly difficult for the human body to apply such force and power and then cease them suddenly, as would be necessary to drop the impactor. This presents a design problem that can have a number of different mechanical solutions:

- Single pulley and two persons pulling on the rope (each exerts 300N through 0.4m)
- Single pulley and one person pulling on a lever attached to the rope (exerts 300N through 0.8m)
- Double pulley system and one person pulling on the rope (exerts 300N through 0.8m)
- Double pulley system and one person pulling on a lever attached to the rope (exerts 150N through 1.6m)
- Double pulley system and motor driven capstan operated by one person

The solution that was selected for the Test Rig was to use a double pulley system and a capstan driven by a high voltage DC motor. This may not be convenient or appropriate for the Production Prototype machine, and this will need to be assessed when the machine design is disseminated. A suitable system will need to be developed locally, that matches the available resources where the machine is going to be used.

6.3 Design of the mould

This section explains some of the changes that were made to the design of the mould as a result of experimental work. The Test Rig was designed with the Production Prototype in mind, so many of the essential parts of the Rig closely resemble the prototype design. Assessment of the mould design used in the Test Rig has indicated where it is in need of modification prior to incorporation in the Production Prototype.

6.3.1 Confirming mould stiffness and strength calculations

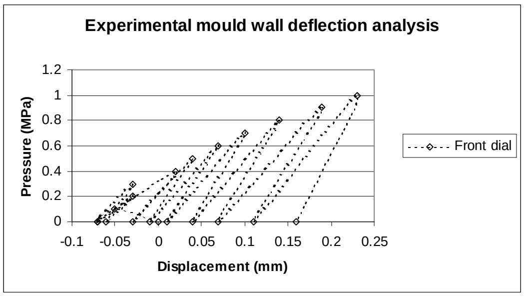
The Test Rig mould was designed on the presumption that a maximum pressure of 5MPa is applied to the top of a block. This reduces to a pressure of approximately 3.5MPa on the side of this block, which equates to a force of 200kN distributed over the sides of the mould during compaction. We further assumed that the mould side could be modelled as a beam with encased ends supports. Adequate mould stiffness had been assumed to be the primary concern and the design deflection was restricted to be less than 1mm. A spreadsheet was drawn up to determine an appropriate arrangement of ribs for the mould sides. The mould side was selected to be 0.29m wide and 0.2m high with a material thickness of 0.005m, its height necessarily greater than the finished block height (90-100mm).

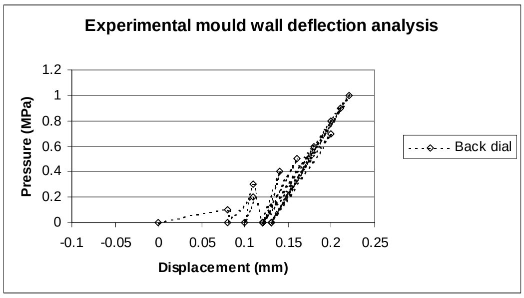
The calculations led to the selection of four 40mm wide ribs placed evenly around the mould. The ribs had the same thickness as the mould sides (5mm) to reduce the number of material sizes required for mould production. Four ribs were selected because they would provide stiffness all over the side of the mould rather than in just one plane. Four was considered to be a low enough number to permit sufficient access to the base of the ribs for welding to be carried out. This mould design led to a calculated maximum central deflection of 0.9mm from a distributed load of 200kN. This mould did not plastically distort during the dynamic compaction tests on full-size blocks.

With the data collected from the dynamic compaction analysis, the mould design detailed above can now be checked. The different dynamic compaction measurements from Chapter 5 suggested that the force delivered to the top of the block during an impact blow could be around 30kN. The maximum stress in the mould had not been calculated previously, and although the mould remained undistorted during block production we still want to double check our calculations with this new force estimate. This force of 30kN was found to give a maximum stress of 109MPa in the mould side, about 40% of the yield stress for M26 steel, giving a factor of safety of about 2.5. We would have preferred 3 or 4, but we can still suggest that this is OK because we have ignored the additional mechanical constraint provided by the joint between the mould side and the base of the mould. This joint will increase the strength of the mould side and therefore raise the safety factor to acceptable levels. This lower force (of 30kN) reduces the side deflection of the mould to 0.13mm or 0.19% of the block width, which is excellent. Now we can be sure that the original mould will perform adequately in terms of both yield stress and deflection displacement.

A further test was conducted by putting the mould in a machine press to test the wall deflection when a measured force was applied to a batch of soil inside the mould. The deflection was monitored using two dial gauges positioned at the midpoint of the block (50mm above the base of the mould). The soil surface was cyclically loaded and unloaded in 0.1MPa increments up to 1MPa. The deflections observed are shown in the figure below. Greater displacement was noticed on the front dial gauge because the front of the mould was the removable section of the mould and would therefore move slightly prior to material deflection. The graphs indicate that the displacement experienced is around 0.22mm at 1MPa. Our calculations had indicated that a force of 30kN (equivalent pressure of 0.52MPa) should yield a displacement of 0.13mm. Therefore our computation (0.25mm/MPa) and experimental (0.22mm/MPa) are in good agreement.

Figure 6.2 – Mould deflection under 1MPa pressure





From these experiments and analysis we can now confirm that the mould design that was developed for the Test Rig is a suitable design for incorporation into the Production Prototype. It performs adequately on strength and stiffness and uses a tolerable amount of metal (11.2kg) to achieve this.

6.3.2 Further mould design developments

Early in the development of the Test Rig it was noted that an integral mould was unacceptable for use with dynamic compaction. In a traditional block press the mould is integral with the machine, having its four sides fixed and the top and bottom plates moving to compress the soil. It was not possible to apply this design to dynamic compaction, as the bottom plate would need to withstand the shock forces applied by the falling impactor. Moreover it would be mechanically very difficult to safely organise block ejection upwards toward the temporarily raised impactor. Finally the benefits of impact would be undermined by having to exert large forces to eject the newly formed block.

A different design of mould was therefore developed to enable compaction to occur onto a flat solid surface and the block removed from the side of the mould rather than from its top or bottom. This design involved breaking the perimeter wall of steel into two parts locked together through some mechanism. Figure 6.2 below illustrates the idea via a plan view of the mould showing the locking mechanisms and the two parts of the mould that come together. Figure 6.3 is a photograph of one half of the finished mould as used in the Test Rig.

Figure 6.3 – Plan view of the two-part mould design

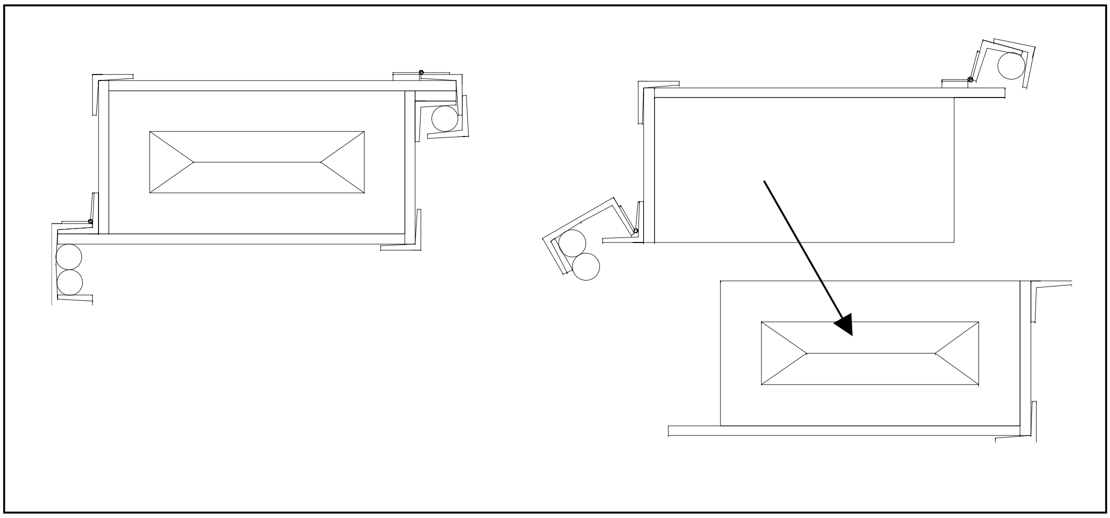
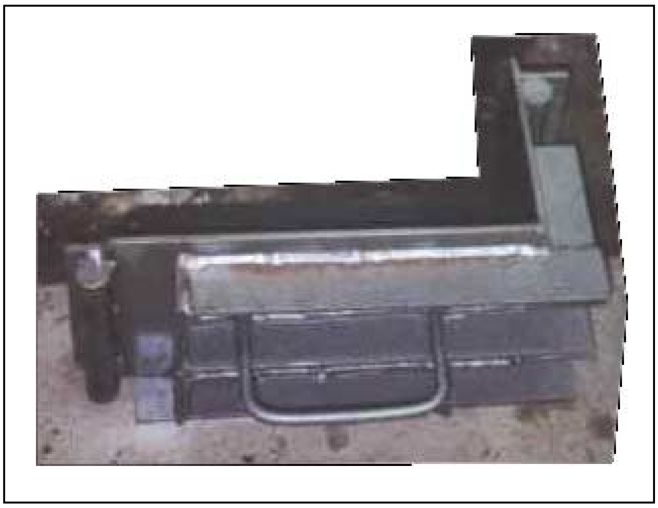


Figure 6.4 – Photograph of the finished front half of the mould

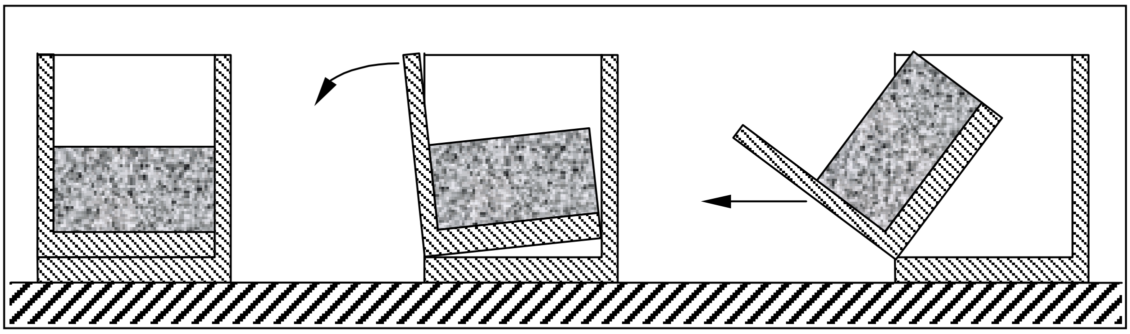


This two-part mould was a novel approach to block ejection and presented some new problems. Overcoming the adhesive forces between the moist compacted soil and the smooth steel mould walls was an exercise that required gentle persuasion. Blocks could only be successfully ejected from the mould by using a slow steady force: impact or jerking action resulted in the block becoming cracked and unsuitable. Furthermore the drag experienced by the block along the sides of the mould would often mar the corner edges upon ejection.

We have established that the initial mould design is adequate for dynamic compaction. There may be problems with fatigue that may need to be addressed during long term testing, but that is outside the scope of this work. The main issue with mould design used in the Test Rig that still needs to be addressed is a better system of block ejection. We know the magnitude of the forces involved in block ejection from our earlier experiments using cylindrical moulds. The ejection forces measured can be converted into the force per unit area of material in contact with the mould. The highest ejection force recorded during these tests, (1.5kN acting on a surface area of $0.0078m^2$), equates to an ejection force per unit area of 200kPa, a value considered excessive.

As viewed from above the Test Rig mould design used two ‘L’ shaped parts that came together to make the rectangular section required for the block, but block edges were getting damaged too easily during ejection. The proposed design for the Production Prototype has one ‘C’ shaped part of the mould fixed to the bed of the machine and a flat front with an attached base that can be drawn out of the machine from the front with the block on top of it. The figure below shows a cross-section through the mould illustrating the proposed method of block removal.

Figure 6.5 – Proposed mould design for improved block removal shown in end elevation



The advantage with this method is that a peeling action is used to remove the block from the back face of the mould as the front part is rotated slightly. Then this is followed by a pulling action to draw the block and front/base out of the rest of the mould. This way the majority of the block is supported by the base throughout the ejection process and should therefore reduce the damage caused to the corners. In order to design this mould we need to know the resistive forces that need to be overcome during the block ejection.

Unfortunately assessing the forces applied to this design of mould is not straightforward. The ejection force necessary for block removal needs to be split into two separate parts. Firstly the adhesion of the soil block to the mould walls usually dependent on the clay content, moisture content and smoothness of the mould walls. And secondly, the frictional shear force between the soil block and mould walls, where the frictional shear force depends on the normal force applied onto the surface.

In quasi-static compaction a large fraction of the force exerted to the top is felt on the sides of the mould. Some of this lateral force remains after the force from the top is removed as elastic strain in the mould. This elastic strain exerts a normal force that generates a high frictional shear force between the block and the mould walls. The process of splitting the mould into two parts releases any elastic strain in the mould and therefore reduces the normal force to almost zero. The remaining force that still needs to be applied is to overcome the adhesion between the compacted material and the mould walls. This adhesive force is most easily overcome through a peeling action rather than a shear or direct pulling action.

Experiments were conducted at different scales to assess the maximum ejection force required to eject a compacted block with density around 2000kg/m³. During these experiments the peak force was recorded as the quasi-static ejection force was applied to the compressed block. This peak force drops off rapidly once the block begins to move within the mould walls (i.e. the adhesive force has been overcome). In the Brepak operation manual (Webb & Lockwood, 1987) one is instructed to "jerk" the block ejection lever downwards to free the block from the side walls, to overcome the adhesive forces perhaps. The data in the table below summarises the recorded ejection forces for the different compression machines used and from the mould wall area calculates the shear friction stress in each occasion.

Table 6.2 – Shear friction stress summary for different compression machines

Compaction device	Mould wall area	Ejection force	Shear friction stress
400kN press	0.0774m²	15kN	195kPa
Brepak (400kN)	0.0774m²	(2kN)‡	(25kPa)
100kN press	0.0023m²	1.1kN	480kPa

‡ - Estimated force applied to block after "jerk" operation

If the same jerk action is used with the design of mould described in Figure 6.4 then the adhesive forces can be overcome and the remainder of the ejection force required will be coming from the ends of the block that are wiping past the ends of the mould. Assuming that the frictional shear stress is the lower 25kPa (despite the open mould) this would result in a maximum ejection force of 625N. Such a force can be applied with a short lever inserted between the front handle and the front of the mould. To ensure that the base of the mould can withstand the hinge moment of the force distributed across the base plate, it has been increased in thickness to 10mm.

6.4 Prototype model exploration and design selection

Six different models were made using Lego® over the duration of the project. Lego® is not the ideal modelling medium, but it did assist greatly in the development of several concepts that were adopted in the final machine design. The models were generally created to explore specific design questions raised. The following paragraphs describe the different versions of the models created and explain why different features were either included or rejected in the final design. Table 6.2 below summarises those features.

Mark I – This model was produced in response to the need for a machine design for the work conducted by Montgomery in 1997. It incorporated a flywheel driven rotary actuator that lifted a lever arm with the impactor attached to it. The rotary actuator lifted the arm upwards raising the impactor until the rotary actuator moved out of the way of the lever arm causing it to fall back onto the surface of the block. A parallel linkage ensured that the impactor was constrained to fall in a vertical plane and directly into the mould. The design was rejected because of the problems in mould and impactor alignment, flywheel and rotary actuator complexity, overall machine size and cost.

Mark II – This design was based on the crank slider mechanism, or inverted piston arrangement. A crank arm at the top of the machine rotated with the assistance of two flywheels. The connecting rod between the crank and the impactor (piston) had an elongated slot on the impactor end to accommodate the different block heights during compaction. The design employed a series of roller bearing guides to constrain the impactor to fall into the mould. Whilst this design was compact and relatively simple, it was top heavy from the flywheels and the roller bearing added unacceptably high levels of complexity and cost.

Mark III – Marks I & II did not include any satisfactory method for mould filling or block ejection. These were not trivial issues and the modelling process identified some problem areas that needed to be considered. The design used in Mark III used a rope and pulley system to lift the impactor that was constrained by running along vertical bars connected to the machine. Two extra features of the design were the inclusion of a mould filling system using side access and the lifting of the mould to eject the finished block. Experience had suggested that the static impactor provided insufficient force to successful de-mould a compacted block, so a locking mechanism to keep the impactor stationary relative to the moving mould still needed to be added. The letterbox-style single-sided access point for mould filling was a nice idea but involved several moving parts in areas of high soil contamination and would have therefore been difficult to manage successfully.

Mark IV – This design adapted Mark III to include a lever mechanism to lift the impactor and a further mechanism to assist the lifting of the mould whilst keeping the impactor position fixed to eject the block. Mould filling was now accomplished by a dual hopper system filling the mould via larger side chutes. Overall this was an attractive system but the complexity of having the impactor guided within another guide (for the mould) was deemed too complex and awkward to use.

Mark V – The real breakthrough with this design was the incorporation of a two-part mould that could be opened from the side permitting the finished block to be removed from the front of the machine. The novel mould design was used with a single point pulley lifting system for the impactor constrained between vertical guides. A single point chute mould filling system was included to improve access to the machine front and to reduce complexity. A safety mechanism was also included in the mould design so that when the mould was open the impactor could not fall into the open mould space. The only complex component of the design was now the mould and this was seen to be a tolerable compromise.

Mark VI – This last design attempted to incorporate into the Mark V design a degree of automation in the lifting and dropping mechanism. The mould design was also slightly modified to make it easier to operate. Instead of an overhead pulley system a more elaborate system of levers and guides similar to those used in Mark I were included. The design was to be a manually assisted counterweighted lever that was lifted upwards with the impactor until a certain point where the impactor would slide off the lever and drop into the mould. The lever could then be pulled back down and re-engage it with the impactor for another lift cycle. The system also included an upper locking off point so that the impactor could be securely placed prior to mould opening.

After the conceptual modelling of the different design ideas, more detailed design could commence. The Lego® models indicated where problems might be encountered and these would need to be addressed in the detailed design. Between the production of two block making test-rigs and the Lego® models it was hoped that the final selected solution would be an acceptable design for dissemination.

Table 6.3 – Summary of features of different prototype models

Mark	**I**	**II**	**III**	**IV**	**V**	**VI**
Lifting Mechanism	Rotary actuator	Lifting crank	Rope & pulley	Lever, rope & pulley	Rope & pulley	Counter-weighted lever
Dropping Mechanism	Rotary actuator	Falling crank	Rope release	Lever release	Rope release	Rotary actuator
Impactor Constraint	Parallel link	Roller bearings	Linear bearings	Sliding	Sliding	Sliding
Soil Filling	N/A	N/A	Side slot	Double side chute	Single side chute	Single side chute
Mould Design	Straight sided	Straight sided	Straight sided	Straight sided	Two Part	Two Part
De-mould Mechanism	N/A	N/A	Egg laying	Lift mould	Open mould	Open mould
Basis of Test Rig				✓	✓	
Basis of Production Prototype					✓	

The final design that was selected was Mark V with the additional modifications made to the mould as detailed in the previous section. Mark VI was considered to be too complex for dissemination at this stage in the design development. Its features could be incorporated at a later date if desired or deemed necessary.

The final Lego® design attempted to incorporate everything necessary for machine operation in one complete unit. However, in the interests of simplicity only the dynamic mechanism and mould have been incorporated in the Production Prototype. This is the minimum necessary to produce blocks and is detailed in the following paragraphs and diagrams. The other features necessary for machine operation can be determined locally, e.g. the lifting mechanism for the impactor. This leaves the machine design open to interpretation, adaptation and improvisation, which is more appropriate for a developing country.

The configuration used in the Lego® model Mark V was adapted to make the Test Rig. Since this design has proved to be successful, the design proposed for Production Prototype is very similar. Modifications were made to the impactor constraint and the mould as a result of the tests conducted on the rig. Minor modifications were also made to the soil loader as it was too small to contain enough soil to make a complete block with a single charge. The impactor guide was also extended upwards to permit greater travel of the impactor, and to provide greater support when the impactor was raised to permit mould filling.

Other details of the design, such as the impactor guide support, are not essential to the functioning of the machine but are advisable additions to the design. Another feature that had been included in the machine design is the safety mechanism within the mould. Whilst the machine will perform adequately without this, its addition makes the machine much safer to use. The pictures of the CAD models shown in the following figures illustrate the different features of the final design.

Figure 6.6a – CAD model of the mould

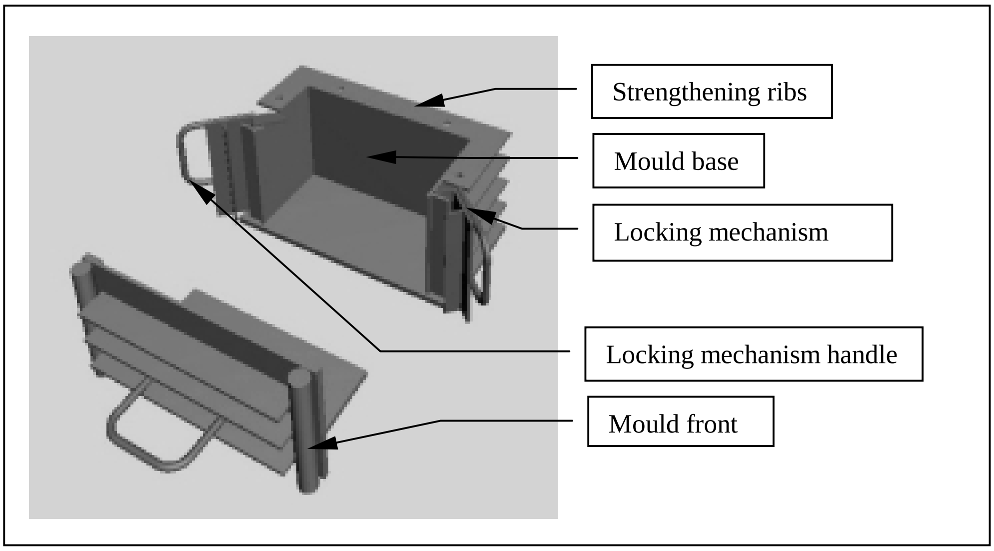


Figure 6.6b – CAD model of the final design

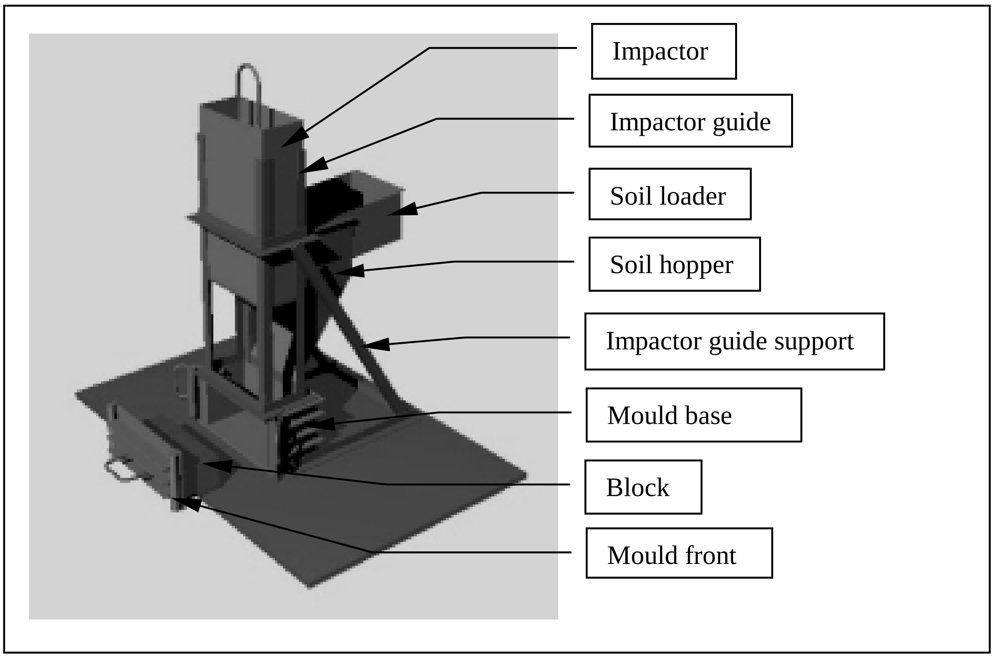
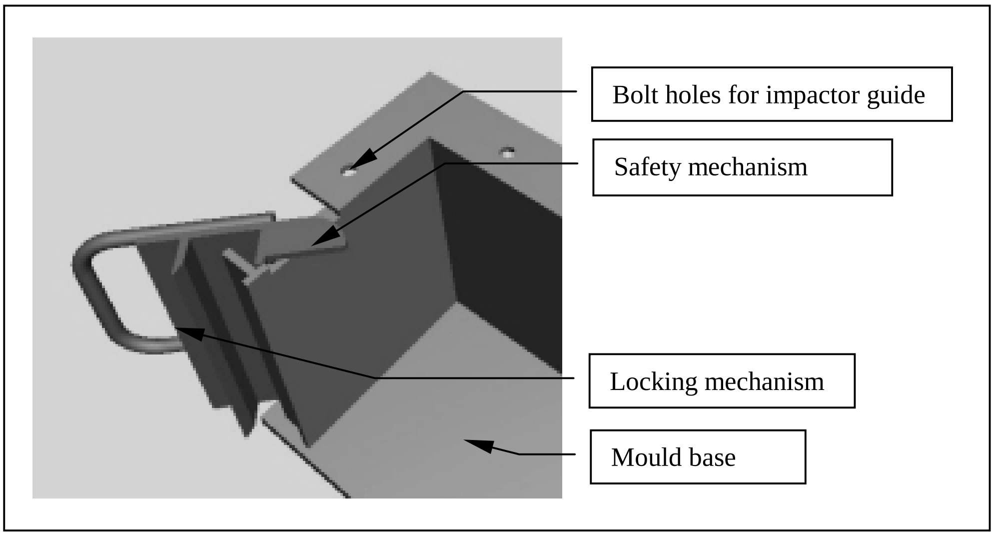


Figure 6.6c – CAD model of the mould safety mechanism



A complete set of drawings was generated from this model clearly showing the different parts of the machine. Scaled versions of these drawings can be found in Appendix D. The dimensional accuracy required for the design was limited to ±0.5mm. The use of the CAD modelling package has helped identify many of the problems in the dimensional accuracy of the design. The different parts are separately modeled and then assembled. The assembly process immediately highlights any problems in the model. This is a very useful tool and gives us greater confidence that the final design will perform as we expect.

6.5 Production guidelines

In the interests of completeness we will now present a brief set of Instructions for Machine Use (for block manufacture). This will help us to also confirm that all major components of the machine are suitably designed and will also be useful for the field testing process described in the following chapter.

6.5.1 Machine foundation selection

The process of dynamic compaction has always required a firm foundation onto which the impact blows can be delivered. It was not known how significant the stiffness of the foundations was until tests were carried out on full-size blocks. Apart from reducing the potential consolidation achieved by each blow, an elastic foundation can also have the detrimental effect of increasing likelihood of material de-lamination, and a more elastic foundation will reflect higher energy shock waves.

The calculated flexural rigidity (EI) (per meter width) of the strong floor used for dynamic compaction experiments was 272MNm^2. A less firm foundation was also created by suspending a 20mm metal plate above the strong floor. This metal plate had a calculated flexural rigidity (per meter width) of 0.133MNm^2. Those blocks produced on the softer foundation were about 10% less dense than those compacted on the strong floor. This is a modest difference in terms of density resulting from a 2000-fold decrease in the flexural rigidity of the foundations. This suggests that the achieved block density is quite insensitive to changes in the foundation flexural rigidity, which is good.

A more noticeable and damaging side effect observed when using the more elastic foundation is that de-lamination of the block is more common. This leads us to suggest that compaction should take place on the most solid and firm foundations available. This may involve the production of a suitable foundation where the machine is set up incurring greater expense, but helping to ensure a better quality block free from any compaction defects.

6.5.2 Block production instructions

These instructions assume that acceptable procedures are already being used for soil preparation and block curing. They list in order the actions that need to be carried out during block production.

1. Lift impactor to locking height, insert bar between the impactor bottom and the upper cross member of the impactor guide, lower impactor onto the bar and ensure it is held safely in position
2. Open and check mould, clear it of any debris and close it again, ensuring the locking mechanism is functioning properly
3. Add the measured quantity of soil to the soil loader
4. Rotate the soil-loader so that the contents fall down the soil hopper and into the mould
5. Lift the impactor slightly and remove the bar
6. Gently lower the impactor onto the surface of the soil
7. Lift the impactor to the desired height, (this can be done visually using points marked on the impactor guide), and drop the impactor onto the surface of the soil
8. Confirm that the impactor did not hit the edge of the mould by listening for the sound of metal hitting metal
9. Continue to apply the required number of blows

10. Lift impactor to locking height, insert bar between the impactor bottom and the upper cross member of the impactor guide, lower impactor onto the bar and ensure it is held safely in position
11. Open mould by releasing the two locking mechanisms
12. Using the ‘rotate and pull motion’ draw the block out of the mould
13. Lift the finished block from away from the front part of the mould
14. Place the block in the curing area
15. Clear the mould of any loose soil and close the mould
16. Repeat items 3 to 15 to make another block
17. An indentation test should be conducted on a number of blocks in each batch to confirm adequate consolidation is occurring
18. Density measurements should also be made frequently as a part of the production feedback

We now have sufficient information and detailed design to share with a collaborator and to begin the process of design dissemination and field trials. The next chapter deals with this next exciting phase of the project.

7 Technological dissemination and field trials

Having established the potential of dynamic compaction theoretically and experimentally we need to conduct field trials of the process in a more representative environment. We wish to assess the Production Prototype for suitability for manufacture in a representative workshop, to test it for short term durability, productivity and ergonomic acceptability, to make any necessary design refinements and to assess the characteristics of produced blocks. The nature and scope of this work cannot be carried out in the UK so an overseas partner was needed to help provide the necessary facilities and environment for field trials to be conducted. In order to improve the readability of this chapter, the first person will be used to distinguish between work carried out by the author and the collaborators.

7.1 Overseas collaboration

Potential partners were identified in Botswana, Ethiopia, India and Tanzania who were all interested in the technology and were prepared to assist in some way. However, it was decided that the Indian partner had the best mix of facilities, expertise and local connections for machine production, development, testing and dissemination. We were extremely fortunate to find a collaborator with strengths in all these areas and who was also willing to collaborate with us without any additional financial assistance.

Our collaborator was Development Alternatives (DA) which is based in India with its head offices located in Delhi. It was established in 1983 as a non-profit corporate organisation and to date has been involved in a number of different areas of sustainable development. It has worked on the application of several technologies in the fields of :

- Construction: Compressed Earth Block, Ferro-cement roof channel, Micro-cement roof tile
- Textiles and paper: Manually driven “powerloom”, Recycled hand-made paper production
- Energy: Biogas electricity generation plants, charcoal briquette production machines
- Water: Portable water testing kits, check-dam construction

The main ethos of DA is to identify locally sustainable practices that generate income and to encourage collaboration between entrepreneurs and local communities in the deployment of these practices. DA has had many successful projects in different regions of India and is always interested to hear of a new technology that may be suitable for sustainable development. When we expressed our wish to collaborate with them in the development of a new type of block machine they were interested enough to accept the challenge and meet the primary practical needs of the project. We were reasonably convinced that the team at DA could manufacture our machine (called ‘Block Impacterre’) and also provide very useful development suggestions from their experience in block machine manufacture. Their connections with other organisations and knowledge of possible sites for building trials would also be very useful for further technological dissemination.

A 4-week visit was set up and suitable funding for the project was obtained. Engineering drawings of the machine were sent out about 4 weeks before the scheduled visit. Two copies of the machine plans were sent under separate cover by mail to DA. The arrival of the plans was confirmed and further communication was done through e-mail.

7.2 Experience with machine

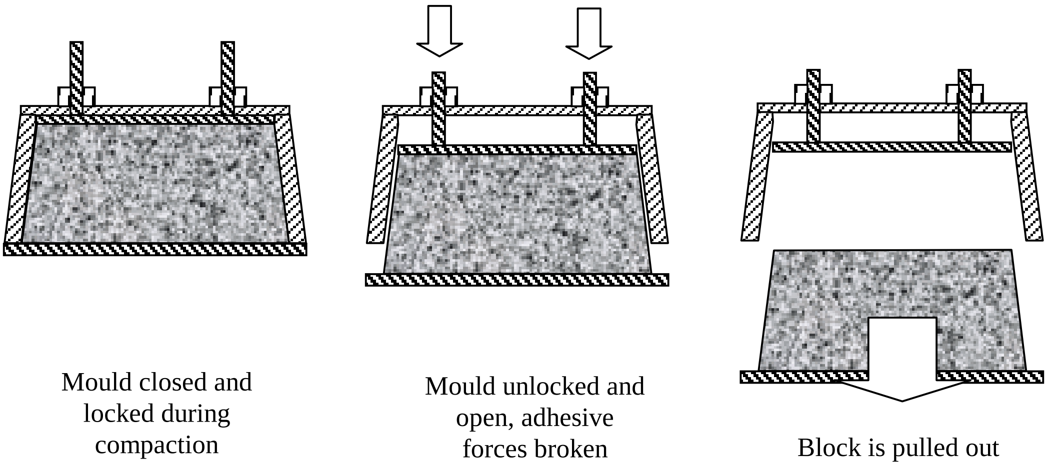
Upon arrival in India contact with DA headquarters was made and from there I went to the workshop where the machine was being produced. I had anticipated that further machine production would be necessary, but it was with great surprise and delight that I found the machine was ready for assembly and testing. This section will summarise some of the machine modifications and block production issues faced during the testing.

The machine was fully built but some of the finer details had been omitted. These were not a major problem, but would need attention eventually. The impactor for example had not been filled with concrete, but with sand. This proved to be a poor solution and the sand was replaced eventually by concrete. The machine foundation was a large metal plate about 20mm thick that unfortunately had protrusions on the underside causing the machine to bounce during each blow. This produced a foundation similar to the soft foundations that were experimented on in the UK. The dimensional tolerance of the machine construction was generally within the ±0.5mm that was recommended. However, the design of the impactor guide was not completely understood and consequently the mould dimensions were very slightly smaller than the internal impactor guide dimensions. This resulted in the impactor hitting the side of the mould on its descent, thereby slightly damaging the mould and causing less energy to be delivered into the block.

Another issue that was immediately brought to our attention was the problem with de-moulding. This was a design fault rather than a manufacturing problem. It was quickly apparent that the proposed de-moulding procedure of the “rotating and pulling” the front of the mould was not going to work successfully. It seemed that the adhesive force between the block and the walls of the mould was higher than anticipated. Several blocks were made and great difficulty was experienced in removing them from the mould, furthermore most had major defects. The proposed solution was to redesign the mould to include a slight taper to assist block ejection. A further proposal was to install a plate mounted on linear bearings on the back face of the mould to push the finished block out the mould rather than try pulling it out. These suggestions were based on the team’s understanding of other block making machines that they had worked on.

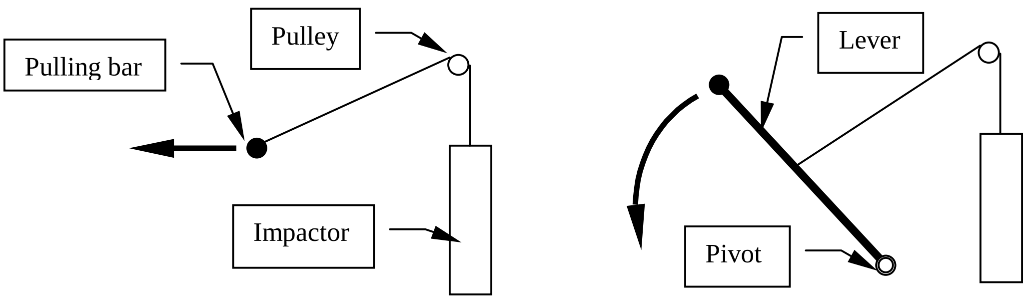
The diagram in the figure below shows a plan view of the modified mould design illustrating the concept of using the taper in the mould to release the adhesive forces between the block and the mould walls. Once the bond is broken then the block can be pulled out from the front of the mould with ease.

Figure 7.1 – Tapered mould design with ejection mechanism (plan view)



The original sand-filled impactor was not weighed, but with the single overhead pulley and rope arrangement three people were needed to lift it. Two people could not exert the necessary force to lift the impactor for a sustained period and so a third person was introduced to help. A bar was attached to the end of the rope to give each operator something to grasp and to ensure that the three operators were operating in unison. The figure shown below illustrates this lifting mechanism and the adapted lever system described below.

Figure 7.2 – Modified lifting mechanism for the impactor



This lifting system was later modified to include a lever that provides a 2:1 lever ratio and reduced the necessary force so that only two operators were necessary. The concrete impactor mass when it had been cast was approximately 66kg. This could still be lifted and dropped using the rope pulley and lever system by two operators.

Generally the concept of dynamic compaction was well received. Our collaborators hoped that the main advantage of this method of compaction would be the improved surface finish of the block. This was one of the major problems with CSSB and initial trials indicated that dynamic compaction offered a superior block finish compared with other block presses. The team was also impressed with the level of compaction achieved by the machine and was hopeful of its potential.

7.3 Machine assessment and block analysis

Once the modifications were made to the machine, block manufacture and testing could be conducted. The main aim of the block manufacture was to test the block characteristics and to compare these with the characteristics achieved by other available block machines. Two other machines were at the DA workshop that could be used to manufacture CSSB, one was a diesel-driven hydraulic press and the other a manual lever-operated press.

7.3.1 The testing procedure

In order to make a meaningful analysis of the blocks produced by the different machines, they all need to made in similar ways and analysed using the same tests. A suitable sample of soil was available on site and this was used to make all of the blocks, we will call it soil-I. A particle size distribution analysis was carried out on soil-I and the results can be found in the Appendix A. A constant quantity of cement (5% by weight) was mixed into each of the batches of soil. During testing the same quantity of water was added to the mix each time, however moisture content analysis indicated a slight difference in the moisture content for each batch of material. The blocks were produced on the three machines using their best machine settings. The pressure applied using the manual machine (Balram or BAL) and the hydraulic machine (Hydraform or HYD) could not be modified, however the level of compaction delivered by the dynamic compaction machine (Block Impacterre or BI) depends on the number of blows applied and the impactor drop height. These two variables were kept as constant as possible during the tests. The blow number did not vary by more than ±1 blow and the impactor drop height unfortunately varied between 350-450mm.

Once the blocks were manufactured they were then carefully weighed to ±25g and measured to calculate their volumes. An indentation tester similar to the one used in the UK had been made in India and this was used on the surfaces of the blocks to determine the uniformity of the blocks. It was also used to gain the necessary data to calibrate the indentation tester from the compressive strength of the blocks. The finished blocks were then cured under plastic for 6 days during which they were sprinkled with water regularly. At the end of day 6, half of the blocks were put underwater, while the other half were placed in an oven for 24 hours at 105°C. At the end of their time in the oven or under water, they were then re-weighed and re-measured to calculate their respective wet and dry densities. All the blocks then had the indentation test repeated on them before they were crushed in a compression machine, noting the maximum load before failure. The results of these tests can be found in Appendix H.

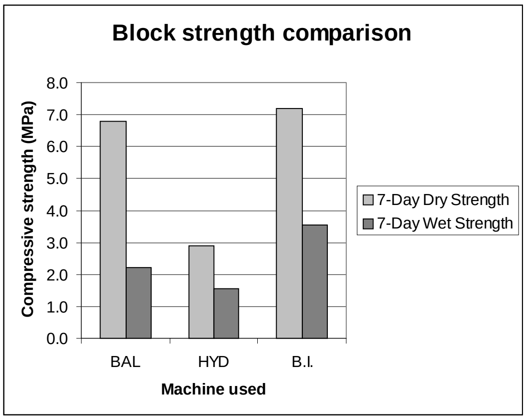
It was not possible to conduct all of the desired testing during the 4-week visit. Consequently, during the testing procedure a member of the team from DA observed the testing and carried out some of the tests personally. That way future tests should be conducted in similar manner and with similar levels of accuracy. The data from these tests were then sent back to the UK for further analysis and inclusion into the thesis report. This data can also be found in Appendix H.

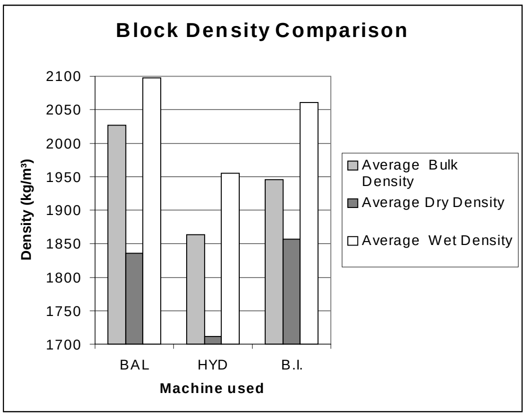
7.3.2 Initial machine comparison

We now wish to compare the output of the three different machines and draw some initial conclusions. The most obvious criterion for block comparison is the wet compressive strength (W.C.S.). We believe that density is a good surrogate for strength and the indentation tester also indicates possible strength of the material, but initially these methods were not calibrated for the soil and the conditions, so we must only use the W.C.S. measure.

The results of the wet and dry compression tests and the density calculations for the bulk (freshly ejected), dry (oven dried) and wet (soaked) density of the blocks from the three machines can be seen in the figure below.

Figure 7.3 – Initial results for comparing block machines





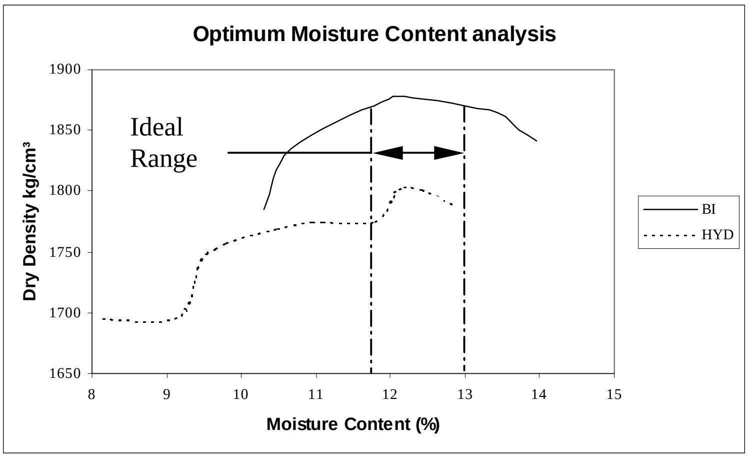
Although these are only the initial test results it can be seen that the BI blocks have performed significantly better than both the BAL and HYD blocks. Part of the reason for the difference is the different moisture contents used in the different machines. Testing of the soil after mixing indicated that the average M.C. for BAL was 12%, HYD was 10% and BI was 8%. The M.C. for BI was lower than desired and we would have expected even better block characteristics with higher water content.

The team at DA estimated that the cost of building Block Impacterre would be similar to the Balram (£500) as it contains similar quantities of material and has similar machining complexity. The block production rates for the BI was estimated to be between 60-100 blocks per hour. Unfortunately the block performance results do not provide conclusive evidence, as there were unavoidable differences in the production using the different machines. Furthermore, we do not know the inherent variability of the production methods used so it is difficult to conduct statistical analysis on these results.

7.3.3 Further block production and testing

After returning to the UK more blocks were produced using the *Block Impacterre* and the *Hydraform* machines in India. Unfortunately the block production and analysis regime was not exactly the same as the initial tests and consequently cross comparison will not be possible. However the larger numbers of block produced do provide a much better data set for statistical analysis. A new mixture of raw material containing soil-I (65 or 63%) and sand (30%) was used during the block production instead of the 95% soil-I used initially. Cement content was either 5% or 7% by mass and the optimum moisture content for *density* was established for both machines experimentally (shown in the figure below). DA further modified the BI machine to include an impactor stop to ensure a constant drop position for each blow.

<u>Figure 7.4 – OMC analysis of two machines</u>



Unfortunately the data received from the blocks made during these tests was not recorded in a way that identified individual blocks adequately. Many blocks had their density calculated and the indentation test conducted on them and several of these were crushed, but there is no obvious way of determining what density the crushed blocks had. Consequently the density and indention tests will have to be analysed in isolation from the W.C.S. results. Further confirmation of the indentation test performance and the density/strength relationship would have been beneficial, but this is not possible with the data received.

The data in the following two tables summarise the results of block production using Block Impacterre and Hydraform. For each machine two groups of blocks were made, Group A (batches 1-4) had 5% cement, Group B (batches 5-8) had 7% cement. The data shows the average P.D.D. and the average indentation diameter (ϕ) for each cement content used. The variation in the moisture content observed during different batch production has also been included for completeness and to check the reliability of the production method. Statistical analysis has been conducted on the data to determine the significance of changing the cement content on the P.D.D. and ϕ.

<u>Table 7.1 – Density and indentation results of Block Impacterre blocks</u>

Group (10/batch)	Cement Content	M.C.		P.D.D. (kg/m³)		Indentation ϕ (mm)	
		Average	C. of V.	Average	C. of V.	Average	C. of V.
A (1-4)	5.0%	10.85%	4.32%	1890	1.92%	17.4	11.46%
B (5-8)	7.0%	10.36%	1.97%	1863	2.04%	18.3	13.23%

Statistical analysis						
Comparing	Standard Deviation of pop'n	Standard Error of means	Standard Error Difference	Difference Of Means	DOM/SED	Significance Normal Distribution
M.C.	0.0047	0.0023				
	0.0020	0.0010	0.0026	0.0048	1.90	94.26%
P.D.D.	36	6				
	38	6	8.31	27.43	3.30	99.90%
Indentation	2.0	0.3				
	2.4	0.4	0.50	0.92	1.86	93.72%

The analysis shows that for Block Impacterre blocks there is an almost 95% probability that the ranges of moisture contents used for the two groups are not from the same population, indicating a change in production method. This helps to account for the 99.9% probability that the P.D.D. achieved are not from the same data set either, (i.e. they are *statistically* significantly different). This is because even a small variation in the moisture content can change the P.D.D. by a noticeable amount. However, the results do not indicate that there is a commercially significant difference in the sets of blocks produced as their P.D.D. are within ±1% of each other.

Table 7.2 – Density and indentation results of Hydraform blocks

Group (10/batch)	Cement Content	M.C.		P.D.D. (kg/m^3)		Indentation ϕ (mm)	
		Average	C. of V.	Average	C. of V.	Average	C. of V.
A (1-4)	5.0%	10.99%	6.09%	1769	2.47%	18.4	9.25%
B (5-8)	7.0%	11.18%	2.76%	1764	0.56%	18.7	5.63%

Statistical analysis						
Comparing	Standard Deviation of pop'n	Standard Error of means	Standard Error Difference	Difference Of Means	DOM/SED	Significance Normal Distribution
M.C.	0.0067	0.0033				
	0.0031	0.0015	0.0037	0.0019	0.52	39.70%
P.D.D.	44	7				
	10	2	7.09	4.63	0.65	48.44%
Indentation	1.7	0.3				
	1.1	0.2	0.32	0.27	0.86	61.02%

By contrast, in the case of Hydraform blocks, the data shows that there is no statistically significant difference between the two groups in the moisture content used. Having established that the production method is not significantly different we can assess the difference in the P.D.D. and ϕ. The results show that there is no statistically significant difference in the P.D.D. or ϕ. From this we can conclude that the additional cement has not affected these two output measures significantly.

The data presented in the two tables above give us more material for machine comparison. It appears that the P.D.D. achieved by the *Block Impacterre* machine is higher than the density achieved by the *Hydraform*. This can be confirmed as statistically and practically significant for the 5% cement batches. Unfortunately there is a statistically significant difference between the moisture content used in the two machines for the 7% cement batches that makes further comparison inconclusive. On average the *Block Impacterre* delivers a 6% increase in density above the *Hydraform* machine. Such an increase in density will also yield an increase in the compressive strength that is of great practical significance (possibly 60%).

Despite the large variation in the indentation diameters (ϕ) recorded, there is a statistically significant difference between the results collected from blocks made by each machine. Again, only the 5% cement batches can be analysed. Not only is the difference in ϕ noticeable *statistically*, the difference also displays the correct phenomenon that a denser block yields a smaller value for ϕ. For *Block Impacterre* a P.D.D. of 1890kg/m^3 yields a ϕ of 17.4mm, whilst for *Hydraform* a P.D.D. of 1769kg/m^3 yields a ϕ of 18.4mm. This demonstrates that the indentation tester provides meaningful results, but its sensitivity to changes in P.D.D. is too small for practical application. We would prefer a sensitivity of about 1mm/25kg/m^3 rather than 1mm/120kg/m^3. This could be achieved by changing the cone angle of the indentation device and/or increasing the weight of the falling mass striking the indentation pin.

We also received results for the strength of the blocks produced by both of the machines. Blocks were crushed in batches of five to determine their 7-day and 28-day W.C.S. for both cement contents used. This data has been summarised in the table below indicating the average W.C.S. for each configuration. In every case the recorded strength of the Block Impacterre blocks exceeded the Hydraform blocks and the average ratio of BI strength : HYD strength is 1.425. However only three out of the four comparisons (5% cement 7-day, 5% cement 28-day & 7% cement 7-day) show a statistically significant difference. The 7% cement 28-day BI specimens have a number of irregularities. Firstly, the C. of V. is unusually high, 23.8% instead of around 10%. Secondly, the blocks are weaker than the 5% cement specimens; they should be stronger with more cement. Thirdly, their measured density is about 2.5% lower then the 5% cement blocks, a difference that could result in a 25% loss of strength. These factors point to some error in the block production for the batches used to gain the 7% cement 28-day strength data for BI.

Table 7.3 – Strength analysis of blocks produced by different machines

Batch (n=5) Soil-I with 30% sand Machine	Cement Content (%)	Curing Period Days	Average W.C.S. MPa	C. of V. (%)	Ratio of W.C.S BI / HYD	Statistically Significant >95%
Block Impacterre	5.0	7	4.6	10.1		
Hydraform	5.0	7	3.2	6.4	1.44	Yes
Block Impacterre	7.0	7	5.6	10.2		
Hydraform	7.0	7	3.8	2.9	1.46	Yes
Block Impacterre	5.0	28	6.4	10.3		
Hydraform	5.0	28	3.7	1.0	1.72	Yes
Block Impacterre	7.0	28	5.8	23.5		
Hydraform	7.0	28	5.4	5.8	1.08	No

Statistical analysis						
Comparing W.C.S.	Standard Deviation	Standard Error	Standard Error Difference	Difference Of Means	DOM/SED	Significance Normal Distribution
5% cement	0.47	0.21				
7-days	0.21	0.09	0.23	1.41	6.13	>99.99%
7% cement	0.57	0.25				
7-days	0.11	0.05	0.26	1.77	6.85	>99.99%
5% cement	0.66	0.30				
28-days	0.04	0.02	0.30	2.69	9.09	>99.99%
7% cement	1.36	0.61				
28-days	0.31	0.14	0.62	0.43	0.69	50.98%

The strengths recorded here are noticeably higher than those experienced during initial tests using only soil-I, (see graphs in Figure 7.3). It would appear that the addition of sand to the soil significantly improves the strength of the material as it increased the average BI block 7-day strength from 3.6MPa to 4.6MPa and the average HYD block strength from 1.7MPa to 3.2MPa. The inherent variability of the strength (±10%) seems to be consistent with the previous tests so we can presume that the tests have been conducted in a satisfactory manner. It is a shame that the BI blocks inherent variability is so much higher than the Hydraform bocks, but this is probably due to the very different compaction mechanism employed during consolidation. The variability of ±2% on density and ±10% on strength is an acceptable range for experimental analysis.

The strengths demonstrated by the BI blocks are all much higher than the original specifications of 2MPa strength after 7-days. If we assess these blocks using the Indian standard for masonry walling (28-day W.C.S. of 3.5MPa) we find that they all comply. Furthermore, all of the BI blocks achieve this strength after only 7 days. The general trend exists that the 28-day W.C.S. is about 25% higher than the 7-day W.C.S. In order to achieve a 3.5MPa strength at 28-day one wishes to achieve at least 2.5MPa at 7-days. Again, all of the blocks above comply with this. Recent work on CSSB durability conducted by (Kerali, 2001) shows that 28-day W.C.S. of 3.5MPa is an acceptable standard to ensure adequate durability. Therefore we can say that these blocks have adequate strength and durability.

7.4 Dissemination overview

Apart from gaining the practical experience in the field with the technology in a more representative environment, the field trials also gave us more experience in the process of technology dissemination. From an academic perspective this is valuable to us as it helps to provide and promote technology in ways that will be more accessible and appealing to those who will derive most benefit from it.

One of the big problems with technological dissemination is transferring information. This is a problem for both the end user communicating their real needs and desires to the research body and also for the researcher sharing their work with those would most benefit from their research. During the visit to India there was the opportunity to speak directly with those who were more actively involved in supplying technological solutions to local needs. From these conversations it was found that current CSSB technology was not cost effective and with a poor track record CSSB was considered a second-rate material. Those who can afford to, build used burnt brick rather than CSSB. This poor perception of the material was not obvious from the information available to us at the start of the project. We knew that CSSB didn’t perform as well, but we didn’t know that it already had a bad name, which would be difficult to overcome.

The technological advancements that were proposed during the project all took a poor material and made it slightly better. There was no major breakthrough in terms of material performance, only cost reduction and material savings were realised. These improvements alone would not be enough to sway people from a tried and trusted less expensive material (burnt brick) to CSSB. A switch seems much more likely if burnt brick was much scarcer as it is in the region of Bangalore in India. CSSB technology has been much more successful there due to a lack of appropriate alternatives.

Getting people to change their minds about techniques or products requires clever marketing and communication. However, successful marketing tools are not easily applicable to this level of technology and propagated in areas of poor communication. The team at DA suggested that an effective method of getting new technology accepted is to initiate it with a middle-class group rather than with the very poor. Something perhaps initially expensive, but desirable and cutting edge, which is a modest challenge with a low-cost mud brick machine. The Hydraform machine presents a high tech solution at a high price and leaves no possibility of local artisanal replication or even maintenance. However, if the Block Impacterre was initially marketed as a high tech solution that provides the same sort of product as the Hydraform machine but for a much lower price, that would make it more attractive. As the new-technology is accepted by the more prestigious and wealthy it becomes much more desirable to the poor. Now, the low complexity of the Block Impacterre machine leaves the potential for artisanal copying and maintenance if some marketable high-tech features are left out, without greatly compromising the machine performance.

Another issue that was not obvious from the information available in the literature was that environmentalists in India consider the use of soil for building a bad practise. Whilst in the United States earth building has become more acceptable for environmental reasons, in India the use of soil for building reduces the soil that could be cultivated for crops. Instead environmentalists recommend the use of waste materials such as fly-ash from power stations. This material has been used successfully with the Hydraform machine to make blocks with a compressive strength of 30-40MPa. This impressive product is only hampered by the poor availability of the fly-ash material in areas away from the power stations, and the fact that Indian annual production of fly-ash is only a small fraction of the annual tonnage of walling materials used in the country.

We have demonstrated that the dynamic compaction technique is superior to quasi-static compression both during experiments and field trials. With the information gained from the overseas collaboration there are still outstanding questions about the economic viability of the new technology, its acceptance among the low-cost building market and it's environmental implications. For the purposes of this research we are only able to investigate the economic viability of the technology compared with the other machines available. The next chapter analyses machine productivity and block viability.

8 Commercialisation and feasibility study of impacted blocks

We now have details of the characteristics and performance of dynamically compacted blocks and the machine used to make them. We wish to quantitatively compare these with those of a suitable competitor to conduct a feasibility analysis. This chapter is split into three sections. The first investigates the features of the machines used in India to produce CSSB (including the Block Impacterre developed during my research). The second section normalises the material produced by each machine to a suitable standard and compares the requirements of each machine to produce adequate CSSB. The final section summarises these findings and suggests how feasible the dissemination of *Block Impacterre* into the block-making market would be.

8.1 Machine analysis and comparison

We wish to compare the machines used during the production of CSSB using a number of different criteria. Three criteria that are easily assessed for each machine are its respective cost, production rate and energy consumption. A further criterion for assessment is the wet compressive strength of the material that each machine produces with similar raw materials. Our data for this criterion is divided up into the results gained when I was in India, (which included the investigation of three machines) and the comparative analysis of only Block Impacterre and Hydraform conducted by collaborators after I returned to the UK. The Hydraform machine in its various models is a market leader in several countries. Due to problems with some of this data only the 7-day wet compressive strength has been included in this analysis.

The table below presents the summary of the different criteria for machine comparison. Some of the values presented have been estimated in good faith using the experience of the collaborator. Several values for the 7-day wet compressive strength have been presented. These highlight the difference between the machines when sand is added to the soil mix for CSSB production and also the improvement obtained.

Table 8.1 – Machine comparison

Machine name	Balram	Hydraform	Block Impacterre
Origin	India	South Africa	Warwick University
Machine cost (2001) £1 = Rs 70	Rs 36,000 £510	Rs 200,000 £2860	(Rs 35,000) (£500)
Production rate Blocks/hr	120-180	50-100	(60-100)
Energy consumption per block	0.75kJ	180kJ	3.7kJ
Average 7-day W.C.S with 5% cement, 95% soil (3 samples)	2.2MPa	1.6MPa	3.6MPa
Average 7-day W.C.S with 5% cement, 65% soil, 30% sand (5 samples)	N/A	3.2MPa	4.6MPa
Average 7-day W.C.S with 7% cement, 63% soil, 30% sand (5 samples)	N/A	3.8MPa	5.6MPa

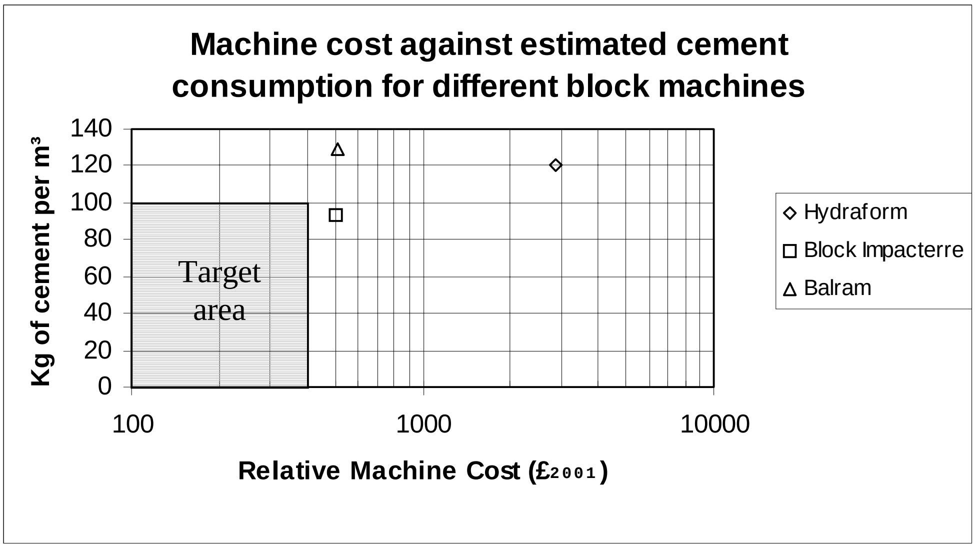
Figures in parenthesis are projected

Block Impacterre performs adequately in terms of production rate and energy consumption, and delivers better results on W.C.S. due to increased material consolidation. *Hydraform* has an excessive energy requirement because of the diesel engine used to power the hydraulic press, something that neither *Balram* nor *Block Impacterre* suffer from being manually operated machines. The assessment indicates that on the combined basis of machine cost, production rate and W.C.S. the Block Impacterre machine is the best performer.

A comparison of different machines was presented in Chapter 2 and identified the target area of a lower-cost machine with a lower cement demand. By applying the data from Chapter 7 and making certain assumptions, we can recreate that comparison for the three machines used in India. We have attempted to normalise the CSSB block performance from each machine by adjusting the cement content. The initial test using only soil and 5% cement indicated that 5% cement was sufficient for *Block Impacterre* to achieve 3.5MPa. But both *Hydraform* and *Balram* required a boost in cement to comply with this standard. Consequently we have selected a cement content of 7% for these two machines. From these figures and the block densities that each machine produces we can calculate the cement requirement per cubic meter of walling material using only soil and cement.

The graph below presents this data for the three machines. Whilst *Block Impacterre* does not achieve the target (<100kg cement per m^3 and < £400 per machine) originally suggested, it is the closest machine to it by a fair margin. Furthermore, a 20% reduction in the cost of *Block Impacterre* would bring the machine into the target area. *Balram* requires a significant reduction in its cement demand as well as a similar reduction in machine cost. *Hydraform* is too far away from the target area to be a possible contender.

Figure 8.1 – Re assessment of CSSB machines using recent data

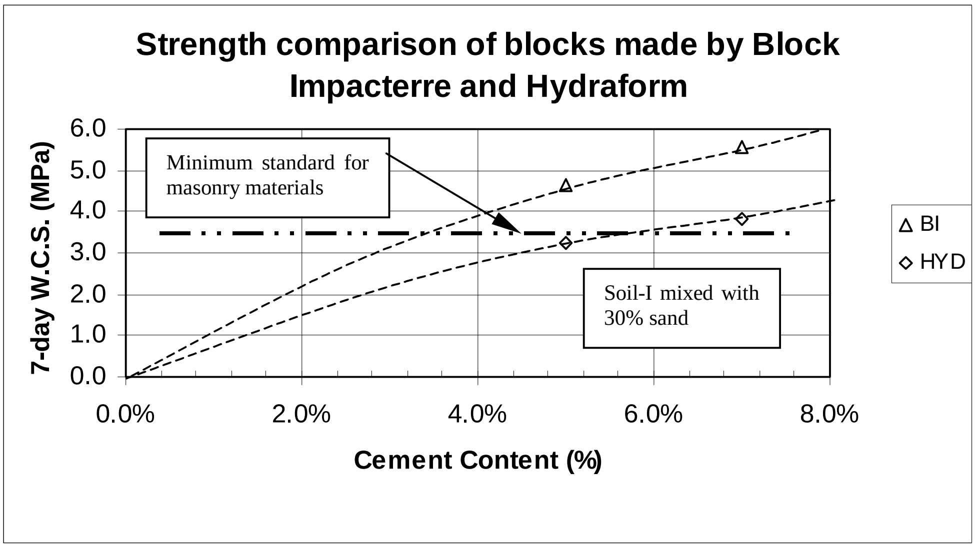


8.2 Material analysis for performance

It has been necessary to use material performance as a criterion for machine comparison. We now wish to assess the material produced by these CSSB machines to see their potential for meeting the needs of low-cost building materials in tropical regions. The building regulations in India require a minimum 28-day wet compressive strength of 3.5MPa for all masonry walling. (Kerali, 2001) recommends a similar figure for durable CSSB walling blocks. We will adopt this as the comparative strength of the materials produced by different machines instead of the 2MPa chosen for the analysis in Chapter 2.

The graph shown in the figure below projects the 7-day W.C.S. for different cement contents for the Hydraform and Block Impacterre. We know the general relationship between strength and cement content from our experience with CSSB, so we can approximately predict from the results gained in India the necessary cement content required for either machine to produce CSSB with a 7-day W.C.S. of 3.5MPa. The material used here includes 30% sand, permitting a reduction in the cement content of the CSSB.

Figure 8.2 – Block strength using different machines and cement contents



From this graph we can project that a block manufactured from soil-I with 30% sand by *Block Impacterre* requires about 3.5% cement to achieve the desired strength, whilst *Hydraform* needs closer to 5.5%. If sand is not used then the machines require 5% and 7% cement respectively to achieve the same standard. We can now use these figures to conduct an analysis of the CSSB production using these two machines and the different raw materials available.

In Chapter 2 and Chapter 3 a comparison of low-cost building material and CSSB variants was presented. Using a similar strategy and assumptions it is possible to calculate the respective energy consumption and cement requirement for walling made with these machines and different raw materials. Any requirement for sand in the CSSB production makes on-site production marginally less suitable. The analysis indicates that *Block Impacterre* uses less cement and less energy than *Hydraform* per square meter of walling produced. Furthermore, with *Block Impacterre*, the addition of sand to the soil yields a cement demand of less than the original target of 15kg/m². Even without the sand the analysis indicates that a normal CSSB requires only 17.5kg/m² instead of the originally calculated 18.7kg/m² in Chapter 3.

Table 8.2 – Comparison of CSSB walling material from different machines

Machine	**Dimensions ($l \times b \times h$)**	**Note**	**Energy**	**Cement**	**Suitability for production**	
					'Locally'	**On-site**
	Mm		MJ/m^2	kg/m^2	Ranking	(1 = best)
Block Impacterre (with 30% sand)	290 × 140 × 90	1	309	14.0	2	3
Hydraform (with 30% sand)	215 × 221 × 116	2	529	22.8	3	4
Block Impacterre (without sand)	290 × 140 × 90	3	273	17.5	1	1
Hydraform (without sand)	215 × 221 × 116	4	466	29.0	2	2

Notes

0. All cement is assumed to have been transported 100km, all sand transported 25km and all material has a 7-day W.C.S. of 3.5MPa.
1. High-density (1925kg/m³) solid blocks manufactured on-site from local soil mixed with 30% sand and 3.5% cement, laid with 10 mm of soil/cement mortar (20% cement) and no render.
2. Medium-density (1775kg/m³) solid blocks manufactured on-site from local soil mixed with 30% sand and 5.5% cement, dry stacked with no external render.
3. High-density (1925kg/m³) solid blocks manufactured on-site from local soil mixed with 0% sand and 5% cement, laid with 10 mm of soil/cement mortar (20% cement) and no render.
4. Medium-density (1775kg/m³) solid blocks manufactured on-site from local soil mixed with 0% sand and 7% cement, dry stacked with no external render.

8.3 Feasibility study

The analysis in the previous two sections have demonstrated that dynamic compaction of CSSB using the *Block Impacterre* provides reasonable reductions in machine and walling cost without compromising the material properties of the finished block. It is possible to financially analyse these machines if we assume their respective working life to be 4 years for 240 days a year. We know the productivity of the machines from earlier analysis and their cement demand for adequate material properties. Taking a cost of cement as £0.05/kg (Rs 3/kg) we can draw up a projected total cost/m² of material produced using each machine.

Table 8.3 – Projected costs of walling for machines during lifetime

Machine and material	Cost (£)	Block production	Cost/m² (£)
Hydraform (soil)	2860	768000 blocks (1)	1.60
Block Impacterre (soil)	500	461000 blocks (2)	0.91
Hydraform (soil, 30% sand)	2860	768000 blocks (1)	1.29
Block Impacterre (soil, 30% sand)	500	461000 blocks (2)	0.74

(1) Average of 100 blocks per hour for 8 hours a day, 240 days a year for 4 years.
(2) Average of 60 blocks per hour for 8 hours a day, 240 days a year for 4 years

It appears that the *Block Impacterre* delivers a 40% reduction in walling costs compared to the *Hydraform* with or without sand in the blocks. The running cost of the machines and their respective maintenance has not been included in the analysis. Assuming that both machines use the same labour force the *Hydraform* also requires diesel to operate and more complex maintenance than the *Block Impacterre*. These factors would push the running costs of the *Hydraform* up higher. From an economic viewpoint, it appears that the process of dynamic compaction is superior to high-pressure quasi-static compression.

Proving that the process and machine is cost competitive is unfortunately insufficient to claim its future success in the market of low-cost walling. Certainly such significant savings would prompt many to try the new technology and this is most certainly what we hope would happen. However, the relative success of the *Hydraform* machine compared to the *Balram* may indicate that a mechanised machine is looked upon as better investment for entrepreneurs. The *Block Impacterre* was never designed to be part of the high-tech market, but was designed to fit into cottage industry type of environment. The problem with a low-tech and low-cost machine may be the limited resources for marketing, advertising and dissemination that is available for that type of product.

9 Conclusions and recommendations

The research conducted during this Ph.D. has increased our understanding in several areas related to the production of low-cost building materials. This chapter aims to draw together the different aspects of block manufacture, machine design and analysis of impact compaction. It is split into four sections and each section makes recommendations for further work. The first section details the investigation of CSSB potential and acceptability. The second section extends this to include the work conducted on dynamic compaction of CSSB. The third reports the analysis of the mechanisms behind dynamic compaction and the implications for CSSB production. The final section includes the findings of overseas field trials and feasibility study of the application of dynamic compaction to CSSB block production in a developing country.

9.1 Acceptability and potential of CSSB

The use of earth as a building material is well known and its highly variable performance is well documented. Specific research in the production of building blocks over the years has revealed certain features of the material. Generally un-rendered low-cement (<6%) and low-density (<1800kg/m^3) CSSB exhibit an unacceptably low tolerance to humid conditions and will deteriorate during less than 10 years. This deterioration is typically in the form of spalling of the exterior surface.

By increasing the cement content, and/or the density, the stability of the material is greatly enhanced and becomes more acceptable for use in humid areas.

Performance of the CSSB is usually defined by its wet compressive strength. Literature suggests that a CSSB that exhibits a 28-day wet compressive strength of 2MPa is considered a first class material (Houben & Guillaud, 1994). Previous research into CSSB production indicated that for suitable soils doubling the cement content more than doubles the wet compressive strength. It has also been noted that a 10% increase in density can approximately double the wet compressive strength. Higher cement content generates a more expensive material, but if a significant increase in density could be realised then the cement content could actually be reduced without harming the performance of the material. From this the potential of improving the performance for stabilised soil and more specifically CSSB has been identified.

There are other reasons for promoting the use of CSSB for low-cost walling. Several sources have indicated that environmentally unacceptable practises are currently involved in the delivery of low-cost walling. The use of clamp-fired brick and river sand is proving to be unsustainable in the long term and resources necessary for their production are becoming scarcer. CSSB has been identified as a more environmentally and socially acceptable alternative, if its production and use is carefully controlled.

An assessment of several different types of building materials indicated that high-density CSSB was the only material that consumes a modest amount of cement and has a low-energy requirement in its production and subsequent erection. Different block variants were also explored and these indicated that taller, interlocking and hollow blocks had the most promise for further reducing the cement requirement of the material.

Testing of the finished CSSB involved a crushing test using sophisticated equipment and destroying the CSSB. The CSSB achieved the desired strength of 3.5MPa after 28-days. Unfortunately a suitable non-destructive test was still unavailable. The relationship between the density of the compacted material and the compressive strength showed potential as a method of strength estimation. Unfortunately this technique requires a careful production regime and calibration for each mixture of soil, cement and water. As this was not seen to be a practical solution an indentation tester was developed instead, which provided an indication of densification and the block strength. Readings could be taken at any point in the production cycle as a comparative measure of block performance. It could indicate large changes in block characteristics without the need for destructive testing. The limits of its accuracy were about 0.5MPa, so only changes in block characteristics larger than this could be identified. Improved resolution of the device could be achieved by modifying the shape of the indentation tip.

9.2 Performance of dynamic compaction

The application of dynamic compaction to stabilised soil was initially investigated using small cylindrical samples. The results of the investigation were then extrapolated to the production of full-size blocks. Trials at the smaller scale confirmed previous findings that dynamic compaction is more energy efficient in consolidating soil than quasi-static compression. This finding was not however replicated during the production of full-size blocks until their moisture content was raised from 6% to over 9%.

It was originally assumed that the lower moisture content suggested by the cement literature was more acceptable than the less handlable blocks with the higher water content. Trials indicated that the concrete literature did not apply to the production of stabilised soil and a much higher water content could be used successfully. The experimental evidence showed that adjusting the moisture content to achieve the greatest consolidation was an effective method of improving the strength of the material. This finding was consistent with the “optimum moisture content” defined in the soils literature.

Previous CSSB production guides suggest relatively basic systems for monitoring and controlling the moisture content, (drop test) and the quantity of material used to produced each CSSB, (volumetric measurement). After seeing how a small variation (2% drop) in the water content can have a pronounced effect on the block properties (50% drop in strength), it would be much better to implement an improved system of moisture control. Careful weighing is difficult to achieve in the field, but a small-scale compaction test may provide a suitable alternative for moisture content optimisation. The material measurement currently done by volume should be done by mass instead. The calculated density of the compacted material can be used as a method of feedback after block ejection only if an adequate system for mass measurement is incorporated in block production. Such systems are currently not in use, but the development of them would be beneficial in the field of CSSB production.

The process of dynamic compaction was investigated and compared with the equivalent quasi-static compression process. Both methods of consolidation exhibited similar levels of variation, ±1% on density and ±10% on wet compressive strength. The low variability of density was helpful in determining the optimum parameters for the dynamic compaction process. Earlier studies indicated that a heavy impactor with a low velocity provided marginally more effective material consolidation. Experiments conducted on small scale confirmed that this was true, and these findings were extrapolated to full-size. Full-size blocks required a higher energy transfer and therefore required a more massive impactor. A practical upper limit for the impactor mass was suggested as 80kg and 60kg was found to yield satisfactory results. The drop height of the impactor also had an upper limit because of the generation of a destructive reflection wave at high impactor velocities. It was found that a drop height of 400mm was acceptable on a firm foundation, but if a reflection wave was generated the drop height should be reduced.

Analysis of the potential of CSSB indicated that different block variants had potential for further reducing the cement and hence cost of the walling material. Two of these variants were tested using dynamic compaction, the cement-rich skin block and the hollow block. Unfortunately neither variant was very successful. The strength of the hollow block variant was too low for building purposes (0.6MPa). This was because the material density at the bottom of the block flange was quite low, as indicated by the indentation test. The potential of this type of block produced via dynamic compaction is uncertain and requires further work. Different methods of soil placement in the mould and different shaped voids may help deliver a more uniform density throughout the block. The method for producing the cement-rich skin variant was too complicated for normal block production. A more automated system could be implemented but this variant yields only a small saving in cement and other variants hold more promise.

9.3 Analysis of impact compaction

We perhaps understand a little bit more about the process of soil consolidation after the experiments conducted on dynamic compaction. We have been able to determine that the closer arrangement of particles is primarily due to shock wave propagation through the material. The presence of free-water in the mixture aids this process significantly as the particles slide over one another into closer proximity with each other. The impact compaction drives out air from the mixture in this re-arrangement process at such a rate that some of the air becomes temporarily trapped and possibly compressed. This trapped or compressed air very slightly hinders further compaction, but not at a level of practical significance.

Careful monitoring of displacement during the impact cycle itself has revealed some interesting phenomenon in the process. From these findings it is possible to conclude that the method of dynamic compaction follows some sort of combined elastic-plastic-viscous model. Initially this model demonstrates high plastic deformation, low energy loss and low hysteresis. As the compaction process continues the model changes to include increased losses, a decreasing plastic component and the development of an elastic component. Towards the end of the possible compaction the plastic component diminishes to zero, high energy loss is experienced through hysteresis and a small elastic component causes impactor bounce.

The discovery that the losses are so significant may seem like a disadvantage to the dynamic compaction process. The most effective compaction occurs during the very first few blows, but it is the latter blows that give the desired levels of compaction necessary for a high-density CSSB. These extra blows only require a small amount of extra time and therefore it seems reasonable to apply many of them. The large number of blows (>30) used for monitoring the impactor motion would not normally be experienced during normal block manufacture. Typically the number of blows would be smaller than this and could be stopped when further blows delivered little extra compaction rather than none at all.

The relative displacement was monitored during the impact cycle to gain an impression of the mechanisms behind impact compaction. An accelerometer was also used to determine the point of maximum acceleration, as the methods used for measuring the accelerations experienced by the impactor were unreliable beyond 25g. Calculations from the stopping distance applied to plastic, elastic and viscous models showed that the maximum accelerations experienced by the impactor were around 65g.

These accelerations experienced by the impactor were converted into forces delivered by the impactor, which were in turn experienced by the mould. These forces were a small fraction of the forces required for high-pressure quasi-static compression yet achieved similar and often better levels of consolidation with a tolerable number of blows (<16). A 10MPa hydraulic press would deliver about 400kN of force to the top of a block. Dynamic compaction was delivering about 30kN of force and also only for a short duration of 5-10ms.

The application of this dynamic lever in the compaction of CSSB provides another significant advantage over quasi-static in the potential use of a two-part mould rather than the very stiff 1-part mould required for quasi-static pressing. Significant forces (5kN) are required to overcome block friction and adhesion to the mould walls. These would be much reduced if the mould could be split into two parts after compaction. Dynamic compaction offers this possibility as the forces on the mould are much smaller and are very short in duration. The need of a large mechanical lever to eject the finished block would partially negate the advantages that dynamic compaction presents in machine design. Dynamic compaction removes large levers and forces applied to the block, so this novel method of mould design needs to be incorporated to maximise the advantages of the production process.

Trials of the two-part mould in the laboratory were a reasonable success. Blocks had to be ejected with care because of the adhesion between the block and the corner of the mould. The adhesion often resulted in small corners being left behind in the mould. After laboratory testing of the two-part mould and the necessary analysis for mould strength and stiffness from the data on the impact forces, the mould was slightly modified. The modified mould design used a different method of block removal, but the same two-part arrangement that was successful with dynamic compaction.

9.4 Feasibility study of dynamically compacted CSSB

The experiments in the laboratory indicated the potential success of dynamic compaction over quasi-static compression of CSSB. These findings needed to be confirmed in a more representative environment and consequently an overseas collaborator was sought out. A development organisation in India was the most promising of the different possible collaborators that we had correspondence with. They were able to comply with our requests for machine manufacture, future testing and possible dissemination if deemed successful.

Using a modest amount of machine tooling and expertise, a dynamic compaction block maker (Block Impacterre) was built in India. The team estimated the cost of machine production and block productivity was predicted following initial trials. Blocks produced were tested whilst I was there and further testing was conducted after I returned to the UK. The testing measured the performance of the material produced by Block Impacterre and a high-pressure competitor (Hydraform). From these findings it was possible to make a comparative analysis of walling material produced by both machines.

The study indicated that Hydraform could produce walling during its projected lifetime at a rate of £1.60/m^2. Block Impacterre with its improvement in consolidation and hence lower cement requirements and much less expensive machine could produce walling costing only £0.91/m^2. This represents a monetary saving of over 40%. The addition of sand to the soil mix makes the added cement even more effective and therefore enables a greater reduction in cement whilst still maintaining the same strength. The addition of 30% of sand reduces the cost of walling to £1.29/m² and £0.74/m² for the Hydraform and Block Impacterre respectively, still representing a saving of over 40% using the dynamic process compared with quasi-static compression.

This analysis indicates that the process of dynamic compaction offers a significant incentive to switch from an alternative low-cost building material to dynamically compacted CSSB. However, such a switch is not guaranteed just because of a significant monetary incentive. Other factors need to be considered in order to assess if this material and this process will be successful. Communication with Indian building advisors suggests that CSSB is not cost effective if burnt brick is available. Typically a burnt brick in Delhi costs about £0.04 whilst a CSSB costs about £0.08. The 40% reduction in cost will go some way to bringing these two materials closer together, but the perceived poor performance of CSSB will drive away some potential customers. However, in areas where burnt brick is not available and consequently CSSB has a reasonable following already, the dynamic process of CSSB production should have significant potential. Further trials are necessary in such an environment along with pilot schemes to help disseminate the technology into communities with a need for low-cost walling.

The work carried out during this Ph.D. has been directed towards meeting a need for low-cost housing. It initiated with a concept that impact compaction provided a better alternative to material consolidation than slow squeezing. The research has taken this initial premise through stages of conceptualisation and laboratory testing during which time the understanding of the process has been improved. Test results confirmed the initial potential of the process and this prompted machine design and dissemination for use in developing countries. The additional collaboration and field trials conducted on the finished machine have been very promising. Finally the analysis of the potential of the machine and its product has given the Ph.D. all aspects of research through to product dissemination. This has made the work both interesting and highly rewarding and will hopefully be useful to others in the future.

Bibliography

1. Agarwal, Anil (1981) *Mud, mud,* IIED, London.

2. Agevi, E. (1999) "Technology Dissemination in Kenya", *Development Alternatives Newsletter,* Vol: 9 (**11**), 7-9.

3. Akroyd, T. N. W. (1962) *Concrete properties of manufacture,* Pergamon Press.

4. Ashurst, John and Ashurst, Nicola (1989) *Practical Building Conservation - Brick, terracotta and earth,* Gower Technical Press.

5. Bedford, A. and Fowler, W. (1997) *Statics,* Addison-Wesley.

6. Bee, Becky (1997) *The Cob Builders Handbook.*

7. Bondarik, G. K. (1967) *Dynamic and Static Sounding of Soils in Engineering Geology,* Israel Program for Scientific Translations, Ltd. Jerusalem, Israel.

8. Borland (1987) *Turbo C: The complete reference,* Osborne/McGraw-Hill.

9. Bramer, Brian and Bramer, Susan (1997) *C for Engineers,* Arnold.

10. British Standards Institution. (1971) *Specification for Portland cement (ordinary and rapid hardening).* BS 12: 1971. BSI. England.

11. British Standards Institution. (1990) *Stabilised materials for civil engineering purposes. Part 1. General requirements, sampling, sample preparation and tests on materials before stabilisation.* BS 1924: Part 1: 1990. BSI. England.

12. British Standards Institution. (1990) *Stabilised materials for civil engineering purposes. Part 2: Methods of test for cement-stabilised and lime-stabilised materials.* BS 1924: Part 2: 1990. BSI. England.

13. British Standards Institution. (1990) *Method of test for soils for civil engineering purposes. Part 1. General requirements and sample preparation.* BSI 1377: Part 1: 1990. BSI. England.

14. British Standards Institution. (1990) *Method of test for soils for civil engineering purposes. Part 2. Classification tests.* BS 1377: Part 2: 1990. BSI. England.

15. British Standards Institution. (1990) *Method of test for soils for civil engineering purposes. Part 4. Compaction-related tests.* BS 1377: Part 4: 1990. BSI. England.

16. Britto, A. M. and Gunn, M. J. (1987) *Critical State Soil Mechanics via Finite Elements,* Ellis Horwood Limited.

17. Carvill, James (1997) *Mechanical Engineer's Data Handbook*, Butterworth-Heinemann.

18. Cedergren, H. R. (1967) *Seepage, Drainage, and Flow Nets,* John Wiley & Sons.

19. Centre for the Development of Industry (1998) *Compressed earth block standards,* CRATerre - EAG/CDI.

20. Chan, W. W. J., & Wu, C. M. L. (2000) "Durability of concrete with high

cement replacement", *Cement and Concrete Research,* Vol: 30 865-879.

21. Coates, R. C., Coutie, M. G., and Kong, F. K. (1994) *Structural Analysis,* Chapman & Hall.

22. Craig, R. F. (1997) *Soil Mechanics,* E & FN Spon.

23. Davies, Ian (1978) *Shelter after disaster,* Oxford Polytechnic Press Oxford, UK.

24. Department of Engineering (1996) *Engineer Data Book,* University of Warwick England.

25. Easton, David (1996) *The Rammed Earth House,* Chelsea Green England.

26. Flugge, Wilhelm (1967) *Stresses in shells,* Springer Verlag New York.

27. Gee (1997) *The Health and Safety Factbook,* Gee Publishing Limited.

28. Ghali, A. and Neville, A. M. (1997) *Structural Analysis: A Unified Approach,* Spon.

29. Gooding, D. E. M. (1993) *Improved processes for the production of soil-cement building blocks,* Unpublished doctoral dissertation: Warwick, University of Warwick.

30. Gooding, D. E. M., & Thomas, T. (1995). *The Potential of Cement-Stabilised Building Blocks as an Urban Building Material in Developing Countries,* ODA Report, UK.

31. Guillaud, Hubert, Joffroy, Thierry, and Odul, Pascal (1995) *Compressed earth blocks - Volume 2: Manual of design and construction,* GATE Germany.

32. Hanson. (2001) Hanson Bricks [Web Page]. URL www.hanson-brickseurope [Accessed: 2001].

33. Harr, M. E. (1962) *Groundwater and Seepage*, McGraw Hill.

34. Hausmann, Manfred. R. (1990) *Engineering Principles of Ground Modification*, McGraw Hill Sydney.

35. Head, K. H. (1980) *Soil Laboratory Testing - Volume 1*, Pentech Press, London.

36. Head, K. H. (1980) *Soil Laboratory Testing - Volume 2*, Pentech Press, London.

37. Holmes, Stafford and Wingate, Michael (1997) *Building with Lime,* Intermediate Technology Publications London, UK.

38. Houben, Hugo and Guillaud, Hubert *Earth Construction: A Comprehensive Guide,* Intermediate Technology Publications, UK.

39. Houben, Hugo, Rigassi, Vincent, and Garnier, Philippe (1994) *Compressed earth blocks: production equipment,* Centre Devt Industry (CDI) Brussels, Belgium.

40. Hutchings, Howard (1995) *Interfacing with C*, Butterworth-Heinemann Ltd.

41. Ingles, O. G. and Metcalf, J. B. (1972) *Soil Stabilisation Principles and Practice,* Butterworths, Sydney.

42. International Labour Office (1987) *Small-scale manufacture of stabilised soil blocks*, International Labour Office, Geneva.

43. International Labour Office (1990) *Small-scale brickmaking*, International

Labour Office, Geneva.

44. Jones, Norman (1997) *Structural Impact,* Cambridge.

45. Jones, R. (1962) *Non-Destructive Testing of Concrete,* Cambridge University Press Cambridge.

46. Keable, Julian (1996) *Rammed earth structures: a code of practice,* IT Publications London, UK.

47. Keddie, James and Cleghorn, William (1980) *Brick Manufacture in Developing Countries,* Scottish Academic, Edinburgh.

48. Kelkar, Vasant S. and Sewell, Robert T. (1987) *Fundamentals of the analysis and design of shell structures*, Prentice Hall, USA.

49. Kerali, A. G. (2001) *Durability of Compressed and Cement-Stabilised Building Blocks*, University of Warwick, UK

50. Kumar, A. (1999) "Sustainable building technologies for mass application", *Development Alternatives Newsletter*, Vol: 9 (**11**), 1,3,4.

51. Lambe, T. William and Whitman, Robert V. (1969) *Soil Mechanics*, John Wiley and Sons.

52. Lea, F. M. (1970) *The chemistry of cement and concrete*, Edward Arnold.

53. Mallet, Geoff (1991) *Fatigue of Reinforced Concrete*, HMSO.

54. Mbumbia, L., Mertens de Wilmars, A., & Tirlocq, J. (2000) "Performance characteristics of lateritic soil bricks fired at low temperatures: a case

study of Cameroon", *Construction and Building Materials*, Vol: 14 121-131.

55. Miller, G. A., & Azad, S. (2000) "Influence of soil type on stabilization with cement kiln dust", *Construction and Building Materials,* Vol: 14 89-97.

56. Montgomery, D. E. (1997) *Design and Realisation of a Test Rig to Research the Production of Full Size Dynamically Compacted Soil-Cement Blocks,* University of Warwick, UK.

57. Neville, A. M. (1995) *Properties of Concrete,* Longman.

58. Norton, J. (1997) *Building with Earth,* Intermediate Technology Publication London.

59. Orchard, D. F. (1979) *Concrete Technology,* Applied Science Publishers.

60. Parry, J. P. M. (1979) *Brickmaking in developing countries,* Building Research Establishment, UK.

61. Parsons, A. W. (1992) *Compaction of soils and granular materials: A review of research performed at the Transport Research Laboratory,* HMSO, UK.

62. Popovics, Sandor (1998) *Strength and related properties of concrete: A quantitative approach,* John Wiley & Sons.

63. Powers, Treval C. (1968) *The properties of fresh concrete,* John Wiley & Sons.

64. Rigassi, Vincent (1995) *Compressed earth blocks - Volume 1: Manual of production*, GATE.

65. Sabir, B. B., Wild, S., & Bai, J. (2001) "Metakaolin and calcined clays as pozzolans for concrete: a review", *Cement and Concrete Composites,* Vol: 23 (**6**), 441-454.

66. Scott, R. A., & Pearce, R. W. (1975). "Soil compaction by impact" *Ground treatment by deep compaction,* Institute of Civil Engineers, London.

67. Shan, H.-Y., & Meegoda, J. N. (1998) "Construction use of abandoned soils", *Journal of Hazardous Materials,* Vol: 58 133-145.

68. Smith, Edward W. and Austin, George S. (1996) *Adobe, pressed-earth, and rammed-earth industries in New Mexico,* Authority of State of New Mexico, USA.

69. Thermalite. (2001) Thermalite [Web Page]. URL www.thermalite.co.uk [Accessed: 2001].

70. UNCHS/ILO (1995) *Shelter provision and employment generation,* Habitat/ILO Nairobi.

71. United Nations (1964) *soil-cement it's use in building,* United Nations, New York.

72. United Nations (1972) *The manufacture and utilisation of Portland cement,* United Nations.

73. Walker, P. (Aug/1999) "Bond Characteristics of Earth Block Masonry", *Journal of Materials in Civil Engineering,* Vol: 11 (**3**), 249-256.

74. Webb, D. J. T. (1988) *Stabilised Soil Building Blocks,* University of Newcastle-

upon-Tyne, UK.

75. Webb, D J T and Lockwood, A J (1987) *BREPAK Operators Manual*, BRE, Watford, UK.

76. Wolfs-kill, L. A., Dunlop, W. A., & Callaway, B. M. (1965) *Handbook for Building Homes of Earth*.

77. Yogananda (1999) "Stabilised Mud Block Homes in Bangalore", *Development Alternatives Newsletter*, Vol: 9 (**11**), 13.

Appendices

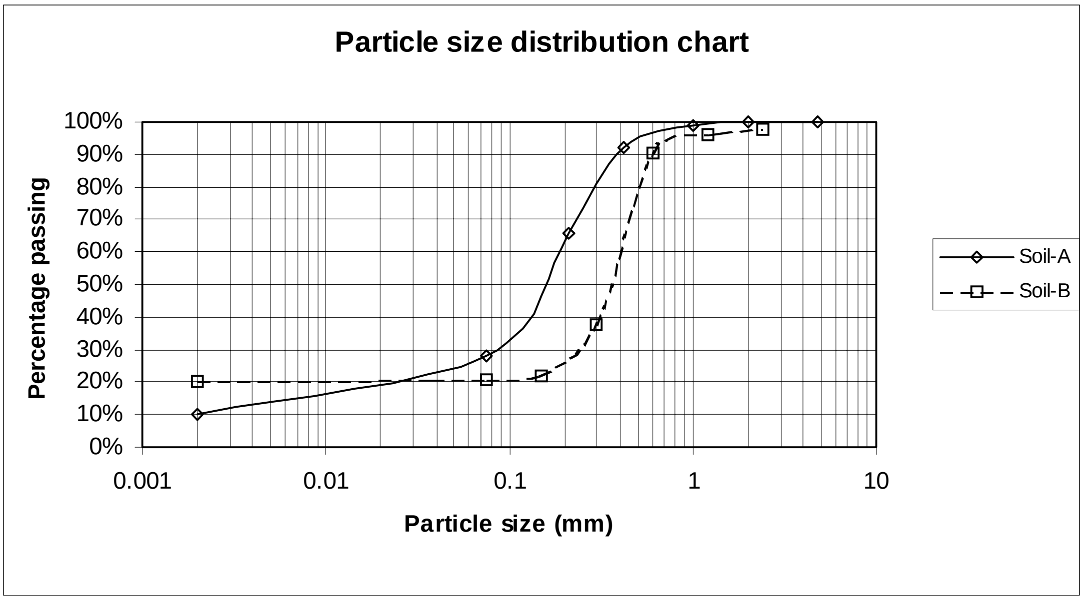
Appendix A - Analysis and production of soils used

Analysis of soil-A, soil-B and soil-I

Approximate values for the three main soil fractions for each soil

Particle	Size (mm)	soil-A (%)	soil-B (%)	soil-I (%)
Clay	0.000-0.002	15	20	7
Silt	0.002-0.063	8	1	32
Sand and gravel	> 0.063	77	79	61

Graphical results of sieve analysis



Mixing of soil-B

The raw materials for the laboratory soil was generally mixed dry to ensure that the cement present in the mix would not start to hydrate until a known amount of water was added. This enabled large quantities of soil-B to be weighed and mixed waiting for further processing. Typically the dry mix was weighed out on a set of Avery Scales rated to 50kg to the following proportions:

Oven dry builders sand	15.2 kg
Kaolin clay grade-E	3.8 kg
Cement	1.0 kg
Total	20.0 kg

This results in a 4:1 mixture of sand to Kaolin clay and an overall percentage of cement equal to 5% by mass.

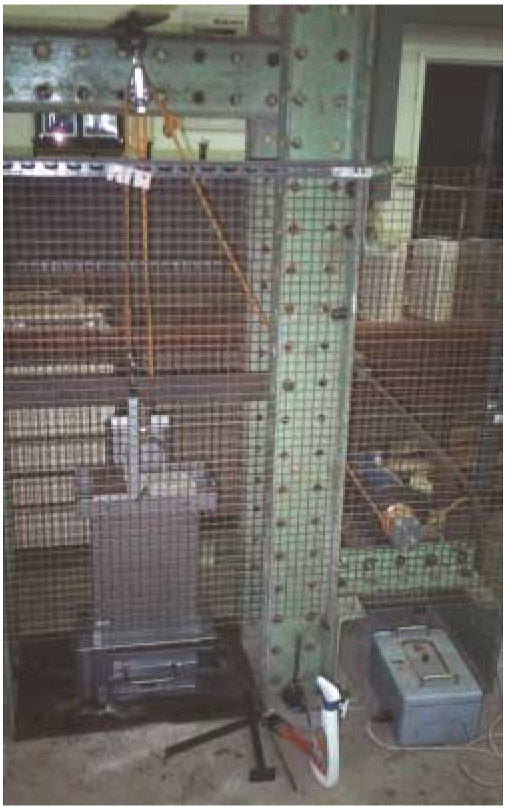
The dry quantities were placed into plastic bags and mixed together when necessary in a steel drum with a lockable lid. Due to the high level of fines present in the mixture the sealed drum was necessary to keep the dust levels down and to also to ensure that the fines added remained in the mix rather than becoming airborne.

Water was added to the weighed out samples in a batch size of one and the water was mixed using a Hobart mixer. The mixer motor had been condemned, consequently the mixing paddle has to be turned by hand via the main crank shaft and gearbox through an attached handle. The handle was rotated at a rate of around 60 revolutions per minute and the mixture was mixed for at least 3 minutes. The mixture had always taken a uniform colour at this time and further mixing was not only exhausting, but also seemingly unnecessary.

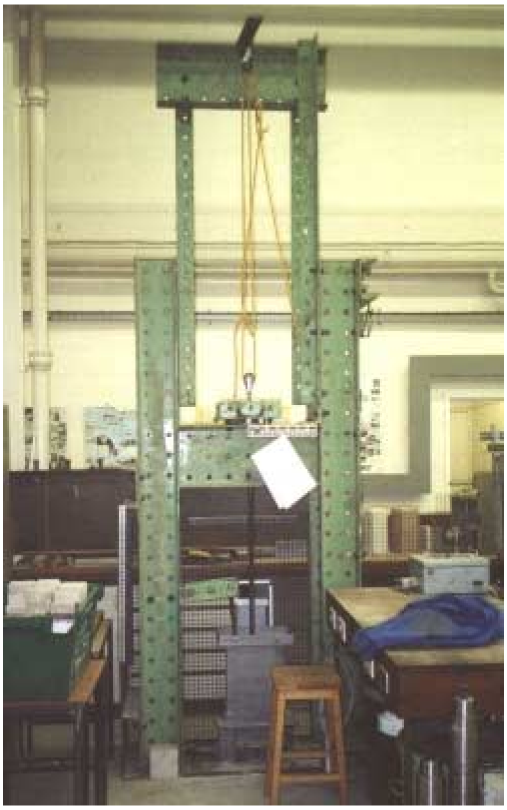
Some tests required soil-B without any cement present and the quantities were mixed together in the electric drum mixer and mixed with water in the correct quantity. The finished mix was then placed into plastic bins and used when necessary ensuring the mixture was never left open any longer than necessary to extract a quantity of soil.

Appendix B - Photographs

The laboratory Test-Rig



Original test rig (cement impactor)



Modified test rig (steel impactor)

Two part mould in use, illustrating the problem with block ejection

The Production Prototype

Production prototype

Overhead pulley added

Impactor lifting mechanism added

Ejection mechanism added

New cast concrete impactor

Overview of prototype

Blocks

Early compacted block

First block in India

Improved block ejection

Finished blocks

Minor block defects

Hydraform blocks

Different types of walling seen in India

Balram block walling (self build)

The alternative

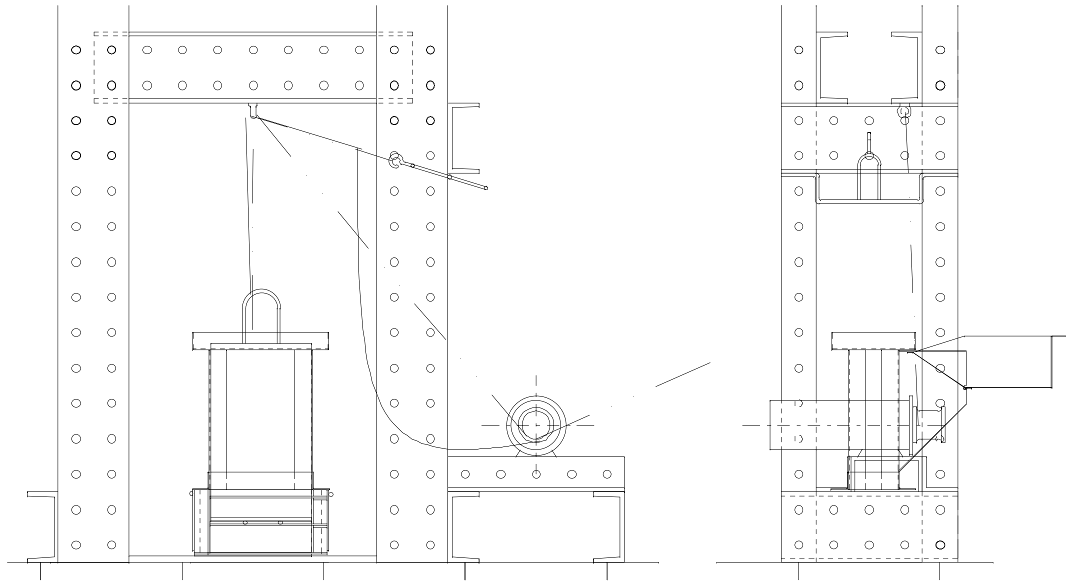
Bricks from waste material

Poorly fired bricks

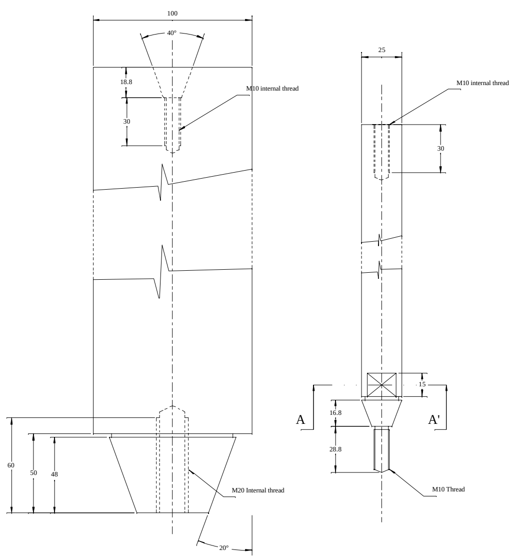
Stone cladding over bricks

Hydraform house

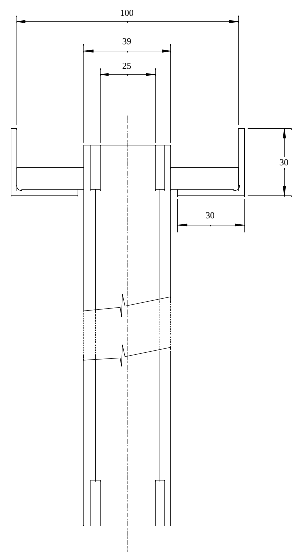
Appendix C - Test Rig design



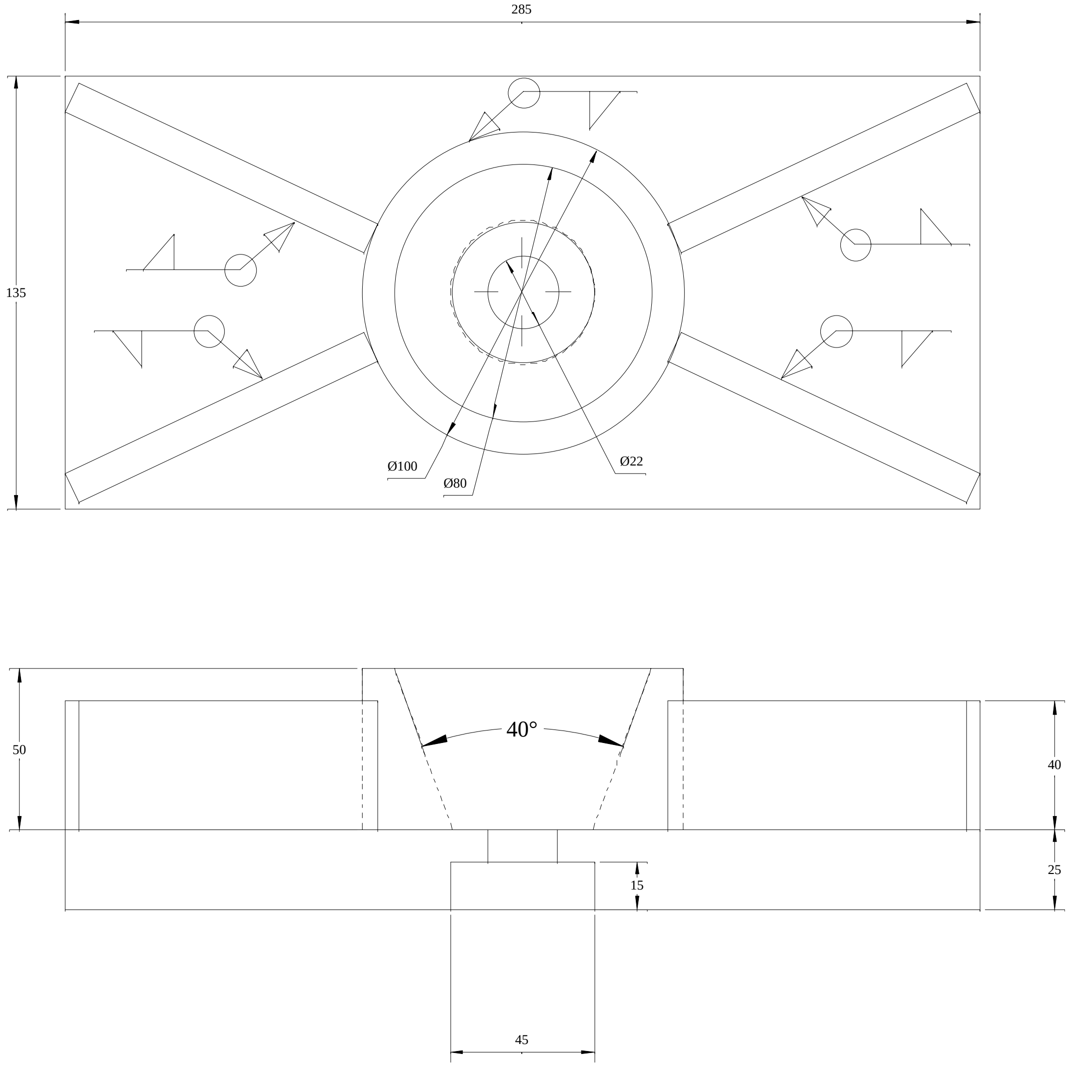
Overview of initial set up of Test Rig using the concrete impactor lifted via a pulley and motor driven capstan.



Metal impactor and lifting rod details



400mm linear bearing



Metal impactor base details

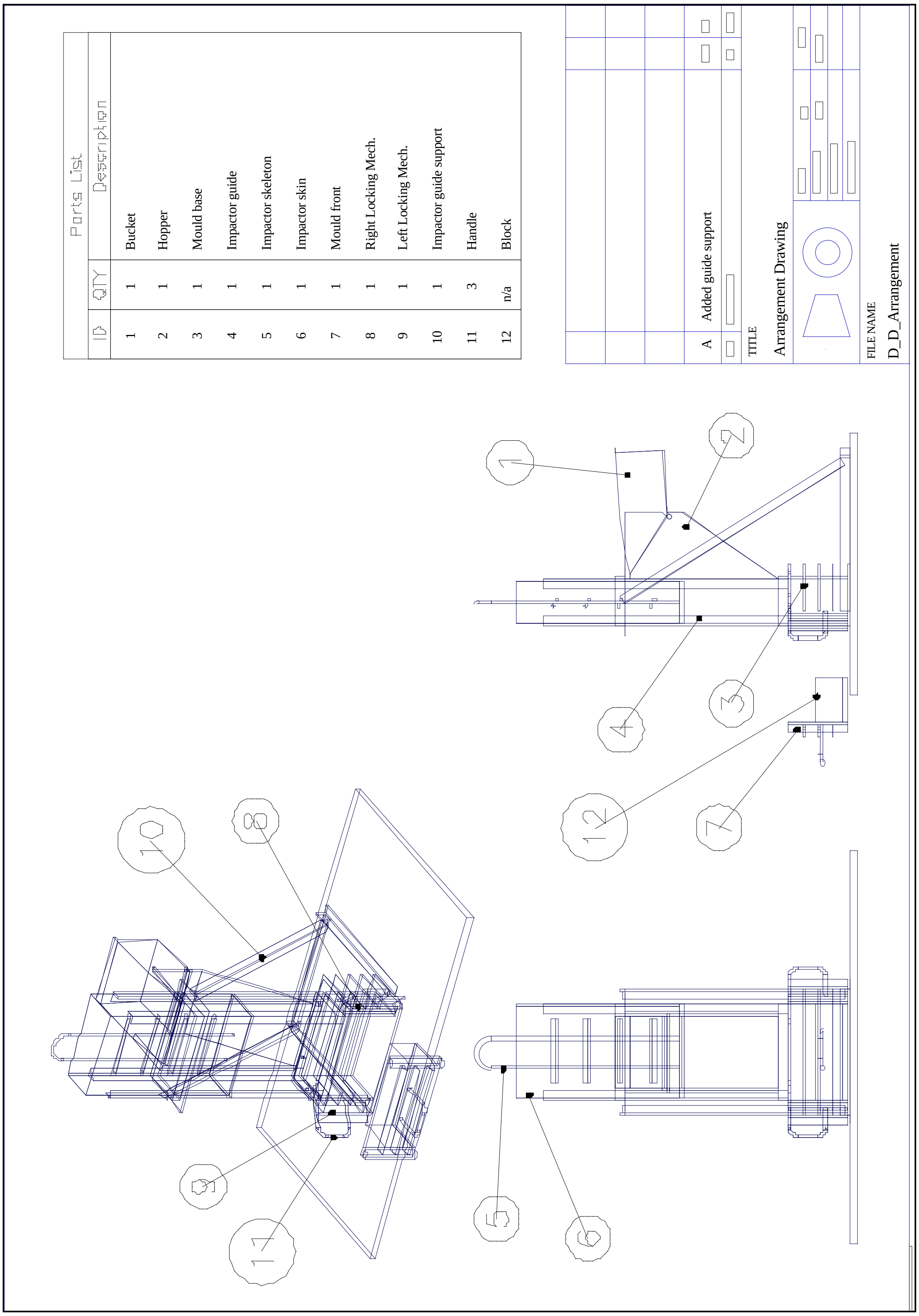
Appendix D - Machine drawings

The following pages contain 9 drawings that have been copied from the CAD program used to make them, (Pro-Desktop 2000i²). The original drawings were sent to India for them to manufacture the machine before the overseas trip. All of the following drawings have been scaled to fit onto the page and consequently are not displayed at the scale indicated on the drawing. The first drawing in this collection is the Arrangement drawing that identifies the different components of the machine. The following table summarises each of the drawings with reference to this arrangement drawing and identifies them with the part number that is used.

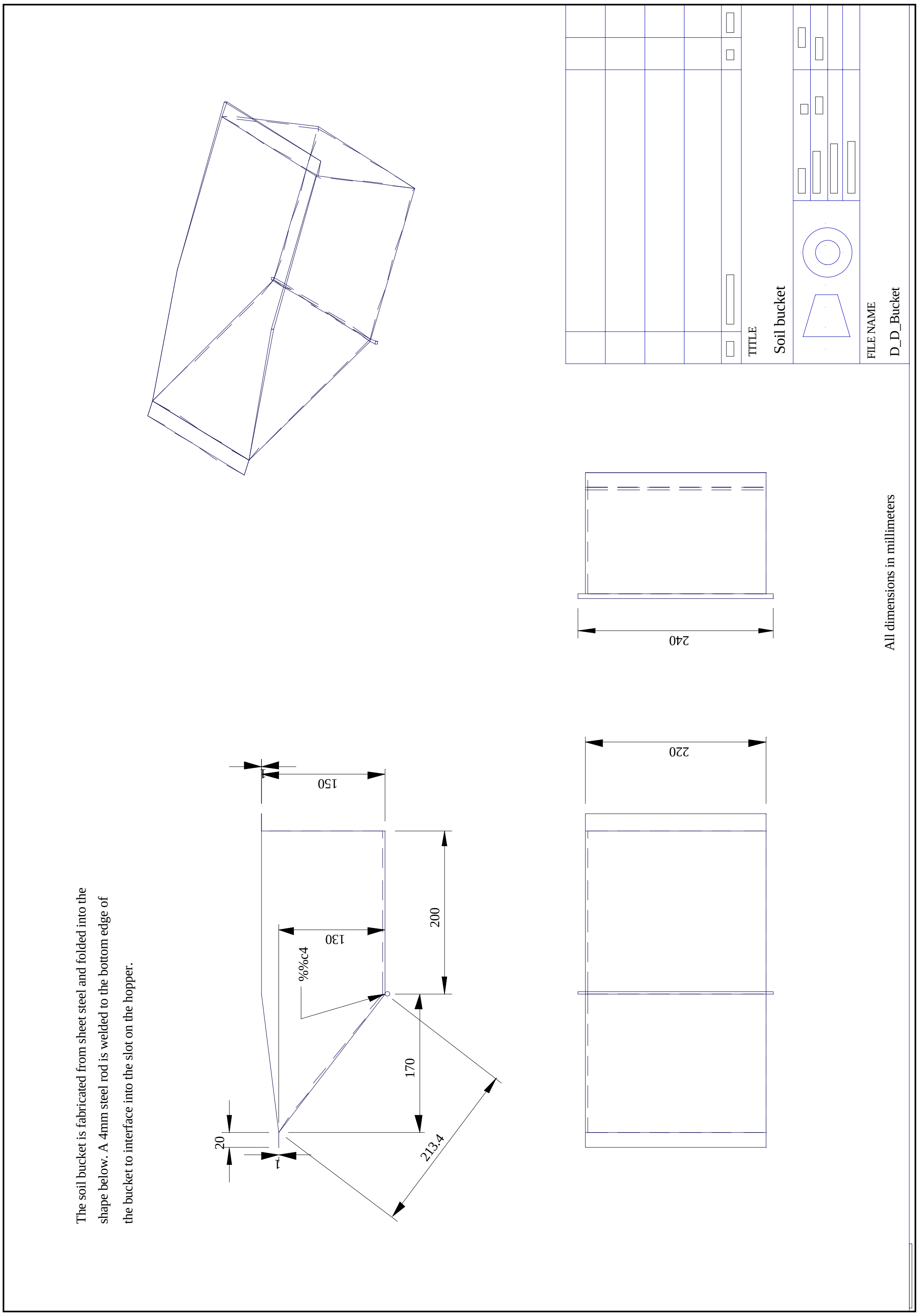
Part No.	Drawing name
1	D_D_Bucket
2	D_D_Hopper
3	D_D_Mould base
4	D_D_Impactor guide
5, 6	D_D_Impactor
7	D_D_Mould front
8, 9, 11	D_D_Locking handles
10	D_D_Impactor guide support

Note: the conversion process has caused some of the text on the drawings to be lost. Also the symbol '∅' has been replaced by '%%c'.

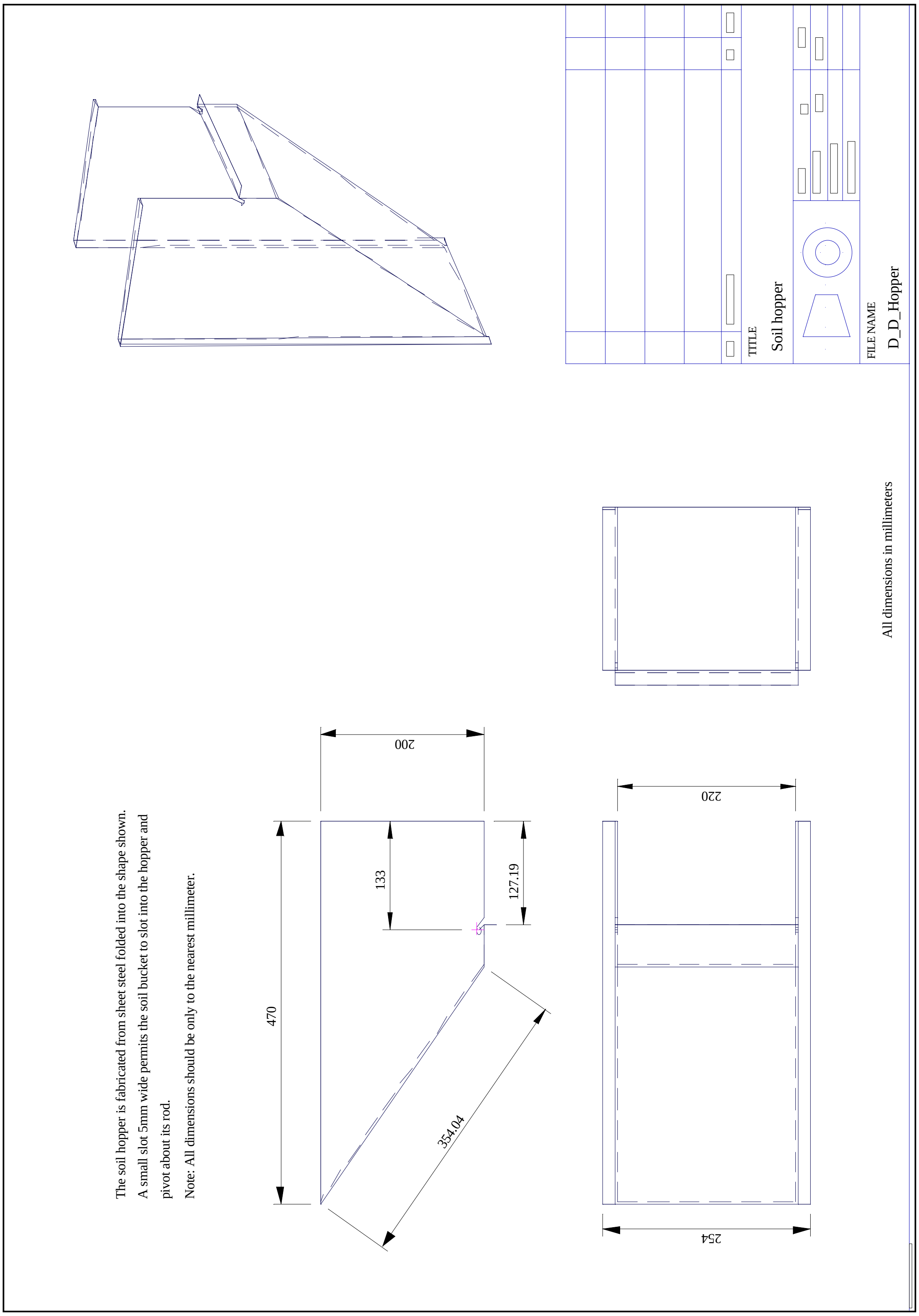
D_D_Arrangement



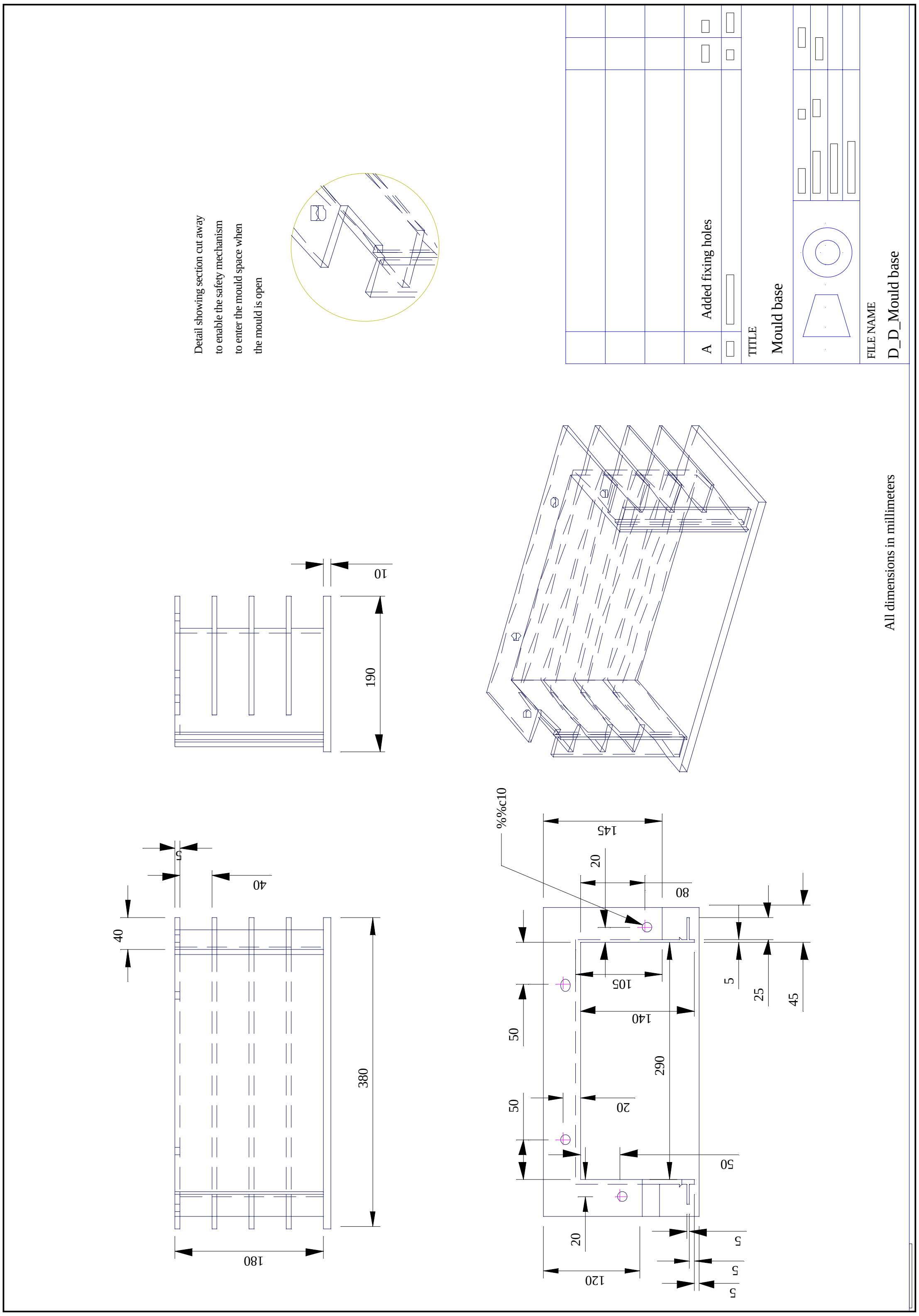
D_D_Bucket


The soil bucket is fabricated from sheet steel and folded into the shape below. A 4mm steel rod is welded to the bottom edge of the bucket to interface into the slot on the hopper.
150
1
130
%%c4
200
170
213.4
20
1
240
220
TITLE
Soil bucket
FILE NAME
D_D_Bucket
All dimensions in millimeters

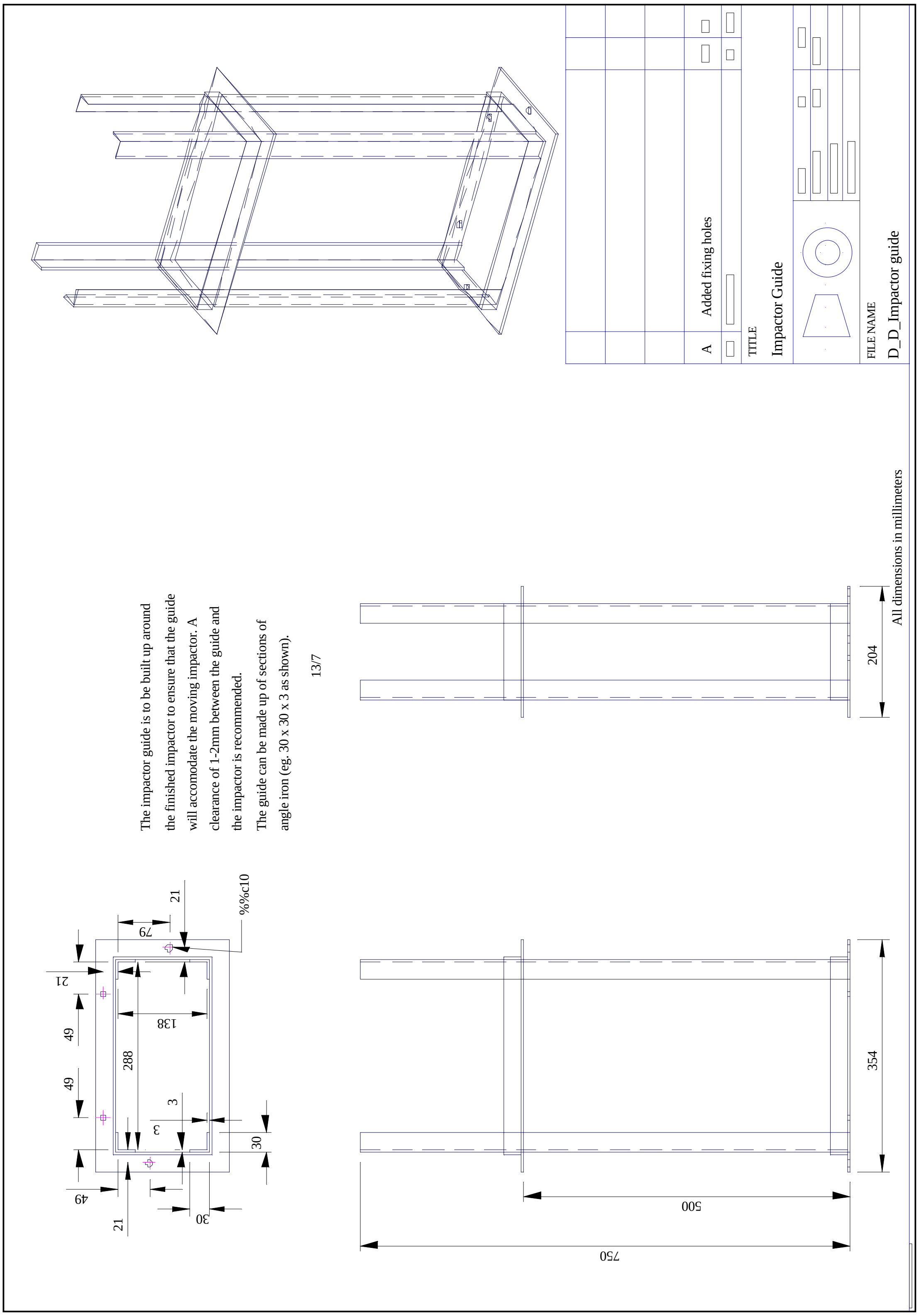
D_D_Hopper


TITLE
Soil hopper
FILE NAME
D_D_Hopper
All dimensions in millimeters
The soil hopper is fabricated from sheet steel folded into the shape shown.
A small slot 5mm wide permits the soil bucket to slot into the hopper and
pivot about its rod.
Note: All dimensions should be only to the nearest millimeter.
200
133
127.19
470
354.04
220
254

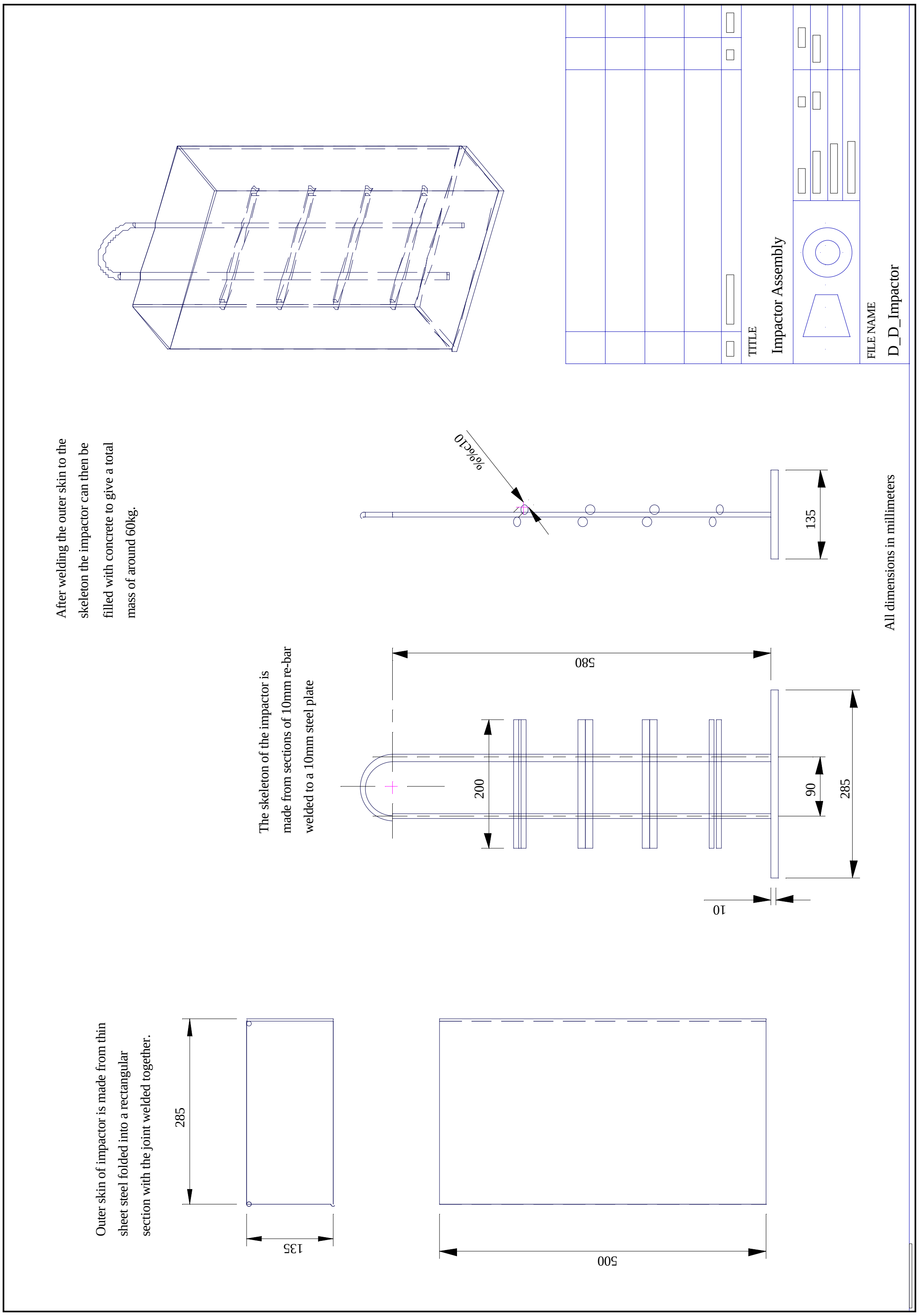
D_D_Mould base



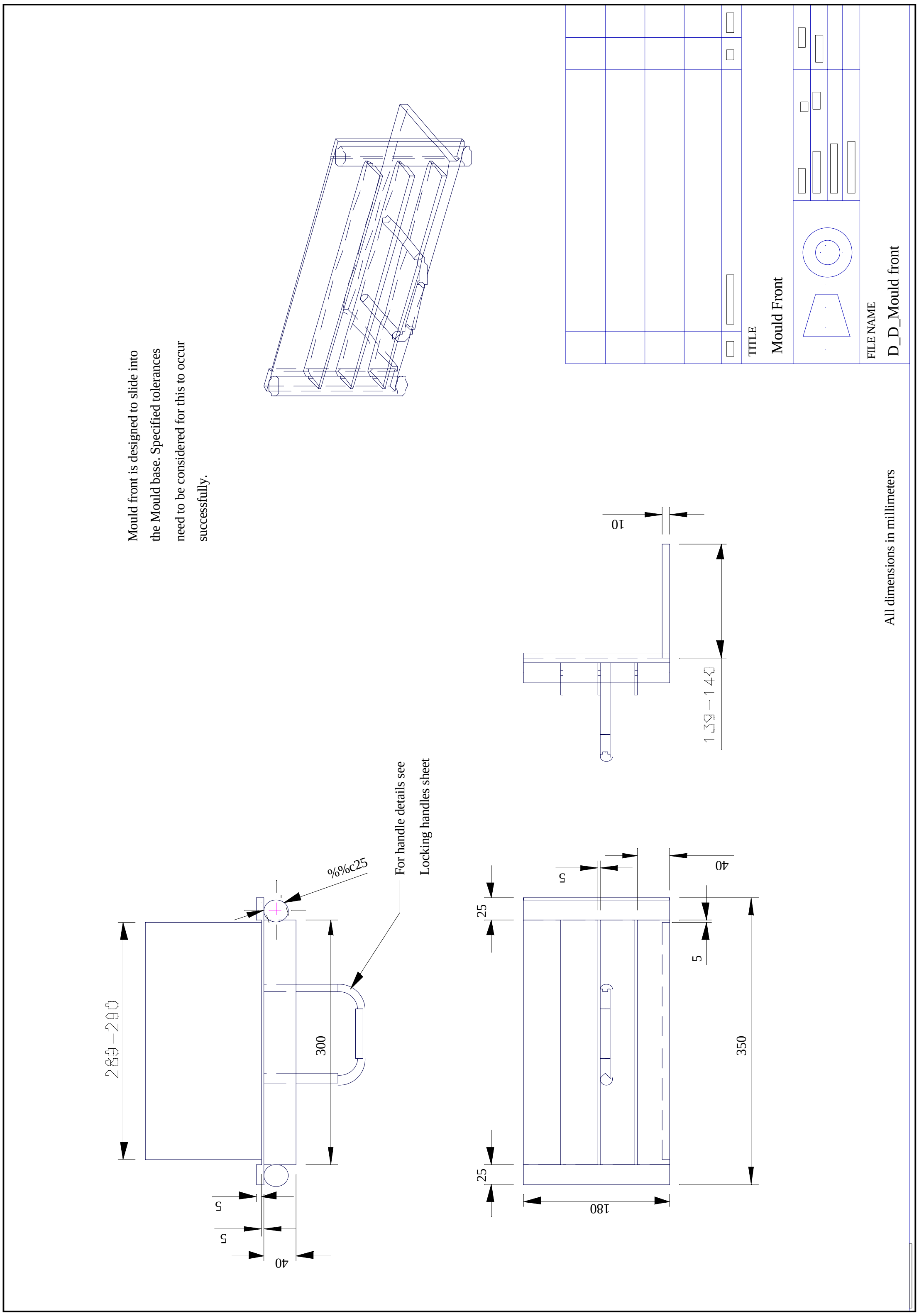
D_D_Impactor guide



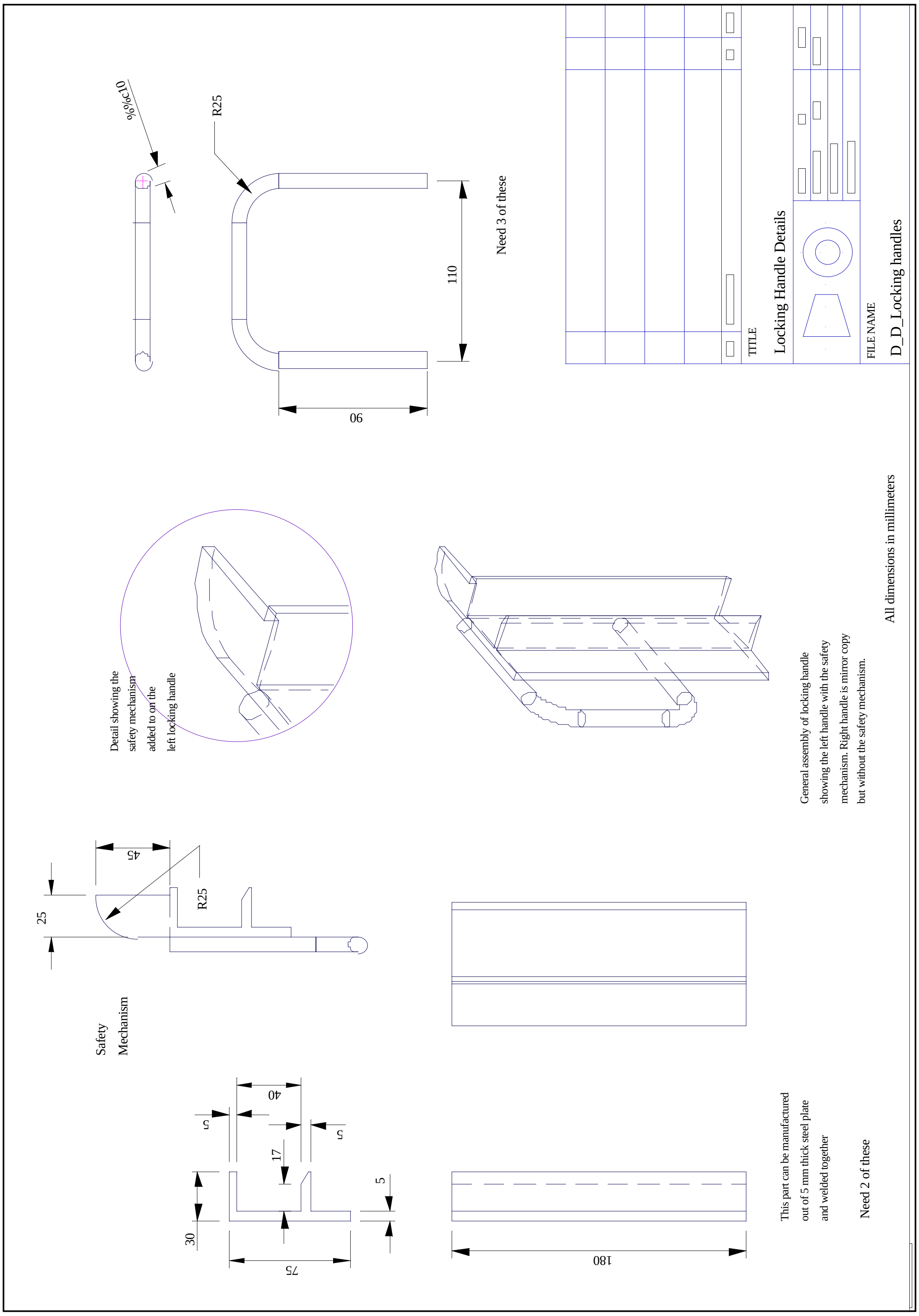
D_D_Impactor


After welding the outer skin to the skeleton the impactor can then be filled with concrete to give a total mass of around 60kg.
The skeleton of the impactor is made from sections of 10mm re-bar welded to a 10mm steel plate
Outer skin of impactor is made from thin sheet steel folded into a rectangular section with the joint welded together.
All dimensions in millimeters
%%c10
135
580
200
90
285
10
285
135
500
TITLE
Impactor Assembly
FILE NAME
D_D_Impactor

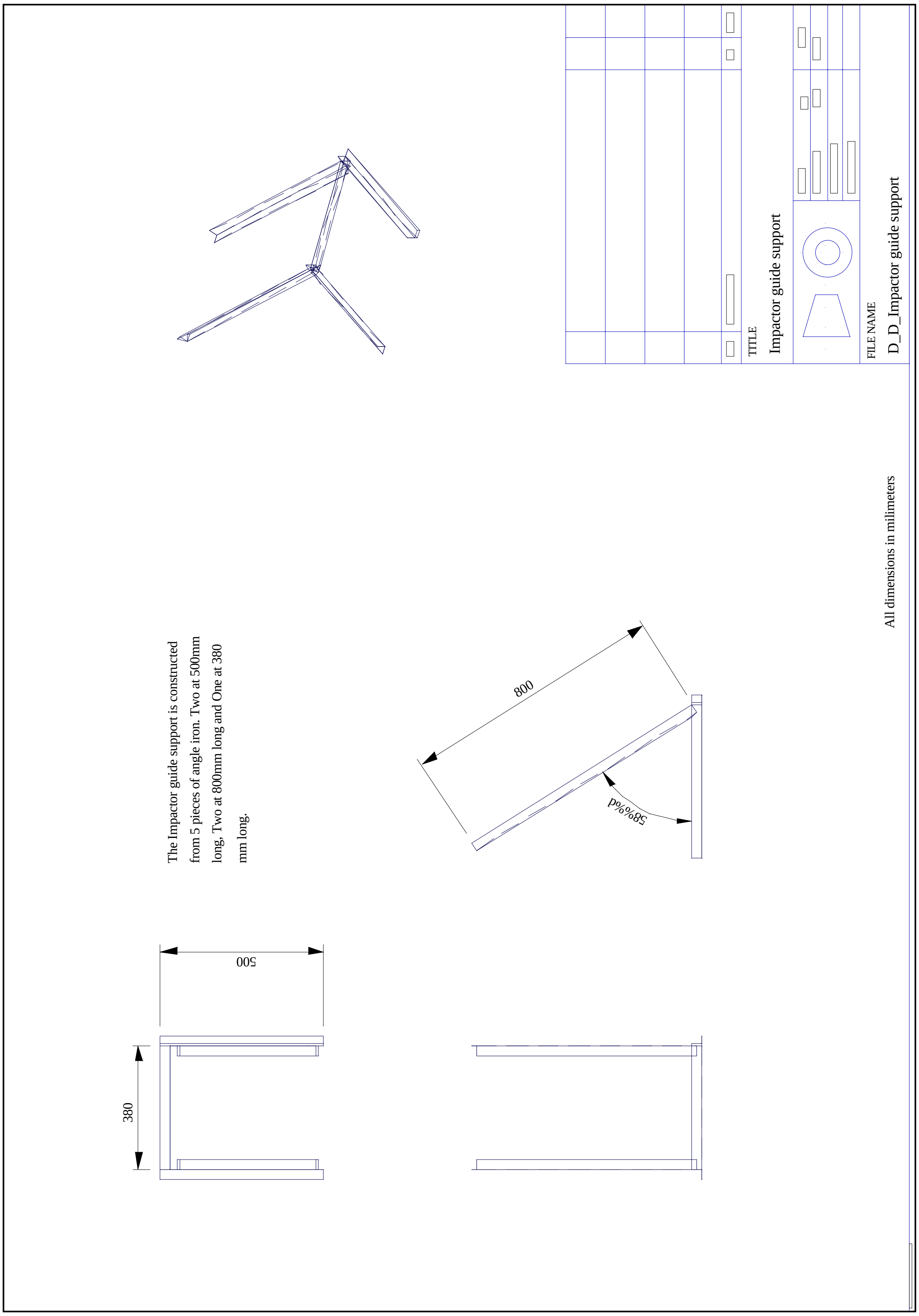
D_D_Mould front



D_D_Locking handles



D_D_Impactor guide support



Appendix E - Material calculations – cement minimisation

Comparing the energy and material requirements of some typical building materials

		Hollow	Hollow					
		Cement	Cement	CSSB	CSSB	Clay	Clay	Thermalite
		Block	Block	High	Low	Brick**	Brick**	Block
Specifications	Units	(Nearby)	(Far)‡	Density	Density	Kiln	Clamp	
Block Length	m	0.300	0.300	0.290	0.290	0.215	0.215	0.440
Block Width	m	0.150	0.150	0.140	0.140	0.105	0.105	0.140
Block Height	m	0.200	0.200	0.090	0.090	0.065	0.065	0.215
Material Density	kg/m³	2200	2200	2000	1700	1350	1350	480
Void Volume	%	50%	50%	0%	0%	0%	0%	0%
Block Mass	kg	9.9	9.9	7.3	6.2	2.0	2.0	6.4
Soil Content	%	0%	0%	95%	90%	100%	100%	0%
Sand Content	%	30%	30%	0%	0%	0%	0%	0%
Gravel Content	%	55%	55%	0%	0%	0%	0%	0%
Cement Content	%	15%	15%	5%	10%	0%	0%	15%
Comp. Str.	MPa	7	7	3	1.5	20	7	7
Raw Materials								
Soil Mass	kg	0.00	0.00	6.94	5.59	1.98	1.98	0.00
Sand Mass	kg	2.97	2.97	0.00	0.00	0.00	0.00	0.00
Gravel Mass	kg	5.45	5.45	0.00	0.00	0.00	0.00	0.00
Cement Mass	kg	1.49	1.49	0.37	0.62	0.00	0.00	0.95
Production								
Processing Energy	kJ/kg	0.4	0.4	0.8	0.6	1514.4	8076.9	0.9
Construction								
Mortar thickness	m	0.01	0.01	0.01	0.015	0.01	0.015	0.003
Render thickness	m	0	0	0	0.015	0	0	0
Material Density	kg/m³	1800	1800	1800	1800	1800	1800	1800
Soil Content	%	0%	0%	80%	80%	0%	0%	0%
Sand Content	%	80%	80%	0%	0%	80%	80%	0%
Cement Content	%	20%	20%	20%	20%	20%	20%	50%
Soil Mass/block	kg	0.00	0.00	0.79	1.89	0.00	0.00	0.00
Sand Mass/block	kg	1.10	1.10	0.00	0.00	0.44	0.67	0.00
Cement Mass/block	kg	0.28	0.28	0.20	0.47	0.11	0.17	0.25
Transportation								
Soil Mass	kg	0.00	0.00	7.73	7.48	1.98	1.98	0.00
Sand Mass/block	kg	4.07	4.07	0.00	0.00	0.44	0.67	0.00
Gravel Mass/block	kg	5.45	5.45	0.00	0.00	0.00	0.00	0.00
Cement Mass/block	kg	1.76	1.76	0.56	1.09	0.11	0.17	1.20
Soil distance	km	0	0	0	0	0	0	0
Sand distance	km	20	50	50	50	50	50	50
Gravel distance	km	20	50	50	50	50	50	50
Cement distance	km	100	100	100	100	100	100	100
Finished blocks distance	km	10	10	10	10	10	10	50
Energy								
Extraction & Processing								
Soil (100 kJ/kg)	kJ	0	0	773	748	198	198	0

Sand (200 kJ/kg)	kJ	814	814	0	0	88	134	0
Gravel (100 kJ/kg)	kJ	545	545	0	0	0	0	0
Cement (6000 kJ/kg)	kJ	10562	10562	3372	6556	658	1004	7214
Block Production	kJ	4	4	6	4	3000	16000	6
Transport								
Truck (35kJ/kg/km)	kJ	16288	26280	4525	5999	1844	2450	15333
Total Per Block unit	MJ	28.21	38.21	8.68	13.31	5.79	19.79	22.55
Comparators								
Block units/m²		15.4	15.4	33.3	31.2	74.1	67.9	10.4
Soil	kg/m²	0.0	0.0	257.6	233.5	146.7	134.6	0.0
Sand	kg/m²	62.5	62.5	0.0	0.0	32.5	45.5	0.0
Gravel	kg/m²	83.6	83.6	0.0	0.0	0.0	0.0	0.0
Cement	**kg/m²**	**27.0**	**27.0**	**18.7**	**34.1**	**8.1**	**11.4**	**12.4**
Total Energy	**MJ/m²**	**433**	**587**	**289**	**416**	**429**	**1344**	**234**
Suitability for local Production	1 - 3 †	1	1	2	2	2	1	2
Suitability for on-site Production	1 - 3 †	2	2	1	1	3	2	3
Notes: ‡	Sand and gravel is transported 50km instead of 20km							
†	Ranking 1 = Best, 3 = Worst							
*	Non-uniform distribution of cement in the block							
**	Brick wall includes a double brick buttress at 1 meter centers for enhanced stability							

Comparing the energy and material requirements of High-density CSSB variants

	Units	Normal CSSB	Hollow CSSB	Cement Rich skin CSSB	Interlock CSSB	Tall CSSB	Rendered CSSB	Tall Hollow Interlock CSSB
Specifications								
Block Length	m	0.290	0.290	0.290	0.297	0.290	0.290	0.297
Block Width	m	0.140	0.140	0.140	0.140	0.140	0.140	0.140
Block Height	m	0.090	0.090	0.090	0.097	0.140	0.090	0.147
Material Density	kg/m³	2000	2000	2000	2000	2000	2000	2000
Void Volume	%	0%	30%	0%	0%	0%	0%	30%
Block Mass	kg	7.3	5.1	7.3	8.1	11.4	7.3	8.6
Soil Content	%	95%	95%	96%	95%	95%	97%	95%
Sand Content	%	0%	0%	0%	0%	0%	0%	0%
Gravel Content	%	0%	0%	0%	0%	0%	0%	0%
Cement Content	%	5%	5%	4.0%	5%	5%	3%	5%
Comp. Str.	MPa	3	3	3	3	3	3	3
Raw Materials								
Soil Mass	kg	6.94	4.86	7.02	7.66	10.80	7.09	8.13
Sand Mass	kg	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Gravel Mass	kg	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cement Mass	kg	0.37	0.26	0.29	0.40	0.57	0.22	0.43
Production								
Processing Energy	kJ/kg	0.8	1.2	0.8	0.7	0.5	0.8	0.7
Construction								
Mortar thickness	m	0.01	0.01	0.01	0.003	0.01	0.01	0.003
Render thickness	m	0	0	0	0	0	0.015	0
Material Density	kg/m³	1800	1800	1800	1800	1800	1800	1800
Soil Content	%	80%	80%	80%	80%	80%	80%	80%
Sand Content	%	0%	0%	0%	0%	0%	0%	0%
Cement Content	%	20%	20%	20%	20%	20%	20%	20%
Soil Mass/block	kg	0.79	0.79	0.79	0.24	0.89	1.43	0.27
Sand Mass/block	kg	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cement Mass/block	kg	0.20	0.20	0.20	0.06	0.22	0.36	0.07
Transportation								
Soil Mass	kg	7.73	5.65	7.80	7.90	11.69	8.52	8.40
Sand Mass/block	kg	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Gravel Mass/block	kg	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Cement Mass/block	kg	0.56	0.45	0.49	0.46	0.79	0.58	0.50
Soil distance	km	0	0	0	0	0	0	0
Sand distance	km	50	50	50	50	50	50	50
Gravel distance	km	50	50	50	50	50	50	50
Cement distance	km	100	100	100	100	100	100	100
Finished blocks distance	km	10	10	10	10	10	10	10
Energy								
Extraction & Processing								
Soil (100 kJ/kg)	kJ	773	565	780	790	1169	852	840
Sand (200 kJ/kg)	kJ	0	0	0	0	0	0	0
Gravel (100 kJ/kg)	kJ	0	0	0	0	0	0	0
Cement (6000 kJ/kg)	kJ	3372	2714	2933	2780	4741	3467	2973
Block Production	kJ	6	6	6	6	6	6	6
Transport								
Truck (35kJ/kg/km)	kJ	4525	3374	4269	4445	6744	4580	4729
Total Per Block unit	MJ	8.68	6.66	7.99	8.02	12.66	8.91	8.55

Comparators								
Block units/m²		33.3	33.3	33.3	33.3	22.2	33.3	22.2
Soil	kg/m²	257.6	188.2	260.1	263.4	259.7	284.1	186.7
Sand	kg/m²	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Gravel	kg/m²	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Cement	**kg/m²**	**18.7**	**15.1**	**16.3**	**15.4**	**17.6**	**19.3**	**11.0**
Total Energy	**MJ/m²**	**289**	**222**	**266**	**267**	**281**	**297**	**190**
Suitability for local Production	1 - 3 †	2	2	1	2	2	2	2
Suitability for on-site Production	1 - 3 †	1	1	2	1	1	1	1
Notes								
Hollow	30% material removal from block core (deep frog arrangement)							
Cement rich skin	10% cement in first 20mm of exterior block surface, 3% in body of block							
Interlock	Thin mortar joint of only 3mm required							
Tall	Increased block height reduces mortar per square meter							
Rendered	15mm render on a block with only 3% cement in body of block							

Comparing the energy and material requirements of CSSB machines used in India

			Hydraform	Block	Hydraform	Block
			CSSB	Impacterre	CSSB	Impacterre
Block Production		Units		CSSB		CSSB
Specifications	Block Length	m	0.215	0.290	0.215	0.290
	Block Width	m	0.221	0.140	0.221	0.140
	Block Height	m	0.116	0.090	0.116	0.090
	Material Density	kg/m³	1875	1925	1875	1925
	Void Volume	%	0%	0%	0%	0%
	Block Mass	kg	10.3	7.0	10.3	7.0
	Soil Content	%	64.5%	66.5%	93.0%	95.0%
	Sand Content	%	30.0%	30.0%	0.0%	0.0%
	Gravel Content	%	0.0%	0.0%	0.0%	0.0%
	Cement Content	%	5.5%	3.5%	7.0%	5.0%
	7-day W.C.S.	MPa	3.5	3.5	3.5	3.5
Raw Materials	Soil Mass	kg	6.67	4.68	9.61	6.68
	Sand Mass	kg	3.10	2.11	0.00	0.00
	Gravel Mass	kg	0.00	0.00	0.00	0.00
	Cement Mass	kg	0.57	0.25	0.72	0.35
Production	Production rate	b/hr	100	60	100	60
	Production lifetime	hr	7680	7680	7680	7680
	Processing Energy	kJ/kg	17.4	0.5	17.4	0.5
Machine cost	Purchase Price	£	2860	500	2860	500
	Cost/m²	£	0.15	0.04	0.15	0.04
Construction	Mortar thickness	m	0	0.01	0	0.01
	Render thickness	m	0	0	0	0
	Material Density	kg/m³	1600	1600	1600	1600
	Soil Content	%	80%	80%	80%	80%
	Sand Content	%	0%	0%	0%	0%
	Cement Content	%	20%	20%	20%	20%
	Soil Mass/block	kg	0.00	0.70	0.00	0.70
	Sand Mass/block	kg	0.00	0.00	0.00	0.00
Transportation	Cement Mass/block	kg	0.00	0.17	0.00	0.17
Raw Materials	Soil Mass	kg	6.67	5.38	9.61	7.38
Total (Block unit)	Sand Mass	kg	3.10	2.11	0.00	0.00
	Gravel Mass	kg	0.00	0.00	0.00	0.00
	Cement Mass	kg	0.57	0.42	0.72	0.53
Distance	Soil	km	0	0	0	0
	Sand	km	25	25	25	25
	Gravel	km	50	50	50	50
	Cement	km	100	100	100	100
Energy	Finished blocks	km	10	10	10	10
Extraction &	Soil (100 kJ/kg)	kJ	667	538	961	738
Processing	Sand (200 kJ/kg)	kJ	620	422	0	0
	Gravel (100 kJ/kg)	kJ	0	0	0	0
	Cement (6000 kJ/kg)	kJ	3410	2525	4340	3159
	Block Production	kJ	180	3.7	180	3.7
Transport	Truck (35kJ/kg/km)	kJ	8319	5781	6149	4304
Total	Per Block unit	MJ	13.20	9.27	11.63	8.20
Material	Block units/m²		40.1	33.3	40.1	33.3

	Soil	kg/m²	267.3	179.2	385.4	246.0
	Sand	kg/m²	124.3	70.3	0.0	0.0
	Gravel	kg/m²	0.0	0.0	0.0	0.0
	Cement	**kg/m²**	**22.8**	**14.0**	**29.0**	**17.5**
Energy	**Total**	**MJ/m²**	**529**	**309**	**466**	**273**
Cost	**Total**	**£/m²**	**1.29**	**0.74**	**1.60**	**0.91**
Suitability	Local Production	1 – 4 †	3	2	2	1
	On-site Production	1 – 4 †	4	3	2	1

† - Ranking 1 = best

Appendix F - Numerical results of small scale tests

First investigation results of pressure density relationship

soil-B with 5% cement and a 6% Moisture Content compressed to 20MPa in 32mm wall thickness mould

		First sample		Second sample		Third sample		
		Pre -	Projected	Pre -	Projected	Pre -	Projected	
Applied	Applied	Ejected	Dry	Ejected	Dry	Ejected	Dry	Average
Pressure	Force	Height	Density	Height	Density	Height	Density	P.D.D.
MPa	kN	mm	kg/m³	mm	kg/m³	mm	kg/m³	kg/m³
0	0.00	63.8	1349	63.5	1355	63.4	1358	1354
2	4.65	46.6	1847	48.2	1785	48.1	1789	1807
4	9.30	44.3	1943	45.7	1884	45.6	1889	1905
6	13.95	42.9	2005	44.4	1937	44.2	1947	1963
8	18.59	42.1	2046	43.6	1974	43.4	1985	2002
10	23.24	41.4	2077	42.9	2004	42.7	2016	2032
12	27.89	40.9	2104	42.4	2030	42.1	2042	2059
14	32.54	40.4	2128	41.9	2052	41.7	2066	2082
16	37.19	40.0	2149	41.5	2073	41.2	2087	2103
18	41.84	39.7	2169	41.1	2091	40.9	2106	2122
20	46.49	39.4	2186	40.8	2108	40.5	2124	2139
0	0.00	41.3	2083	43.0	2003	42.8	2009	2032

soil-B with 5% cement and a 8% Moisture Content compressed to 20MPa in 32mm wall thickness mould

		First sample		Second sample		Third sample		
		Pre -	Projected	Pre -	Projected	Pre -	Projected	
Applied	Applied	Ejected	Dry	Ejected	Dry	Ejected	Dry	Average
Pressure	Force	Height	Density	Height	Density	Height	Density	P.D.D.
MPa	kN	mm	kg/m³	mm	kg/m³	mm	kg/m³	kg/m³
0	0.00	64.1	1342	64.3	1339	63.3	1359	834
2	4.65	47.3	1817	46.9	1835	47.3	1821	1113
4	9.30	45.0	1913	44.6	1931	44.9	1917	1170
6	13.95	43.6	1971	43.2	1991	43.5	1976	1205
8	18.59	42.7	2015	42.3	2032	42.6	2019	1230
10	23.24	42.1	2046	41.7	2062	42.0	2049	1248
12	27.89	41.5	2072	41.2	2088	41.5	2075	1264
14	32.54	41.1	2095	40.8	2110	41.0	2098	1277
16	37.19	40.7	2116	40.4	2130	40.6	2119	1289
18	41.84	40.3	2135	40.1	2148	40.3	2138	1300
20	46.49	40.0	2153	39.7	2166	39.9	2155	1311
0	0.00	41.8	2057	42.0	2048	42.0	2048	1248

soil-B with 5% cement and a 10% Moisture Content compressed to 20MPa in 32mm wall thickness mould

		First sample		Second sample		Third sample		
		Pre -	Projected	Pre -	Projected	Pre -	Projected	
Applied	Applied	Ejected	Dry	Ejected	Dry	Ejected	Dry	Average
Pressure	Force	Height	Density	Height	Density	Height	Density	P.D.D.
MPa	kN	mm	kg/m³	mm	kg/m³	mm	kg/m³	kg/m³
0	0.00	63.5	1355	63.5	1356	63.7	1352	838
2	4.65	46.3	1857	45.7	1882	46.1	1865	1139
4	9.30	44.3	1944	43.6	1973	43.9	1959	1193
6	13.95	43.1	1998	42.4	2029	42.7	2016	1226
8	18.59	42.2	2039	41.6	2070	41.8	2058	1250
10	23.24	41.6	2067	41.0	2099	41.2	2087	1267
12	27.89	41.2	2091	40.5	2123	40.8	2112	1281
14	32.54	40.8	2110	40.2	2143	40.4	2133	1293
16	37.19	40.5	2127	39.8	2161	40.0	2151	1304
18	41.84	40.2	2140	39.5	2176	39.7	2168	1313
20	46.49	40.0	2154	39.3	2189	39.4	2182	1321
0	0.00	41.3	2081	41.4	2077	41.4	2076	1263

Second investigation results of pressure density relationship

soil-B with 5% cement and a 6% Moisture Content compressed to 8, 10, 12MPa in 8mm wall thickness mould

		First sample		Second sample		Third sample		
		Pre -	Projected	Pre -	Projected	Pre -	Projected	
Applied	Applied	Ejected	Dry	Ejected	Dry	Ejected	Dry	Average
Pressure	Force	Height	Density	Height	Density	Height	Density	P.D.D.
MPa	kN	mm	kg/m³	mm	kg/m³	mm	kg/m³	kg/m³
0	0.00	62.5	1377	63.0	1366	65.0	1324	1355
2	4.65	47.4	1815	46.6	1846	49.3	1747	1803
4	9.30	45.1	1908	44.4	1939	47.1	1827	1892
6	13.95	43.6	1972	43.0	1999	45.8	1878	1950
8	18.59	42.9	2007	42.1	2042	44.9	1915	1988
10	23.24	42.3	2036	41.5	2072	44.3	1943	2017
12	27.89	41.7	2062	41.0	2098	43.7	1967	2042
0	0.00	43.4	1983	43.7	1969	44.6	1929	1960
0	0.00	62.8	1370	63.6	1353	63.6	1353	1359
0.5	1.16	51.8	1662	52.8	1631	53.2	1617	1637
1	2.32	49.1	1752	50.1	1719	50.5	1704	1725
2	4.65	46.9	1834	47.5	1810	48.2	1785	1809
3	6.97	45.6	1885	46.2	1862	46.9	1836	1861
4	9.30	44.6	1928	45.3	1899	45.9	1874	1900
6	13.95	43.2	1994	44.0	1955	44.5	1933	1960
8	18.59	42.3	2035	43.1	1996	43.6	1973	2001
10	23.24	41.6	2067	42.5	2027	42.9	2005	2033
0	0.00	43.4	1983	44.2	1947	44.6	1929	1953
0	0.00	61.8	1392	62.8	1370	63.6	1353	1372
0.5	1.16	51.4	1675	52.2	1647	52.6	1635	1652
1	2.32	48.7	1766	49.3	1745	50.0	1721	1744
2	4.65	46.3	1858	46.9	1835	47.5	1812	1835
3	6.97	45.0	1913	45.5	1891	46.2	1864	1889
4	9.30	44.1	1951	44.5	1932	45.3	1900	1928
6	13.95	42.8	2011	43.2	1993	44.0	1956	1987
8	18.59	41.9	2054	42.2	2039	43.1	1999	2030
0	0.00	44.3	1942	44.6	1929	44.6	1929	1934

Third investigation results of pressure density relationship

soil-B with 5% cement and a 6% Moisture Content compressed to 4, 6, 8, 10, 12MPa in 8mm wall thickness mould

Applied Pressure MPa	Energy Transfer J	Ejection Force kN	Ejected Height mm	P.D.D. kg/m³	Bulk Density kg/m³	7-day W.C.S. kN	7-day W.C.S. MPa
12	111	1.2	43.4	1983	2102	4.11	1.77
12	111	1.2	43.7	1969	2087	4.60	1.98
12	111	1.2	44.6	1929	2045	5.59	2.40
10	97	1.1	43.8	1965	2082	4.95	2.13
10	97	1.1	44.2	1947	2064	4.28	1.84
10	97	1.1	44.4	1938	2054	3.82	1.64
8	83	1.1	44.3	1942	2059	3.65	1.57
8	83	1.1	44.6	1929	2045	3.83	1.65
8	83	1.1	44.8	1921	2036	3.58	1.54
6	70	0.4	44.6	1929	2045	3.24	1.39
6	70	0.5	44.7	1925	2041	3.76	1.62
6	70	0.6	44.9	1916	2031	3.69	1.59
4	54	0.5	45.3	1900	2013	3.36	1.44
4	54	0.5	45.9	1875	1987	2.89	1.24
4	54	0.5	46.2	1863	1974	2.87	1.23

Density, strength and ejection force variation for cylindrical samples

All samples compressed to 10MPa using soil-B (5% cement) at 6% M.C. in 8 mm wall mould

			Order in batch		
	Units	Batch	First	Second	Third
Ejection force	kN	1	0.64	1.22	1.23
Ejection force	kN	2	0.90	0.88	1.21
Ejection force	kN	3	0.90	0.96	0.99
Ejection force	kN	4	1.13	1.00	1.06
Ejection force	kN	5	1.04	1.22	1.14
Ejection force	kN	6	1.10	1.12	1.16
Average	kN		0.95	1.07	1.13
Standard deviation	kN		0.18	0.14	0.09
Coefficient of variation	%		18.9%	13.4%	8.0%
Ejected height	mm	1	44.1	44.3	44.8
Ejected height	mm	2	44.2	44.6	44.5
Ejected height	mm	3	44.5	44.8	44.6
Ejected height	mm	4	44.0	44.2	44.3
Ejected height	mm	5	44.0	44.0	44.3
Ejected height	mm	6	44.0	44.5	44.4
Average	mm		44.1	44.4	44.5
Standard deviation	mm		0.20	0.29	0.19
Coefficient of variation	%		0.4%	0.7%	0.4%
P.D.D.	kg/m³	1	1951	1942	1921
P.D.D.	kg/m³	2	1947	1929	1934
P.D.D.	kg/m³	3	1934	1921	1929
P.D.D.	kg/m³	4	1956	1947	1942
P.D.D.	kg/m³	5	1956	1956	1942
P.D.D.	kg/m³	6	1956	1934	1938
Average	kg/m³		1950	1938	1934
Standard deviation	kg/m³		9	13	8
Coefficient of variation	%		0.4%	0.7%	0.4%
7-day W.C.S.	MPa	1	1.80	1.63	1.45
7-day W.C.S.	MPa	2	1.72	1.52	1.63
7-day W.C.S.	MPa	3	1.64	1.53	1.53
7-day W.C.S.	MPa	4	1.72	1.76	1.68
7-day W.C.S.	MPa	5	1.77	1.77	1.74
7-day W.C.S.	MPa	6	1.91	1.48	1.72
Average	MPa		1.76	1.61	1.63
Standard deviation	MPa		0.09	0.13	0.11
Coefficient of variation	%		5.3%	7.8%	7.0%

Indirect and direct compaction experimental results

All samples compacted at 6% M.C.

Indirect Dynamic Compaction						
Number	Impactor	Drop	Total	Energy	Ejected	P.D.D.
of blows	Mass	Height	Energy		Height	
	kg	m	J	J/kg	mm	kg/m³
8	2.5	0.26	51	255	49.5	1738
8	2.5	0.26	51	255	49.6	1735
8	2.5	0.26	51	255	49.7	1731
16	2.5	0.26	102	510	47.6	1808
16	2.5	0.26	102	510	47.6	1808
16	2.5	0.26	102	510	47.4	1815
32	2.5	0.26	204	1020	46.2	1863
32	2.5	0.26	204	1020	46.2	1863
32	2.5	0.26	204	1020	46.1	1867
Direct Dynamic Compaction						
8	2.5	0.2	39	196	45.6	1887
8	2.5	0.2	39	196	45.8	1879
8	2.5	0.2	39	196	46.0	1871
16	2.5	0.2	78	392	43.4	1983
16	2.5	0.2	78	392	44.5	1934
16	2.5	0.2	78	392	44.8	1921
32	2.5	0.2	157	785	43.0	2001
32	2.5	0.2	157	785	42.7	2015
32	2.5	0.2	157	785	43.2	1992

Appendix G - Numerical results of full size blocks

Block data for 12 full-size blocks of varying moisture content

Block production using Bre-pak block press with soil-A and 10MPa pressure

Block Label	Units	3.1	3.2	3.3	4.1	4.2	4.3
Pre wetted soil mass	g	8200	8200	8200	8200	8200	8200
Moisture content	%	0.9	0.9	0.9	0.9	0.9	0.9
Soil mass	g	8126	8126	8126	8126	8126	8126
Water mass	g	74	74	74	74	74	74
Added cement mass	g	446	446	446	451	451	451
Added water mass	g	267	267	267	360	360	360
Total solid mass	g	8572	8572	8572	8577	8577	8577
Moisture content	%	4	4	4	5	5	5
Cement content	%	5.2	5.2	5.2	5.3	5.3	5.3

Block height	mm	113.1	113.1	112.8	112.5	111.9	113.5
Ejected bulk density	kg/m³	1941	1941	1946	1973	1983	1955
Apparent dry density	kg/m³	1867	1867	1872	1878	1888	1861

Curing period	Days	7	7	7	7	7	7
Soaking period	Hours	20	20	20	20	20	20
Compression rate	kN/min	2	5	5	5	5	5
Wet compressive strength	MPa	0.91	0.96	0.88	1.49	1.47	1.54
Calculated 7-day W.C.S.	MPa	0.85	0.90	0.82	1.39	1.37	1.44

Block Label	Units	5.1	5.2	5.3	6.1	6.2	6.3
Pre wetted soil mass	g	8443	8443	8443	8443	8443	8443
Moisture content	%	3.75	3.75	3.75	3.75	3.75	3.75
Soil mass	g	8126	8126	8126	8126	8126	8126
Water mass	g	317	317	317	317	317	317
Added cement mass	g	446	446	446	446	446	446
Added water mass	g	201	201	201	287	287	287
Total solid mass	g	8572	8572	8572	8572	8572	8572
Moisture content	%	6	6	6	7	7	7
Cement content	%	5.2	5.2	5.2	5.2	5.2	5.2

Block height	mm	112.2	112.3	112.1	112.4	112.1	111.2
Ejected bulk density	kg/m³	1995	1994	1997	2011	2016	2032
Apparent dry density	kg/m³	1882	1880	1883	1878	1883	1899

Curing period	Days	7	7	7	7	7	7
Soaking period	Hours	20	20	20	20	20	20
Compression rate	kN/min	5	10	10	10	10	10
Wet compressive strength	MPa	2.15	2.29	2.31	2.57	2.51	2.37
Calculated 7-day W.C.S.	MPa	2.01	2.14	2.16	2.40	2.35	2.21

Block data for 7 full-size blocks of varying moisture content

Block production using Bre-pak block press with soil-A and 10MPa pressure

Moisture Content	%	2.0%	3.1%	4.2%	5.3%	6.4%	7.5%	8.7%
Mass of wet soil	g	8200	8200	8200	8200	8200	8200	8200
Mass of added water	g	117	203	290	380	471	564	660
Block height	mm	110.3	110.2	110.3	109.7	109.1	109.7	108.4
Bulk Density	kg/m³	1857	1878	1896	1926	1958	1968	2013
Projected Dry Density	kg/m³	1831	1833	1831	1841	1851	1841	1863
T.C.	MPa	0.45	>0.45	>0.45	0.43	0.30	0.23	0.23
T.O.D.	MPa	0.40	>0.45	>0.45	0.43	0.25	0.20	0.15
S.C.	MPa	>0.45	>0.45	>0.45	0.43	0.23	0.23	0.20
S.O.D.U	MPa	>0.45	>0.45	>0.45	0.40	0.25	0.18	0.15
S.O.D.L	MPa	>0.45	>0.45	>0.45	0.45	0.33	0.25	0.18
E.C.	MPa	>0.45	>0.45	>0.45	0.45	0.33	0.23	0.20
B.C.	MPa	0.45	>0.45	>0.45	0.40	0.30	0.25	0.18
Penetrometer Average	MPa	N/A	N/A	N/A	0.43	0.28	0.22	0.18
Standard Deviation	MPa	N/A	N/A	N/A	0.02	0.04	0.03	0.03
Coefficient of variation	%	N/A	N/A	N/A	4.8%	14.2%	12.1%	15.3%

Key to abbreviations

Top Centre	T.C.
Top Offset Diagonal	T.O.D.
Side Centre	S.C.
Side Offset Diagonal Upper	S.O.D.U.
Side Offset Diagonal Upper	S.O.D.L.
End Centre	E.C.
Bottom Centre	B.C.

*****Put in some block data for dynamically compacted blocks*****

Appendix H - Field trial results

Block production using different machines during visit to India

Balram								
Wet Block Tests	M.C.	12.2%						
Block label	n/a	11w	12w	13w	14w	15w	16w	
Block Mass		3.69	3.8	3.76	3.73	3.86	3.82	
Total volume	m³	0.001834	0.001878	0.001872	0.001836	0.001899	0.001858	
Block Bulk Density	kg/m³	2012	2023	2008	2031	2032	2056	
P.D.D.	kg/m³	1794	1804	1791	1811	1812	1833	
Average Indentation	mm	21.4	19.8	19.3	19.4	18.4	17.8	
After 9 days curing and 24hours under water the following tests were conducted								
Average indentation	mm	12.5		12.5	11.5		11.625	
Wet Block mass	kg	3.86		3.94	3.89		3.97	
Total volume	m³	0.001853		0.001887	0.001854		0.001872	
Block wet Density	kg/m³	2083		2088	2098		2120	
Wet compressive strength	kN	40		40	40		50	
	MPa	2.1		2.1	2.1		2.6	
Dry Block Tests	M.C.	12.2%						
Block label	n/a	17d	18d	19d	20d	21d	22d	23
Block Mass		3.74	3.7	3.76	3.76	3.84	3.79	3.96
Total volume	m³	0.001873	0.001846	0.001875	0.001845	0.001911	0.001867	0.001929
Block Bulk Density	kg/m³	1997	2004	2005	2038	2010	2030	2052
P.D.D.	kg/m³	1781	1787	1788	1817	1792	1810	1830
Average Indentation	mm	21.4	19.8	19.3	19.4	18.4	17.8	17.5
After 9 days curing and 24hours in an oven at 105°C the following tests were conducted								
Average indentation	mm				7.8	6.9	7.1	6.9
Dry Block mass	kg				3.43	3.45	3.43	3.56
Total volume	m³				0.001847	0.001912	0.001869	0.001928
Block Dry Density	kg/m³				1858	1804	1835	1847
Dry compressive strength	kN				90	160	100	170
	MPa				4.7	8.4	5.2	8.9

Hydraform							
Wet Block Tests							
Block label	n/a	1w	2w	3w	4w	5w	6w
Block Mass		10.7	10.7	10.55	10.8	10.55	10.6
Total volume	m³	0.005798	0.005753	0.005671	0.005771	0.005629	0.005683
Block Bulk Density	kg/m³	1846	1860	1860	1871	1874	1865
P.D.D.	kg/m³	1683	1696	1696	1706	1709	1701
Average Indentation	mm	17.5	17.5	16.5	17.4	16.8	17.3
After 6 days curing and 24hours under water the following tests were conducted							
Average Indentation	mm	11.4		11.1		10.9	
Wet Block mass	kg	11.25		11.05		11.02	
Total volume	m³	0.005797		0.005624		0.005625	
Block wet Density	kg/m³	1941		1965		1959	
Wet compressive strength	kN	35		35		40	
	MPa	1.5		1.5		1.7	
Dry Block Tests							
Block label	n/a	1d	2d	3d	4d	5d	6d
Block Mass		10.6	10.9	10.55	10.6	10.25	10.25
Total volume	m³	0.00592	0.006045	0.005849	0.005715	0.005579	0.005529
Block Bulk Density	kg/m³	1790	1803	1804	1855	1837	1854
P.D.D.	kg/m³	1633	1644	1645	1691	1675	1690
Average Indentation	mm	18.0	17.6	17.9	17.6	18.9	17.4
After 6 days curing and 24hours in an oven at 105°C the following tests were conducted							
Average Indentation	mm	8.6		8.2		8.6	
Dry Block mass	kg	10		9.95		9.68	
Total volume	m³	0.005916		0.005838		0.005562	
Block Dry Density	kg/m³	1690		1704		1740	
Dry compressive strength	kN	60		75		75	
	MPa	2.5		3.1		3.1	

Block Impacterre						
Wet Block Tests	M.C.	8.5%				
Block label	n/a	1w	2w	3w	4w	5w
Block Mass			6.95	7.9	8.15	8.1
Total volume	m³		0.003506	0.00405	0.004263	0.00418
Block Bulk Density	kg/m³		1982	1951	1912	1938
P.D.D.	kg/m³		1826	1797	1761	1785
Average Indentation	mm		18.3	20.0	20.3	19.3
After 6 days curing and 24hours under water the following tests were conducted						
Average Indentation	mm		9.5	9.75	10.375	
Wet Block mass	kg		7.25	8.25	8.65	
Total volume	m³		0.003469	0.004022	0.004237	
Block wet Density	kg/m³		2090	2051	2042	
Wet compressive strength	kN		140	120	95	
	MPa		4.2	3.6	2.9	
Dry Block Tests	M.C.	8.5%				
Block label	n/a	7d	8d	9d		
Block Mass		8.4	7.8	8.4		
Total volume	m³	0.004312	0.004114	0.004274		
Block Bulk Density	kg/m³	1948	1896	1966		
P.D.D.	kg/m³	1795	1747	1811		
Average Indentation	mm	19.1	19.0	19.1		
After 6 days curing and 24hours in an oven at 105°C the following tests were conducted						
Average Indentation	mm	7.75	7.375	7.125		
Dry Block mass	kg	7.95	7.4	7.95		
Total volume	m³	0.004252	0.004073	0.004221		
Block Dry Density	kg/m³	1870	1817	1883		
Dry compressive strength	kN	240	200	275		
	MPa	7.2	6.0	8.3		

Supplementary test results sent to UK by the collaborators

7 DAY TEST RESULTS OF THE BLOCKS OF THE IMPACTERRE MACHINE AND HYDRAFORM
All blocks soaked for 48 hours in water prior to Wet Compressive Strength test

BLOCK IMPACTERRE MACHINE		SOIL :-	65%
SOIL SAMPLE CODE :-	703/Del/118/2001	SAND :-	30%
Date of Production :-	25/09/2001	CEMENT :-	5%
Date of Testing :-	03/10/2001	WATER :-	10%

Sl. No.	Block Length	Block Width	Loading Area	Wet Weight	Indentation Diameter	Failure Load	Block Strength
	cm	cm	cm	kg	mm	(kN)	MPa
1	29.00	14.24	412.96	7.580	7.40	170.00	4.12
2	29.96	14.25	426.81	7.560	8.00	185.00	4.33
3	29.60	14.30	423.28	7.500	8.00	200.00	4.73
4	29.60	14.30	423.28	7.690	8.10	195.00	4.61
5	29.40	14.30	420.42	7.450	7.50	225.00	5.35
Average	29.51	14.28	421.35	7.56	7.80	195.00	4.63
C. of. V	1.2%	0.2%	1.2%	1.2%	4.2%	10.4%	10.1%

BLOCK IMPACTERRE MACHINE		SOIL :-	63%
SOIL SAMPLE CODE :-	703/Del/118/2001	SAND :-	30%
Date of Production :-	26/09/2001	CEMENT :-	7%
Date of Testing :-	03/10/2001	WATER :-	10%

Sl. No.	Block Length cm	Block Width cm	Loading Area cm	Wet Weight kg	Indentation Diameter mm	Failure Load (kN)	Block Strength MPa
1	29.52	14.34	423.32	7.440	7.00	250.00	5.91
2	29.30	14.32	419.58	7.760	7.00	220.00	5.24
3	29.80	14.30	426.14	7.800	6.80	205.00	4.81
4	29.80	14.32	426.74	7.380	6.50	240.00	5.62
5	29.50	14.34	423.03	7.280	7.00	265.00	6.26
Average	29.58	14.32	423.76	7.53	6.86	236.00	5.57
C. of. V	0.7%	0.1%	0.7%	3.1%	3.2%	10.1%	10.2%

7 DAY TEST RESULTS OF THE BLOCKS OF THE IMPACTERRE MACHINE AND HYDRAFORM
All blocks soaked for 48 hours in water prior to Wet Compressive Strength test

HYDRAFORM MACHINE		SOIL :-	65%
SOIL SAMPLE CODE :-	703/Del/118/2001	SAND :-	30%
Date of Production :-	28/09/2001	CEMENT :-	5%
Date of Testing :-	05/10/2001	WATER :-	10%

Sl. No.	Block Length cm	Block Width cm	Loading Area cm	Wet Weight kg	Indentation Diameter mm	Failure Load (kN)	Block Strength MPa
1	21.23	10.00	212.30	11.030	6.00	65.00	3.06
2	21.20	10.00	212.00	11.020	6.80	70.00	3.30
3	21.48	10.00	214.80	11.120	7.00	75.00	3.49
4	21.37	10.00	213.70	11.050	7.80	70.00	3.28
5	21.88	10.00	218.80	11.335	7.00	65.00	2.97
Average	21.43	10.00	214.32	11.11	6.92	69.00	3.22
C. of. V	1.3%	0.0%	1.3%	1.2%	9.3%	6.1%	6.4%

HYDRAFORM MACHINE		SOIL :-	63%
SOIL SAMPLE CODE :-	703/Del/118/2001	SAND :-	30%
Date of Production :-	28/09/2001	CEMENT :-	7%
Date of Testing :-	05/10/2001	WATER :-	10%

Sl. No.	Block Length cm	Block Width cm	Loading Area cm	Wet Weight kg	Indentation Diameter mm	Failure Load (kN)	Block Strength MPa
1	21.48	10.00	214.80	11.110	5.50	80.00	3.72
2	21.90	10.00	219.00	11.355	5.80	80.00	3.65
3	21.76	10.00	217.60	11.250	6.00	85.00	3.91
4	21.90	10.00	219.00	11.350	6.00	85.00	3.88
5	22.05	10.00	220.50	11.400	6.20	85.00	3.85
Average	21.82	10.00	218.18	11.29	5.90	83.00	3.80
C. of. V	1.0%	0.0%	1.0%	1.0%	4.5%	3.3%	2.9%

28 DAY TEST RESULTS OF THE BLOCKS OF THE IMPACTERRE MACHINE AND HYDRAFORM

All blocks soaked for 48 hours in water prior to Wet Compressive Strength test

BLOCK IMPACTERRE MACHINE		SOIL :-	65%
SOIL SAMPLE CODE :-	703/Del/118/2001	SAND :-	30%
Date of Production :-	25/09/2001	CEMENT :-	5%
Date of Testing :-	22/10/2001	WATER :-	10%

Sl. No.	Block Length cm	Block Width cm	Block Height cm	Loading Area cm	Wet Weight kg	Wet Density kg/m³	Indentation Diameter mm	Failure Load (kN)	Block Strength MPa
1	29.06	14.30	8.25	415.56	7.240	2112	7.60	295.00	7.10
2	29.02	14.27	8.71	414.12	7.630	2115	7.70	295.00	7.12
3	29.06	14.13	8.87	410.62	7.685	2110	8.10	235.00	5.72
4	28.90	14.23	8.74	411.25	7.580	2109	7.00	260.00	6.32
5	28.89	14.13	8.80	408.22	7.600	2116	7.20	240.00	5.88
Average	28.99	14.21	8.67	411.95	7.55	2112.34	7.52	265.00	6.43
C. of. V	0.3%	0.6%	2.8%	0.7%	2.3%	0.1%	5.8%	10.9%	10.3%

BLOCK IMPACTERRE MACHINE		SOIL :-	63%
SOIL SAMPLE CODE :-	703/Del/118/2001	SAND :-	30%
Date of Production :-	26/09/2001	CEMENT :-	7%
Date of Testing :-	23/10/2001	WATER :-	10%

Sl. No.	Block Length cm	Block Width cm	Block Height cm	Loading Area cm	Wet Weight kg	Wet Density kg/m³	Indentation Diameter mm	Failure Load (kN)	Block Strength MPa
1	28.90	14.29	9.53	412.98	8.100	2058	7.90	180.00	4.36
2	29.10	14.20	9.20	413.22	7.750	2039	7.10	245.00	5.93
3	28.94	14.29	9.50	413.55	7.970	2029	8.00	200.00	4.84
4	28.90	14.25	8.57	411.83	7.430	2105	6.00	325.00	7.89
5	28.90	14.23	9.10	411.25	7.765	2075	6.00	245.00	5.96
Average	28.95	14.25	9.18	412.57	7.80	2061.09	7.00	239.00	5.79
C. of. V	0.3%	0.3%	4.2%	0.2%	3.3%	1.5%	14.0%	23.4%	23.5%

28 DAY TEST RESULTS OF THE BLOCKS OF THE IMPACTERRE MACHINE AND HYDRAFORM
All blocks soaked for 48 hours in water prior to Wet Compressive Strength test

HYDRAFORM MACHINE		SOIL :-	65%
SOIL SAMPLE CODE :-	703/Del/118/2001	SAND :-	30%
Date of Production :-	28/09/2001	CEMENT :-	5%
Date of Testing :-	05/10/2001	WATER :-	10%

Sl. No.	Block Length cm	Block Width cm	Loading Area cm	Wet Weight kg	Indentation Diameter mm	Failure Load (kN)	Block Strength MPa
1	21.60	10.00	216.00	11.240	9.00	80.00	3.70
2	21.30	10.00	213.00	11.100	8.00	80.00	3.76
3	21.30	10.00	213.00	11.135	8.00	80.00	3.76
4	21.10	10.00	211.00	11.040	6.00	80.00	3.79
5	21.60	10.00	216.00	11.220	8.00	80.00	3.70
Average	21.38	10.00	213.80	11.15	7.80	80.00	3.74
C. of. V	1.0%	0.0%	1.0%	0.7%	14.0%	0.0%	1.0%

HYDRAFORM MACHINE		SOIL :-	63%
SOIL SAMPLE CODE :-	703/Del/118/2001	SAND :-	30%
Date of Production :-	28/09/2001	CEMENT :-	7%
Date of Testing :-	27/10/2001	WATER :-	10%

Sl. No.	Block Length cm	Block Width cm	Loading Area cm	Wet Weight kg	Indentation Diameter mm	Failure Load (kN)	Block Strength MPa
1	21.60	10.00	216.00	11.200	7.70	110.00	5.09
2	21.60	10.00	216.00	11.215	7.00	110.00	5.09
3	21.80	10.00	218.00	11.315	7.00	115.00	5.28
4	21.70	10.00	217.00	11.280	7.00	120.00	5.53
5	21.50	10.00	215.00	11.160	6.80	125.00	5.81
Average	21.64	10.00	216.40	11.23	7.10	116.00	5.36
C. of. V	0.5%	0.0%	0.5%	0.6%	4.9%	5.6%	5.8%

Block Impacterre			**STABILISED**					
Total weight of each Mix =			**70 Kg**	**With 5% Cement**				
Water =	**7.00 Kg**							
BATCH 1	**M.C. =**	**10.65%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	29.068	14.380	8.220	0.003436	7.210	2098	1896	18.60
2	28.854	14.120	8.320	0.003390	7.200	2124	1920	20.00
3	28.900	14.250	8.200	0.003377	7.150	2117	1913	16.60
4	28.900	14.280	8.260	0.003409	7.180	2106	1904	18.68
5	28.838	14.090	8.750	0.003555	7.460	2098	1896	19.48
6	29.090	14.250	8.834	0.003662	7.580	2070	1871	18.00
7	29.150	14.260	8.648	0.003595	7.550	2100	1898	17.00
8	28.850	14.260	8.720	0.003587	7.440	2074	1874	18.80
9	28.888	14.290	8.740	0.003608	7.450	2065	1866	17.28
10	28.930	14.220	8.460	0.003480	7.350	2112	1909	16.60
Avg.	**28.947**	**14.240**	**8.515**	**0.003510**	**7.357**	**2097**	**1895**	**18.10**
S.D.	**0.113**	**0.083**	**0.249**	**0.00010**	**0.161**	**20.444**	**18.476**	**1.201**
C. of V.	**0.39%**	**0.58%**	**2.93%**	**2.95%**	**2.19%**	**0.98%**	**0.98%**	**6.63%**
BATCH 2	**M.C. =**	**10.32%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	29.010	14.300	8.290	0.003439	7.340	2134	1935	19.84
2	29.070	14.328	8.120	0.003382	7.180	2123	1924	19.78
3	28.910	14.270	8.340	0.003441	7.350	2136	1936	19.34
4	28.910	14.260	8.380	0.003455	7.330	2122	1923	18.20
5	28.890	14.200	8.740	0.003585	7.520	2097	1901	19.74
6	28.880	14.190	8.642	0.003542	7.480	2112	1914	19.30
7	28.862	14.210	8.600	0.003527	7.480	2121	1922	18.80
8	28.842	14.176	8.620	0.003524	7.400	2100	1903	20.34
9	28.850	14.204	8.816	0.003613	7.600	2104	1907	19.44
10	28.910	14.186	9.118	0.003739	7.350	1966	1782	20.10
Avg.	**28.9134**	**14.2324**	**8.5666**	**0.003525**	**7.4030**	**2101**	**1905**	**19.49**
S.D.	**0.073**	**0.053**	**0.292**	**0.00010**	**0.120**	**49.606**	**44.966**	**0.629**
C. of V.	**0.25%**	**0.37%**	**3.41%**	**2.95%**	**1.62%**	**2.36%**	**2.36%**	**3.23%**

Block Impacterre			**STABILISED**					
Total weight of each Mix =			**70 Kg**	**With 5% Cement**				
Water =	**7.00 Kg**							
BATCH 3	**M.C. =**	**11.41%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	28.930	14.240	8.360	0.003444	7.180	2085	1871	16.60
2	28.940	14.240	8.610	0.003548	7.300	2057	1847	16.00
3	28.820	14.170	8.680	0.003545	7.250	2045	1836	17.63
4	28.851	14.160	8.620	0.003522	7.270	2064	1853	18.30
5	28.860	14.240	8.640	0.003551	7.360	2073	1861	17.40
6	28.800	14.150	8.620	0.003513	7.300	2078	1865	16.08
7	28.854	14.130	8.700	0.003547	7.250	2044	1835	18.00
8	28.820	14.160	8.660	0.003534	7.350	2080	1867	15.30
9	28.820	14.000	8.810	0.003555	7.320	2059	1848	17.40
10	28.720	14.050	9.130	0.003684	7.490	2033	1825	18.50
Avg.	**28.842**	**14.154**	**8.683**	**0.003544**	**7.307**	**2062**	**1851**	**17.12**
S.D.	**0.063**	**0.080**	**0.193**	**0.00006**	**0.083**	**17.280**	**15.510**	**1.075**
C. of V.	**0.22%**	**0.57%**	**2.23%**	**1.67%**	**1.14%**	**0.84%**	**0.84%**	**6.28%**
BATCH 4	**M.C. =**	**11.01%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	28.850	14.090	8.200	0.003333	7.250	2175	1959	16.10
2	28.880	14.070	8.500	0.003454	7.340	2125	1914	14.00
3	28.880	14.090	8.410	0.003422	7.240	2116	1906	16.90
4	28.850	14.070	8.570	0.003479	7.340	2110	1901	15.64
5	28.820	14.010	8.600	0.003472	7.430	2140	1928	13.50
6	28.800	14.020	8.700	0.003513	7.550	2149	1936	12.70
7	28.870	14.050	8.620	0.003496	7.400	2116	1907	14.30
8	28.830	14.080	8.740	0.003548	7.400	2086	1879	15.20
9	28.820	14.040	8.840	0.003577	7.560	2114	1904	14.80
10	28.800	14.100	8.820	0.003582	7.460	2083	1876	15.50
Avg.	**28.840**	**14.062**	**8.600**	**0.003488**	**7.397**	**2121**	**1911**	**14.86**
S.D.	**0.031**	**0.031**	**0.195**	**0.00008**	**0.109**	**27.903**	**25.135**	**1.265**
C. of V.	**0.11%**	**0.22%**	**2.27%**	**2.15%**	**1.48%**	**1.32%**	**1.32%**	**8.51%**

Block Impacterre			STABILISED					
Total weight of each Mix =			70 Kg	With 7% Cement				
Water =	7.00 Kg							
BATCH 1	M.C. =	10.22%						
	Block	Block	Block	Block	Block	Bulk		Indentation
	Length	Width	Height	Volume	Mass	Density	P.D.D.	Diameter
Block No.	(cm)	(cm)	(cm)	(m3)	(kg)	(kg/m³)	(kg/m³)	(mm)
1	28.890	14.160	8.290	0.003391	7.220	2129	1932	12.20
2	28.890	14.180	8.460	0.003466	7.350	2121	1924	17.10
3	28.850	14.080	8.670	0.003522	7.450	2115	1919	17.30
4	28.850	14.170	8.770	0.003585	7.440	2075	1883	16.20
5	28.800	14.110	8.830	0.003588	7.560	2107	1912	17.00
6	28.700	14.170	8.900	0.003619	7.530	2080	1888	14.30
7	28.890	14.230	9.130	0.003753	7.540	2009	1823	15.30
8	28.880	14.200	9.400	0.003855	7.630	1979	1796	17.80
9	28.890	14.130	9.430	0.003849	7.620	1979	1796	16.90
10	29.000	14.190	7.600	0.003127	7.480	2392	2170	13.00
Avg.	**28.864**	**14.162**	**8.748**	**0.003576**	**7.482**	**2066**	**1875**	**16.49**
S.D.	**0.077**	**0.044**	**0.546**	**0.00022**	**0.133**	**60.865**	**55.221**	**1.163**
C. of V.	**0.27%**	**0.31%**	**6.24%**	**6.16%**	**1.78%**	**2.95%**	**2.95%**	**7.05%**
BATCH 2	M.C. =	10.58%						
	Block	Block	Block	Block	Block	Bulk		Indentation
	Length	Width	Height	Volume	Mass	Density	P.D.D.	Diameter
Block No.	(cm)	(cm)	(cm)	(m3)	(kg)	(kg/m³)	(kg/m³)	(mm)
1	28.830	14.100	8.500	0.003455	7.360	2130	1926	16.40
2	28.980	14.160	8.500	0.003488	7.270	2084	1885	17.00
3	28.920	14.200	8.530	0.003503	7.280	2078	1879	16.50
4	28.840	14.180	8.560	0.003501	7.290	2082	1883	16.20
5	28.890	14.100	8.720	0.003552	7.370	2075	1876	15.00
6	28.890	14.160	8.470	0.003465	7.230	2087	1887	15.80
7	28.890	14.140	8.770	0.003583	7.350	2052	1855	18.20
8	28.840	14.210	8.730	0.003578	7.270	2032	1838	17.50
9	28.860	14.260	8.990	0.003700	7.600	2054	1858	19.20
10	28.900	14.280	8.970	0.003702	7.620	2058	1861	17.20
Avg.	**28.884**	**14.179**	**8.674**	**0.003553**	**7.364**	**2073**	**1875**	**16.90**
S.D.	**0.045**	**0.060**	**0.194**	**0.00009**	**0.137**	**26.653**	**24.103**	**1.209**
C. of V.	**0.16%**	**0.43%**	**2.23%**	**2.52%**	**1.86%**	**1.29%**	**1.29%**	**7.16%**

Block Impacterre			**STABILISED**					
Total weight of each Mix =			**70 Kg**	**With 7% Cement**				
Water =	**7.00 Kg**							
BATCH 3	**M.C. =**	**10.49%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	28.930	14.080	8.420	0.003430	7.140	2082	1884	20.20
2	28.960	14.060	8.660	0.003526	7.220	2048	1853	20.30
3	28.940	14.070	8.670	0.003530	7.290	2065	1869	19.00
4	29.050	14.200	8.740	0.003605	7.250	2011	1820	19.90
5	28.970	14.220	8.524	0.003511	7.180	2045	1851	18.48
6	28.986	14.230	8.810	0.003634	7.370	2028	1836	20.38
7	28.998	14.100	8.908	0.003642	7.440	2043	1849	20.50
8	28.974	14.240	8.818	0.003638	7.350	2020	1828	20.08
9	28.982	14.280	9.110	0.003770	7.510	1992	1803	20.20
10	28.998	14.226	9.420	0.003886	7.660	1971	1784	20.96
Avg.	**28.979**	**14.171**	**8.808**	**0.003617**	**7.341**	**2030**	**1838**	**20.00**
S.D.	**0.034**	**0.083**	**0.289**	**0.00013**	**0.161**	**33.275**	**30.116**	**0.731**
C. of V.	**0.12%**	**0.59%**	**3.28%**	**3.68%**	**2.19%**	**1.64%**	**1.64%**	**3.66%**
BATCH 4	**M.C. =**	**10.16%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	28.976	14.230	8.434	0.003478	7.250	2085	1893	20.76
2	29.028	14.260	8.198	0.003393	7.090	2089	1897	21.28
3	29.000	14.250	8.200	0.003389	7.160	2113	1918	18.86
4	28.990	14.300	8.520	0.003532	7.290	2064	1874	21.60
5	28.994	14.288	8.460	0.003505	7.220	2060	1870	20.40
6	28.970	14.220	8.650	0.003563	7.280	2043	1855	20.80
7	28.978	14.220	8.774	0.003615	7.330	2027	1840	21.40
8	28.974	14.208	8.774	0.003612	7.330	2029	1842	20.34
9	28.984	14.294	8.746	0.003623	7.360	2031	1844	20.18
10	29.008	14.244	9.060	0.003744	7.520	2009	1824	21.00
Avg.	**28.990**	**14.251**	**8.582**	**0.003545**	**7.283**	**2055**	**1866**	**20.66**
S.D.	**0.018**	**0.033**	**0.272**	**0.00011**	**0.117**	**32.978**	**29.936**	**0.789**
C. of V.	**0.06%**	**0.23%**	**3.17%**	**3.11%**	**1.61%**	**1.60%**	**1.60%**	**3.82%**

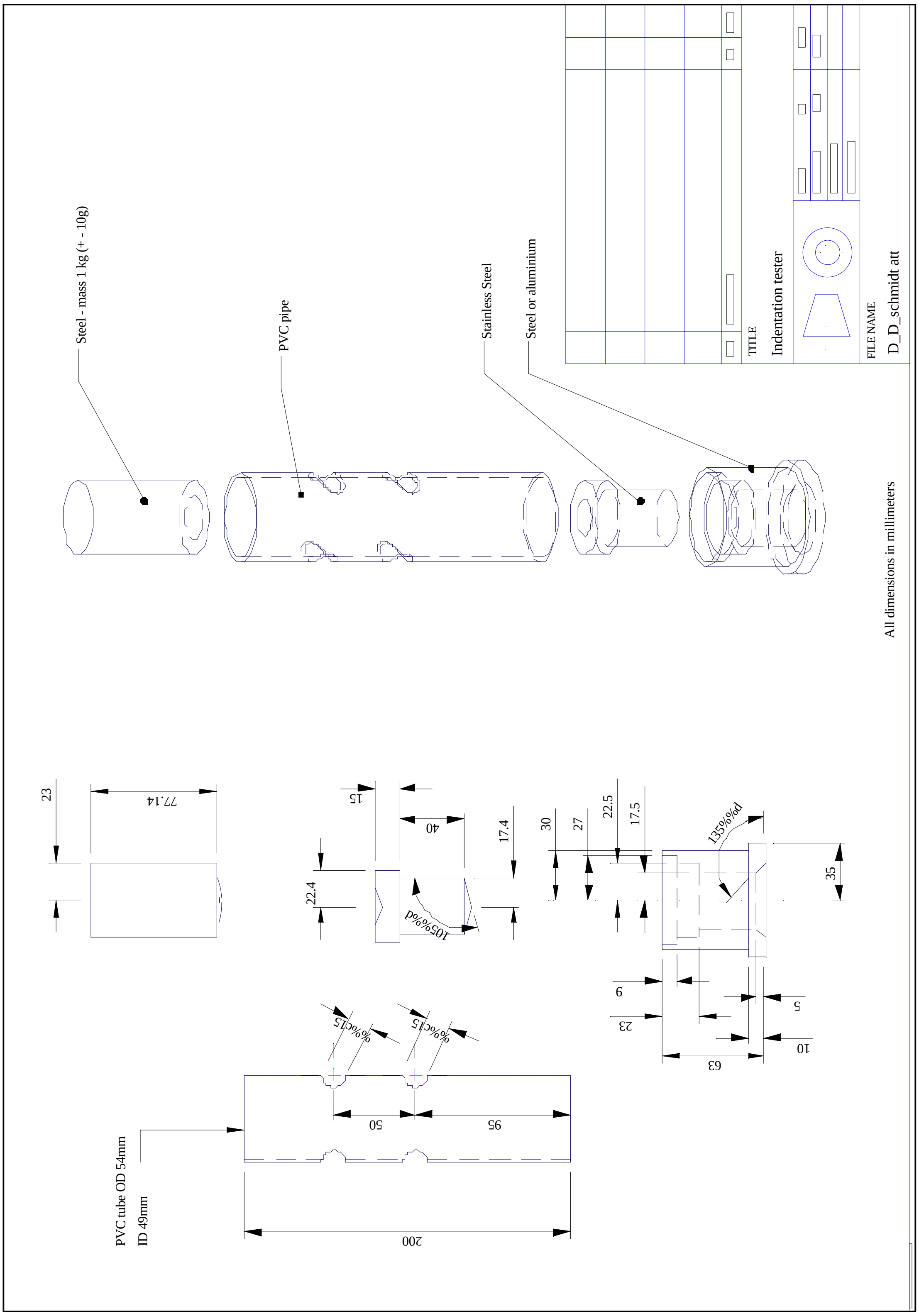
Hydraform Machine				**STABILISED**				
Total weight of each Mix =			**110 Kg**	**With 5% Cement**		**Pressure =**	**85**	
Water =	**11 Kg**							
BATCH 1	**M.C. =**	**11.19%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	21.195	22.100	11.630	0.005448	10.840	1990	1790	19.10
2	21.510	22.100	11.630	0.005529	10.920	1975	1776	21.40
3	21.361	22.100	11.630	0.005490	10.910	1987	1787	21.40
4	21.134	22.100	11.630	0.005432	10.830	1994	1793	19.00
5	20.903	22.100	11.630	0.005373	10.610	1975	1776	17.50
6	21.460	22.100	11.630	0.005516	10.870	1971	1772	17.00
7	21.930	22.100	11.630	0.005637	10.000	1774	1596	16.00
8	21.530	22.100	11.630	0.005534	10.870	1964	1767	15.50
9	21.750	22.100	11.630	0.005590	10.940	1957	1760	16.20
10	21.700	22.100	11.630	0.005577	10.930	1960	1762	16.28
Avg.	**21.447**	**22.100**	**11.630**	**0.005512**	**10.858**	**1975**	**1776**	**17.94**
S.D.	**0.310**	**0.000**	**0.000**	**0.00008**	**0.287**	**64.657**	**58.150**	**2.186**
C. of V.	**1.45%**	**0.00%**	**0.00%**	**1.45%**	**2.65%**	**3.27%**	**3.27%**	**12.18%**
BATCH 2	**M.C. =**	**11.78%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	21.150	22.100	11.630	0.005436	10.660	1961	1754	20.10
2	22.000	22.100	11.630	0.005655	10.020	1772	1585	21.00
3	21.830	22.100	11.630	0.005611	10.860	1936	1732	20.20
4	21.610	22.100	11.630	0.005554	10.760	1937	1733	18.30
5	21.630	22.100	11.630	0.005559	10.820	1946	1741	18.90
6	21.480	22.100	11.630	0.005521	10.740	1945	1740	18.50
7	21.670	22.100	11.630	0.005570	10.860	1950	1744	16.90
8	21.950	22.100	11.630	0.005642	10.930	1937	1733	16.50
9	21.870	22.100	11.630	0.005621	10.970	1952	1746	17.30
10	21.730	22.100	11.630	0.005585	10.820	1937	1733	15.50
Avg.	**21.692**	**22.100**	**11.630**	**0.005575**	**10.824**	**1945**	**1740**	**18.32**
S.D.	**0.250**	**0.000**	**0.000**	**0.00006**	**0.270**	**55.168**	**49.354**	**1.782**
C. of V.	**1.15%**	**0.00%**	**0.00%**	**1.15%**	**2.49%**	**2.84%**	**2.84%**	**9.73%**

Hydraform Machine				**STABILISED**				
Total weight of each Mix =			**110 Kg**	**With 5% Cement**				
Water =	**11 Kg**							
BATCH 3	**M.C. =**	**10.19%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	21.430	22.100	11.630	0.005508	10.850	1970	1788	21.30
2	21.450	22.100	11.630	0.005513	10.840	1966	1784	19.20
3	21.846	22.100	11.630	0.005615	11.050	1968	1786	18.45
4	21.310	22.100	11.630	0.005477	10.750	1963	1781	18.50
5	21.360	22.100	11.630	0.005490	10.790	1965	1784	18.00
6	21.500	22.100	11.630	0.005526	10.850	1963	1782	19.20
7	21.440	22.100	11.630	0.005511	10.810	1962	1780	18.90
8	21.432	22.100	11.630	0.005509	10.870	1973	1791	19.00
9	21.350	22.100	11.630	0.005487	10.810	1970	1788	19.20
10	20.868	22.100	11.630	0.005364	10.630	1982	1799	19.30
Avg.	**21.399**	**22.100**	**11.630**	**0.005500**	**10.825**	**1968**	**1786**	**19.11**
S.D.	**0.238**	**0.000**	**0.000**	**0.00006**	**0.105**	**6.025**	**5.468**	**0.879**
C. of V.	**1.11%**	**0.00%**	**0.00%**	**1.11%**	**0.97%**	**0.31%**	**0.31%**	**4.60%**
BATCH 4	**M.C. =**	**10.79%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	21.105	22.100	11.630	0.005424	10.720	1976	1784	20.50
2	21.848	22.100	11.630	0.005615	11.040	1966	1775	20.30
3	21.410	22.100	11.630	0.005503	10.930	1986	1793	20.00
4	21.300	22.100	11.630	0.005475	10.750	1964	1772	18.90
5	21.480	22.100	11.630	0.005521	10.870	1969	1777	19.00
6	21.582	22.100	11.630	0.005547	10.820	1951	1761	18.50
7	21.868	22.100	11.630	0.005621	10.980	1954	1763	16.00
8	21.850	22.100	11.630	0.005616	10.980	1955	1765	15.80
9	21.575	22.100	11.630	0.005545	10.860	1958	1768	17.86
10	21.550	22.100	11.630	0.005539	10.810	1952	1762	16.50
Avg.	**21.557**	**22.100**	**11.630**	**0.005541**	**10.876**	**1963**	**1772**	**18.34**
S.D.	**0.251**	**0.000**	**0.000**	**0.00006**	**0.105**	**11.615**	**10.484**	**1.751**
C. of V.	**1.16%**	**0.00%**	**0.00%**	**1.16%**	**0.97%**	**0.59%**	**0.59%**	**9.55%**

Hydraform Machine				STABILISED				
Total weight of each Mix =			110 Kg		With 7% Cement			
Water =	11 Kg							
BATCH 1	M.C. =	11.13%						
	Block	Block	Block	Block	Block	Bulk		Indentation
	Length	Width	Height	Volume	Mass	Density	P.D.D.	Diameter
Block No.	(cm)	(cm)	(cm)	(m3)	(kg)	(kg/m³)	(kg/m³)	(mm)
1	21.480	22.100	11.630	0.005521	10.830	1962	1765	20.50
2	21.550	22.100	11.630	0.005539	10.860	1961	1764	19.10
3	21.400	22.100	11.630	0.005500	10.870	1976	1778	19.80
4	21.830	22.100	11.630	0.005611	11.070	1973	1775	19.22
5	21.824	22.100	11.630	0.005609	11.070	1974	1776	20.30
6	21.930	22.100	11.630	0.005637	11.120	1973	1775	18.20
7	21.800	22.100	11.630	0.005603	11.030	1969	1771	18.00
8	21.628	22.100	11.630	0.005559	10.990	1977	1779	18.00
9	21.780	22.100	11.630	0.005598	11.050	1974	1776	19.20
10	21.540	22.100	11.630	0.005536	10.870	1963	1767	17.30
Avg.	**21.676**	**22.100**	**11.630**	**0.005571**	**10.976**	**1970**	**1773**	**18.96**
S.D.	**0.179**	**0.000**	**0.000**	**0.00005**	**0.108**	**6.099**	**5.488**	**1.064**
C. of V.	**0.82%**	**0.00%**	**0.00%**	**0.82%**	**0.98%**	**0.31%**	**0.31%**	**5.61%**
BATCH 2	M.C. =	11.43%						
	Block	Block	Block	Block	Block	Bulk		Indentation
	Length	Width	Height	Volume	Mass	Density	P.D.D.	Diameter
Block No.	(cm)	(cm)	(cm)	(m3)	(kg)	(kg/m³)	(kg/m³)	(mm)
1	21.366	22.100	11.630	0.005492	10.780	1963	1762	19.23
2	21.300	22.100	11.630	0.005475	10.750	1964	1762	20.40
3	21.750	22.100	11.630	0.005590	10.950	1959	1758	18.86
4	21.328	22.100	11.630	0.005482	10.700	1952	1752	20.54
5	21.824	22.100	11.630	0.005609	10.930	1949	1749	18.26
6	21.850	22.100	11.630	0.005616	10.970	1953	1753	19.50
7	21.470	22.100	11.630	0.005518	10.780	1954	1753	17.66
8	21.582	22.100	11.630	0.005547	10.860	1958	1757	17.75
9	21.360	22.100	11.630	0.005490	10.710	1951	1751	17.30
10	21.574	22.100	11.630	0.005545	10.860	1959	1758	18.20
Avg.	**21.540**	**22.100**	**11.630**	**0.005536**	**10.829**	**1956**	**1755**	**18.77**
S.D.	**0.209**	**0.000**	**0.000**	**0.00005**	**0.099**	**5.126**	**4.600**	**1.133**
C. of V.	**0.97%**	**0.00%**	**0.00%**	**0.97%**	**0.92%**	**0.26%**	**0.26%**	**6.04%**

Hydraform Machine				**STABILISED**				
Total weight of each Mix =			**110 Kg**		**With 7% Cement**			
Water =	**11 Kg**							
BATCH 3	**M.C. =**	**10.76%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	22.928	22.100	11.630	0.005893	11.630	1974	1782	20.80
2	21.380	22.100	11.630	0.005495	10.770	1960	1770	19.24
3	21.480	22.100	11.630	0.005521	10.850	1965	1774	19.72
4	21.620	22.100	11.630	0.005557	10.870	1956	1766	18.20
5	21.690	22.100	11.630	0.005575	10.930	1961	1770	18.40
6	21.580	22.100	11.630	0.005547	10.890	1963	1773	18.00
7	21.880	22.100	11.630	0.005624	11.020	1960	1769	19.40
8	21.630	22.100	11.630	0.005559	10.870	1955	1765	16.40
9	21.340	22.100	11.630	0.005485	10.730	1956	1766	17.60
10	21.360	22.100	11.630	0.005490	10.750	1958	1768	18.20
Avg.	**21.689**	**22.100**	**11.630**	**0.005575**	**10.931**	**1961**	**1770**	**18.60**
S.D.	**0.467**	**0.000**	**0.000**	**0.00012**	**0.261**	**5.487**	**4.954**	**1.234**
C. of V.	**2.15%**	**0.00%**	**0.00%**	**2.15%**	**2.38%**	**0.28%**	**0.28%**	**6.64%**
BATCH 4	**M.C. =**	**11.39%**						
	Block	**Block**	**Block**	**Block**	**Block**	**Bulk**		**Indentation**
	Length	**Width**	**Height**	**Volume**	**Mass**	**Density**	**P.D.D.**	**Diameter**
Block No.	**(cm)**	**(cm)**	**(cm)**	**(m3)**	**(kg)**	**(kg/m³)**	**(kg/m³)**	**(mm)**
1	21.780	22.100	11.630	0.005598	11.040	1972	1770	20.20
2	21.800	22.100	11.630	0.005603	11.000	1963	1762	18.00
3	22.200	22.100	11.630	0.005706	11.080	1942	1743	19.20
4	21.370	22.100	11.630	0.005493	10.820	1970	1768	18.50
5	21.480	22.100	11.630	0.005521	10.850	1965	1764	19.00
6	21.700	22.100	11.630	0.005577	10.980	1969	1767	18.64
7	21.400	22.100	11.630	0.005500	10.750	1954	1755	18.00
8	21.770	22.100	11.630	0.005595	10.850	1939	1741	17.60
9	21.900	22.100	11.630	0.005629	11.050	1963	1762	17.40
10	21.980	22.100	11.630	0.005649	11.050	1956	1756	18.10
Avg.	**21.738**	**22.100**	**11.630**	**0.005587**	**10.947**	**1959**	**1759**	**18.46**
S.D.	**0.263**	**0.000**	**0.000**	**0.00007**	**0.118**	**11.443**	**10.273**	**0.838**
C. of V.	**1.21%**	**0.00%**	**0.00%**	**1.21%**	**1.08%**	**0.58%**	**0.58%**	**4.54%**

Appendix I - Indentation tester design



Domestic electricity generation using waterwheels on moored barge

Zoë Jones

M (Eng) Structural Engineering with Architectural Design

Supervisor: J.Wolfram

School of the Built Environment, Heriot-Watt University

2005

DECLARATION

I …Zoë Jones……………………. , confirm that this work submitted for assessment is my own and is expressed in my own words. Any uses made within it of the works of other authors in any form (e.g. ideas, equations, figures, text, tables, programmes) are properly acknowledged at the point of their use. A full list of the references employed has been included.

Signed: ……………Zoë Jones

Date: ……27th May 2005………………………..

Contents

Contents

List of Figures

List of Figures

Page Number

`

Acknowledgements

Julian Wolfram at Heriot Watt University
Brian Linfoot at Heriot Watt University
Phillip Clark at Heriot Watt University
Bruce Davies at Heriot Watt University
Scott Arthur at Heriot Watt University
Ian Grant at Heriot Watt University
David Mann at the Howsham Mill Trust
Gerald Muller at Queen’s University, Belfast
Julia Nolan, Heatherslaw Mill, Ford and Etal Estate
Nick Bard at Renewable Devices
Wim Klunne, at microhydropower.net, Dennis Buller and all on the microhydro yahoo group
Maxine Grant
Basem Sharba
Graham Smith
Maria Lucey
Eve Ritchie
Steven Groves
Duncan Johnston
Robert Ball
Paul Nichol

I would also like to thank Sakina, Amy, Jessie and Peter Jones, for their support and ideas, and friends on and off the course, for their help and patience.

Abstract

This dissertation aims to design a waterwheel for very low head sites in rural Scotland to provide domestic electricity supply. The dissertation uses MathCAD to design a stream wheel and Excel to design the vessel that the wheel is fixed to. Options for the gearbox and generator are briefly considered.

During research it became clear that one of the major challenges facing waterwheel development is a lack of scientific analysis and the general perception that waterwheels are anachronistic and irrelevant. To illustrate the potential of waterwheels three case studies are included; a new wheel in the developing world, a mill renovation in the UK and an old wheel that still powers a working mill.

Keywords: undershot waterwheels, stream waterwheels, basic barge design, basic catamaran design, microhydropower,

Glossary of Terms

Archimedian Screw- age old technology used to lift water from lower heights to irrigate fields. Currently being developed in reverse to produce electricity. Patent owned by Ritz Attro (www.ritz-attro.de)

Gantt Chart- graphical representation of the stages of a project against the estimated time taken to complete each stage.

PERT Chart- chart showing chronological order of the stages of a project, illustrating which stages rely on previously completed components.

Kyoto Protocol- a summit on climate change held in Kyoto in 1997 saw 160 nations sign up to this Protocol laying down targets for greenhouse gas emissions reduction.

Undershot wheel- a waterwheel powered by the kinetic energy of water running below the centre, which pushes the blades around.

Overshot wheel- water enters the wheel near the top and falls into buckets, turning the wheel using the potential energy of the water.

Breastshot wheel- similar to an overshot wheel the water enters about halfway up the height of the wheel.

MathCAD- computer program allowing users to input equations, functions and matrices, alter variables and perform mathematical processes such as integration, differentiation etc. quickly

Head- the difference in the water height either side of the waterwheel

Zuppinger or Poncelet wheel- undershot wheel with curved paddles that use both kinetic energy and, by channelling the water into curved blades potential energy. This leads to higher outputs and greater efficiencies.

Glossary of Terms

Renewable Obligation (Scotland) Bill- government bill introduced in 2002 forcing electricity suppliers to buy 10% of their supply from renewable sources

Critical flow- point at which the flow of the water changes from being streamline to turbulent, signalling the use of different equations, theories and testing procedures

Supercritical flow- beyond the critical flow here flow is turbulent with a Froude number of 1

Froude Number- ratio of the force on an element of fluid to the weight of the element. In mathematical terms $Fr = velocity \div (gravitationalcons\tan t \times characteristiclength)^{0.5}$

Nomenclature

π = Pi ~ 3.14

cos= Cosine function

x= Angle of blade arm at centre of waterwheel relative to the vertical

xL= Angle of blade arm at the centre of waterwheel relative to the vertical at the point of the blade leaving the water.

x1= Angle of blade arm at centre of waterwheel at point of blade beginning to leave the water

Current= Speed of the water horizontally

Vc= Component of the current acting in the blade's direction

Vb= Rotational velocity of the blade

Vr= Relative velocity of blade

Cd= Coefficient of drag

f = Width of blade

d = Depth of blade

L= Distance from centre of waterwheel to bottom of blade

y = Distance from centre of waterwheel to top of blade

Nomenclature

a(x)= Diagonal distance from centre of waterwheel to water line

D(x)= Depth of blade in water

A(x) = Surface area of blade in water

p = constant representing the proportion of velocity the blade picks up from the current

t = Time taken for one blade to complete one revolution

Larm= Lever arm from centre of waterwheel to the centre of the submerged area of blade

F(x) = Force on one blade moving from a vertical position to the point that the blade leaves the water

M(x) = Moment at waterwheel centre created by blade moving from a vertical position to the point the blade leaves the water

WorkDone= Work done on one blade from entering to leaving the water
N= Number of blades

TotalWorkDone= Total work done for entire wheel from entering to leaving the water

PowerAbsorbed= Total amount of power absorbed by wheel during one revolution

Side A= breadth of barge multiplied by the depth of the barge

Side B= length of barge multiplied by depth of barge

Nomenclature

q= distance from point of triangular barge end to the nearest side of barge rectangle

side F= breadth of the watertank multiplied by its depth

side E= length of the watertank multiplied by its depth

1 Introduction

1.1 Recent concerns over global warming and an over reliance on fossil fuels have led to an increased political, academic and public interest in renewable energy. The Kyoto Protocol sets strict standards for countries to limit their carbon emissions and research alternative energies.

1.2 Three viable areas of renewable energy have emerged- solar power, wind power and hydropower. This dissertation focuses on hydropower for a single domestic supply in rural Scotland.

1.3 Microhydropower refers to the production of 300 kW or less using turbines, waterwheels or Archimedian screws (1). With its low head and flow requirements, relatively low cost, "fish friendly" slow rotation, and ease of construction the waterwheel is experiencing a revival. They are especially relevant to small residential projects where the long payback period of turbines is prohibitive (1) and for developing countries, where maintenance and fabrication has to be simple. Three case studies are included in the literature review here to demonstrate waterwheels in use today.

1.4 This dissertation evaluates the typical energy usage of a hypothetical three bedroom household of two adults and two children and then models the dimensions and properties of the waterwheel required to produce this output. The wheel is to be situated on an open small river where there is no head difference and the flow velocity is approximately $2ms^{-1}$.

1.5 The waterwheel is fixed to a type of barge or catamaran. This allows the wheel to move with the change in water level insuring a reliable power output and removes the cost of building a separate channel or weir. It is assumed that the waterway is not travelled on by boats.

1.6 Scotland has been chosen as there is a plentiful supply of water and rainfall and a number of island and highland communities, who would benefit from small scale power generation, as connection to the grid is expensive and unreliable.

2 Literature review

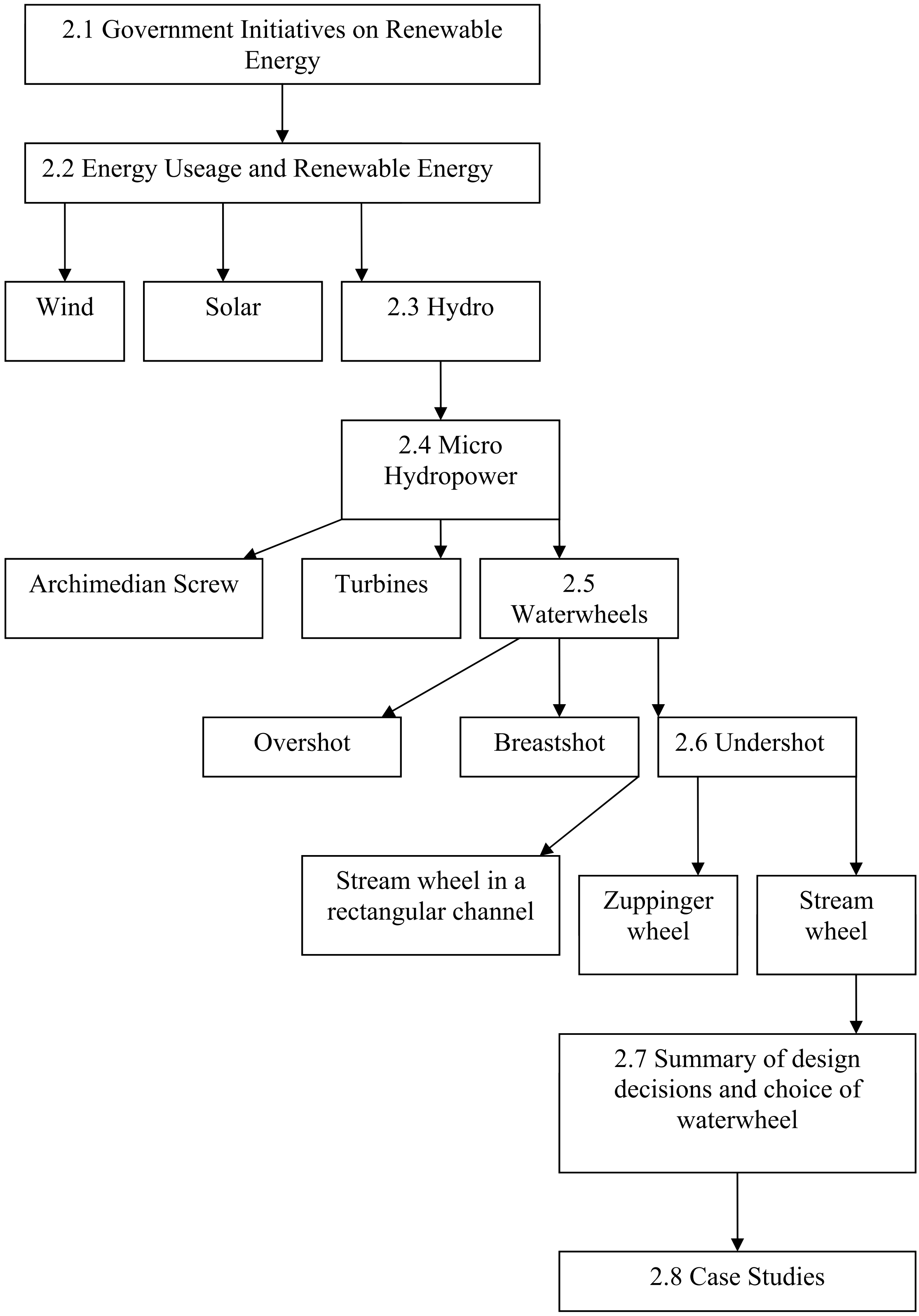


Fig. 1 Diagram showing process of research involved in the Literature Review

2.1 Government Initiatives on Renewable Energy

The UK government pledged under the 1997 Kyoto Agreement to cut greenhouse gas emissions by 12.5% during 2008-2012 leading to a 20% reduction on 1990 emission levels by 2010, in 5 years time (16). According to the 2003 Energy White Paper “Our energy future- creating a low carbon economy” the government aims for a 60% reduction in greenhouse gas emissions by 2050. (16).

2.11 In 2003 only 2.7% of the total energy used came from renewable sources. The Energy White Paper aims to increase this figure to 10% by 2010. This will mean approximately 10,000 MW of electricity from renewable sources, which is roughly equivalent to 3,000-5,000 wind turbines (16).

2.12 In Scotland the Renewables Obligation (Scotland) bill was passed in 2002 forcing the licensed electricity suppliers to begin investigating renewable energy sources and purchasing a percentage of their energy from renewable sources (16). This will hopefully lead to electricity suppliers offering higher buy-up prices for renewable energy, making small scale renewable installations able to pay for themselves.

2.13 The Scottish Climate Change Programme signs Scotland up to generating 18% of its energy from renewable sources by 2010, increasing that to 40% by 2020 (17), with a range of measures, new bodies and funding to promote energy efficiency, renewable energy research and raise public awareness.

2.2 Energy Usage and Renewable Energy Types

Man requires energy at a basic level for heat and light. In primitive times burning wood provided warmth, cooking facilities, light and a place of gathering. Over the last 2000 years man has developed more sophisticated machinery for heating, lighting and entertainment; however the burning of fossil fuels has continued leading to the current situation of climate damage and few alternatives to fossil fuels.

2.21 In the latter part of the last century concern over the rapidly depleting fossil fuel supplies focused world leaders to meet and set Co_2 emission limits. Renewable energy is defined as energy that "is derived from an inexhaustible (wind, sun, sea) or replaceable (waste products, crops) source" (12).

2.22 Inexhaustible resources include:

- **Wind Power**- differences in temperature across the globe cause differences in air density setting up winds that can be used to turn turbines generating electricity.

- **Solar Power**- the sun's energy can be used in photovoltaic cells, where a reaction causes electricity production or in a thermal or solar air heating system where water or air are warmed for heating.

- **Hydropower**- the hydrological cycle draws water from the sea into clouds, releasing rain onto high ground that then flows back to the sea along rivers, streams and tributaries. The flow of this water can be harnessed to turn the blades of a turbine, waterwheel, or the motor of an Archimedian Screw (1)

2.3 Hydropower

At the smaller end of the scale hydropower is classified as:

- **Picohydropower**- up to 10kW
- **Microhydropower**- 10kW-300kW
- **Small Hydropower**- 300kW-1000kW
- **Mini Hydropower**- above 1000 kW

(Taken from 13)

Within this dissertation the intention is to power one domestic dwelling in the UK making the power banding microhydropower.

2.21 Currently supplying only 0.8% (1456 MW) of the total energy consumption in the UK Hydropower has been much underrated, with hydroelectric capacity in this country being estimated at 4,244 MW. (14) The map below (Fig. 2) shows the high concentrations of hydroelectric installations in Scotland and Wales, areas with greatest head differences (15). Large scale dams are seldom built now as they are costly to maintain and their construction has huge environmental impacts, such as water diversion, altering of river slopes and infrastructure creation, all of which disturb aquatic ecosystems (18).

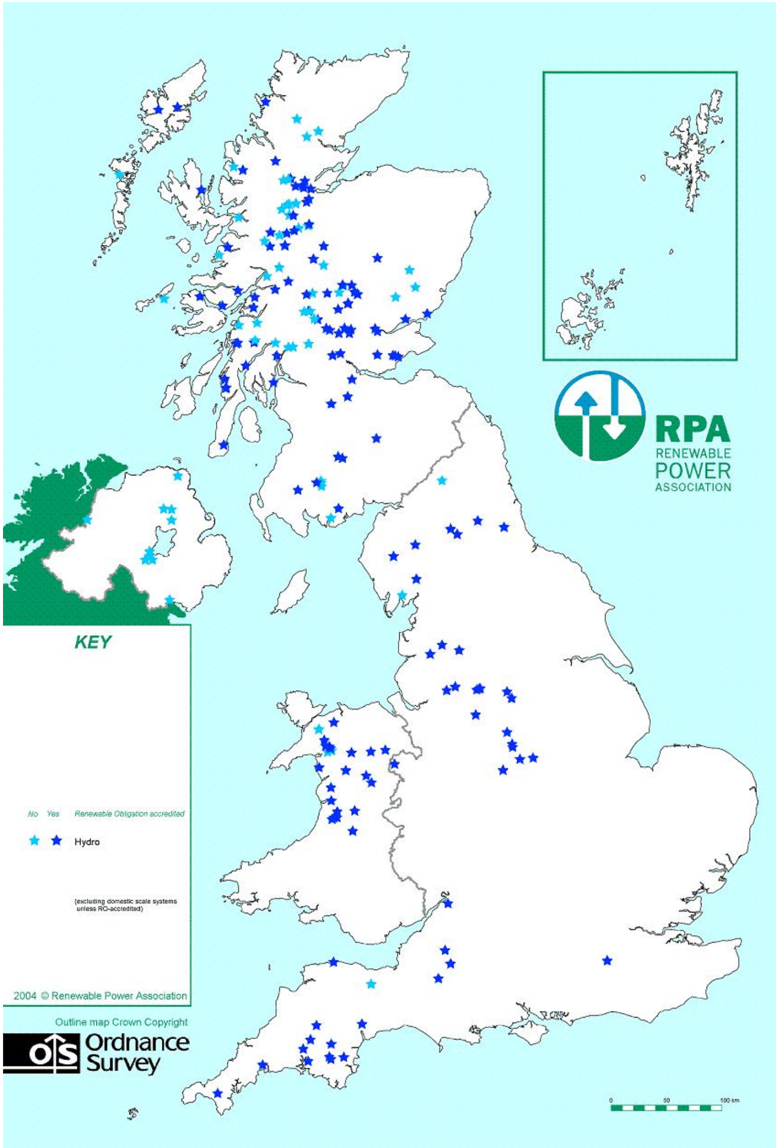


Fig. 2 Map of Hydroelectric Plants in the UK as of end 2004. The dark blue stars signify sites that have been renewable obligation certified and light blue stars are yet to gain certification. (Taken from 15)

2.4 Microhydropower

No clear estimates are available for microhyropower potential in the UK but some experts point to the 20,000 abandoned weir and watermill sites across the UK that could produce between 600 MW and 10,000 MW of power (2). Both the Department of Trade and Industry and the Scottish Executive seem to acknowledge the role that micro hydropower has to play-

" If small-scale hydroelectric power from all of the streams and rivers in the UK could be tapped, it would be possible to produce 10,000 gigawatt hours…. per year- enough to meet just over 3 per cent of our total electricity needs and making a significant contribution to the Government's renewables target…." Department of Trade and Industry, 2005 (14)

"There are…. an increasing number of proposals for small run of river hydro projects and these projects,…..will ensure that hydro will continue to play its part in Scotland's renewable energy mix." Scottish Executive Business and Industry, 2004 (19)

2.41 Yet despite this acknowledgement there has been little encouragement or official research into microhydropower since the "Small Scale Hydroelectric Generation Potential in the UK" Report by the Department of Energy in 1989. Frustratingly this only considered sites of more than 25 kW potential (and so ruling out waterwheels whose average power rating is 17.1kW) and with head differences of over 2m, immediately ruling out undershot and breastshot waterwheels (10). The report also discounted remote sites, with no grid connection as being uneconomical. These are exactly the kind of sites, where connection to the grid is too costly, that would benefit from the reliable independence that microhydropower can provide.

2.42 The publication of "The Layman's guide on how to develop a small hydro site" in 1997 by the Commission of European Communities (21) did promote microhydropower to the public indicating costs, environmental impacts and basic site evaluation methods. This type of document has encouraged a whole group of "do-it-yourself hydro developers" (22) who have experimented independently with different heads, waterwheel types and generation systems.

2.44 Microhydropower can be generated using:

- **Turbines**- Water is funnelled into enclosed systems of blades rotating about the x or y axis. (See figs 3,4,6)Turbines have been well researched and developed yielding high efficiencies, but are still high in cost and complex to manufacture.
- **Waterwheels**- an age old technology where water enters at either the top or middle or it acts along the base. Waterwheels rose in popularity during the industrial revolution, but declined as electrical power took over from mechanical power, and have remained largely ignored ever since.
- **Archimedian Screw**- used for hundreds of years as a motor to raise water from lower fields for irrigation. Recently there has been renewed interest as the screw can be reversed running water from above, through the screw, turning a motor and generating electricity. Sparse experimental evidence exists but efficiencies are estimated at 70-80% (1).

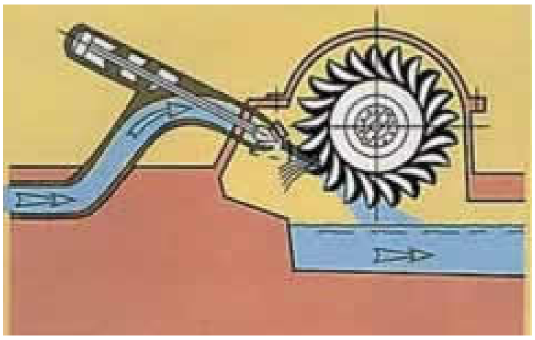


Fig. 3 Pelton turbine (44)

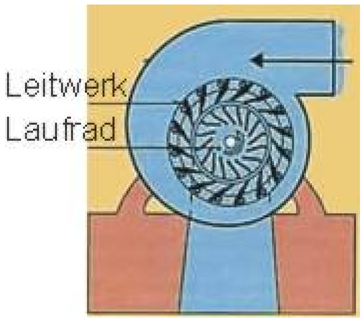


Fig. 4 Francis turbine(44)

Fig. 5 Archimedian Screws installed in a theme park (29)

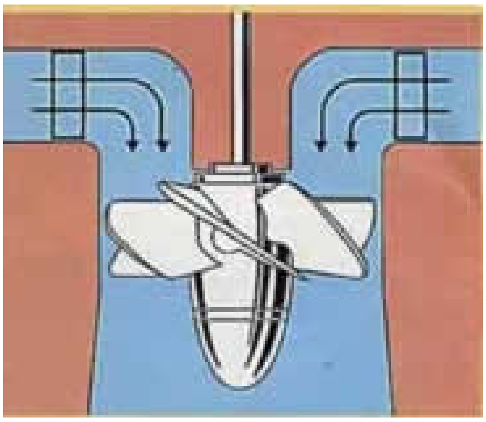


Fig. 6 Kaplan turbine (44)

2.5 **Waterwheels**

History of the Waterwheel

2.51 The inventor of the waterwheel is unknown but the undershot wheel is described by Vitruvius in 27BC (8). Initially the waterwheel was used to lift water and irrigate fields but was later used as a means to generate mechanical power for milling. The rapid industrialization of the Middle Ages led to an increase in waterwheel usage- 5000 mill sites are recorded in the Domesday Book of the 11th century .(2)

2.52 Interest in the waterwheel continued from the 12th to the 20th century (2) reaching a peak in the 19th century. In England in 1850 there were 25-30,000 waterwheels in operation, in Ireland at the same time there were 6,400 and in Germany there were 40,000. (5) Many of the wheels constructed were built using rules of thumb passed down rather than specific scientific analysis. Texts such as "Water or Hydraulic Motors" (1894) by Philip R. Bjorling (7) contain rough formulae and tables for estimating the number of buckets, the theoretical velocity and the power output.

2.53 However the waterwheel was an important energy source that began to attract scientific interest. In 1752-54 John Smeaton built scale models of undershot and overshot wheels to test their efficiencies (8). The French government offered large rewards for more efficient designs, leading to the Poncelet design that was later refined and patented by Zuppinger (8). New roles were devised for the wheel such as on steam paddle ships where fuel was burnt to evaporate water, and create steam, turning the paddles, pushing the boat forwards. (9)

2.54 Waterwheels continued to be used well into the last century; in Bavaria in Germany 7,554 wheels were operating in 1927 (1), and in Switzerland nearly 7,000 small scale hydropower stations were being used up to 1924 (4). However the "lack of strong and reliable gearing systems, coupled with the advent of steam power and the introduction of higher speed water turbines rapidly led to the demise of the waterwheel" (3). In the UK waterwheels were left to rust or were removed, weirs were forgotten about or destroyed, populations moved away from the streams to the city and working knowledge of waterwheels was lost.

2.56 **Advantages of Waterwheels**

- Simpler technology than turbines lending themselves to the developing world for local fabrication and maintenance.
- Fish will hopefully be able to pass through waterwheels unharmed (more research is needed on this) and expensive fish screens will not be necessary (23)
- Faster pay back periods than turbines and in some cases Archimedian screws (1)
- Unlike wind turbines there seems to be less public resistance to waterwheels as they are not so out of place in the countryside for example the virtual wheel in Fig. 7. In fact the public seem to be interested in waterwheels judging by the number of visitor centres that proudly posses one.

Fig.7 A typical weir with a virtual wheel installed (1)

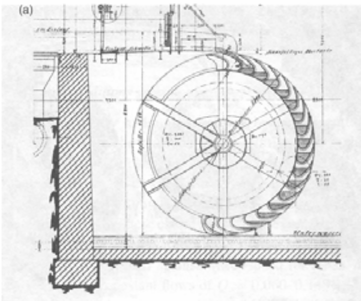
2.57 **Disadvantages of Waterwheels**

- The slow rotation of waterwheels (6-10 rpm) leads to high gearing ratios when trying to generate AC Electricity at 600- 1500 rpm. More research is needed into different gear boxes and configurations.
- Waterwheels do produce a "low *frequency thumping noise*" (25) which is not well understood and could cause complaints. It is thought that altering the blade shape to a "spoon-shape" would lessen the blow on impact whilst maintaining a high drag coefficient. This would be better investigated by experiment.
- A lack of knowledge in the engineering profession. Few model experiments have been carried out on waterwheels and there is still much unknown about the flow, efficiency and physical properties of waterwheel
- If wheels are fixed to the side of the river then changes in flow level will cause fluctuations in power output making them unreliable. If the wheel is fixed to a barge or catamaran then a build up of river bed silt or a reduction in the water level could lead to the blades becoming damaged hitting the river bed. Some sort

of high frequency measuring device would be needed on the barge to check the distance between the wheel and the river base, with regular maintenance checking wear and tear on the blades.

2.58 In this dissertation turbines and the Archimedian screw have not been investigated due to a lack of head at the site. Types of waterwheel include:

- **Overshot wheels**- 2.5m<Head<10m, Flow<0.2 m^3 s^{-1} per m width (1). Water enters above the wheel and falls into buckets turning the wheel with efficiencies of possibly 85% (1)



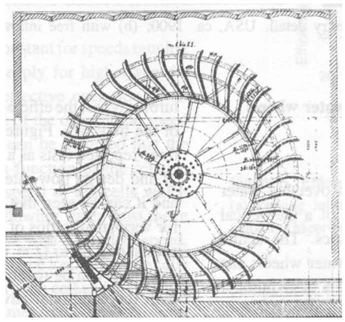
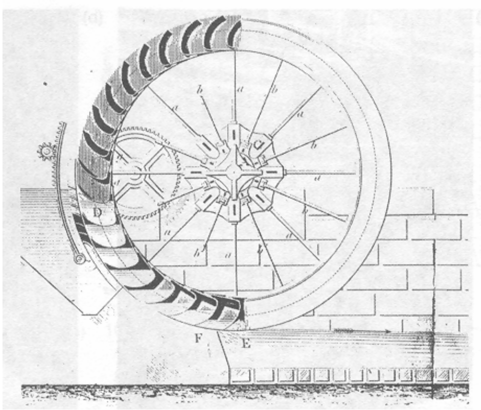


Fig. 8 Overshot wheel (24) Fig. 9 Undershot Zuppinger wheel (24)

- **Breast shot wheels**- 1.5m<Head<3m, 0.3<Flow<0.65 m^3 s^{-1} per m width (1). Water enters half way up the diameter of the wheel, falling into buckets turning the wheel. Recent experiments at Queens University, Belfast indicate efficiencies of up to 87% (23), making them a viable option for low head sites.



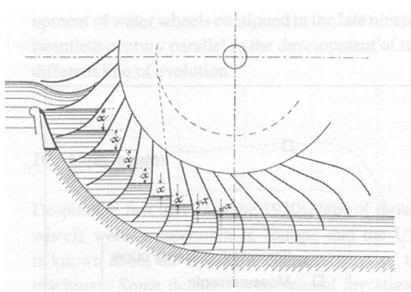


Fig. 10 Breast shot wheel(24) Fig. 11 Working principle for breastshot wheel(24

- **Undershot wheels**- 0.3m<Head<2.0m, 0.45<Flow<1 m^3 s^{-1} per m width (1). Some models use a very small head drop and curved blades to take potential energy from the river (60-77% efficiency (24)), others use the kinetic energy of the river on the blades (only 33% efficiency (8)).

2.6 Types of Undershot Wheels

Although considered inefficient even in the Industrial Revolution the undershot waterwheel continued to be manufactured as they could be sited on small streams in flatter areas, nearer to centres of population (8).There are several types of undershot wheel:

2.61 Zuppinger wheel

Designed by Walter Zuppinger and patented in 1883 this wheel uses only the potential energy of the river making it more efficient. The blades are curved carrying the water down a curved channel from a small weir and releasing it most efficiently, with minimal losses at the entrance or exit.

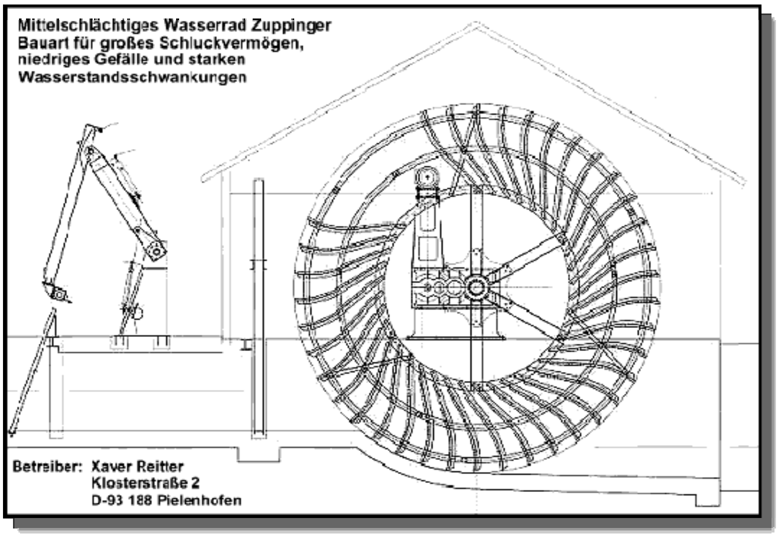


Fig. 12 above Sketch of installed Zuppinger wheel (27)

Fig. 13 right Same installed Zuppinger wheel (26)

Although sparse experimental data exists for all wheels Zuppinger wheels have perhaps been the most investigated. In 1979 students at the Technical University in Stuttgart, Germany tested an existing Zuppinger wheel that had been running since 1886. The test determined the flow rate and power output for a speed of 4.85 rotations per minute (rpm) and two flow rates of 1.48 and 3.1 $m^3 s^{-1}$. For Q/Qmax=0.5 the efficiency reached 77% and for Q/Qmax=1 efficiency reached 71% (24).

Figs 12 and 13 show a recently installed Hydrowatt Zuppinger wheel of diameter of 6.5m, a width of 2.3m, using a head difference of 1m to produce an electrical power output of 20kW at an overall efficiency of 70% (10). German based firm, Hydrowatt have built and installed 15 Zuppinger wheels with heads of 1-2.2m and power outputs of 4-45 kW from 1993 to 2001(10).

2.62 **Impulse or Stream Wheels**

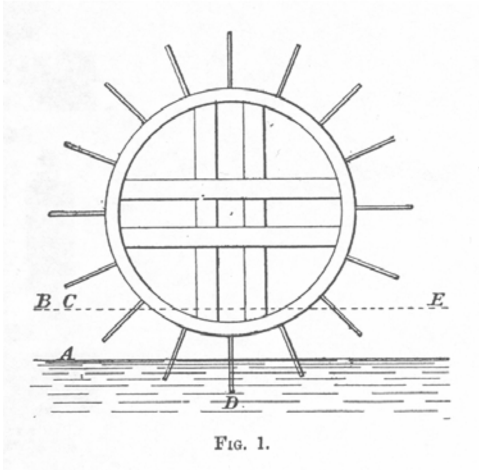


Fig.14 Diagram of midstream wheel ((7) pg 18)

Considered the least economic these large diameter wheels have flat paddles immersed in the flow and use the kinetic energy of the current, requiring zero head.

As this is less than the potential energy the stream wheel is often regarded as being inefficient, however interest has resurged in them as *"their application does not constitute a major change of river"* (25). No civil works are required, the wheel can be moored on a barge or fixed to the side, and the relatively simple dimensions and layout mean that it can be constructed and repaired locally, lending itself more to the developing world than the Zuppinger wheel.

The Universities of Southampton and Berlin TU have joined together to test a 500mm stream wheel shown in Fig. 15 (25) Testing is due to start this month and will investigate the design characteristics such as the power output against speed, the overall efficiency and differences in the upstream and downstream depths, with a view to building an actual stream wheel in Munich. Data on that stream indicates that the flow is supercritical at $5ms^{-1}$ with the depth being only 0.5m (39).

Fig. 15 Stream wheel model at Berlin Technical University, 2005 (53)

2.63 **Stream wheels in rectangular channels**

If an undershot wheel is positioned very near the base of the river bed, and is nearly as wide as the channel then the power output and efficiency will be increased as the flow is forced through a small space at high velocity, becoming critical.

An early investigation into this phenomenon was the Cairo University based paper "Design of momentum water wheels used for mini hydropower" published in 1985 (28). The test set up an undershot waterwheel close to the base of the river bed with a sluice gate beside it. Water is forced through the narrow opening under the gate, increasing the speed as it passes some of its kinetic energy to the wheel, then settling at a lower height and slower speed downstream (See Fig.16 below). The paper estimated that efficiencies of up to 63% were possible with this design.

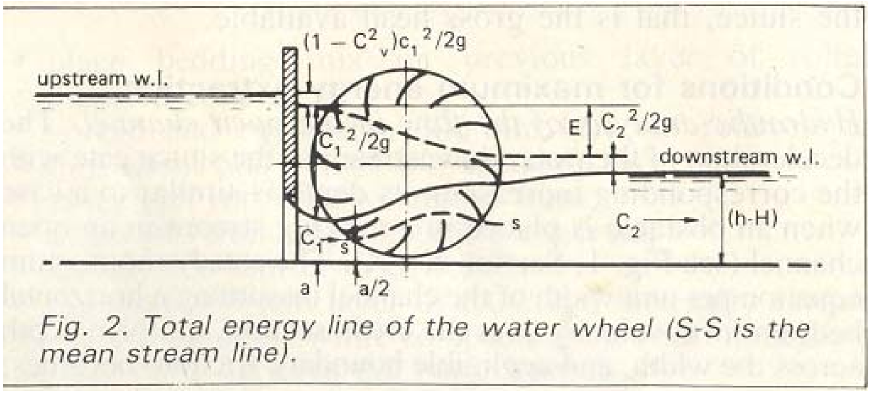
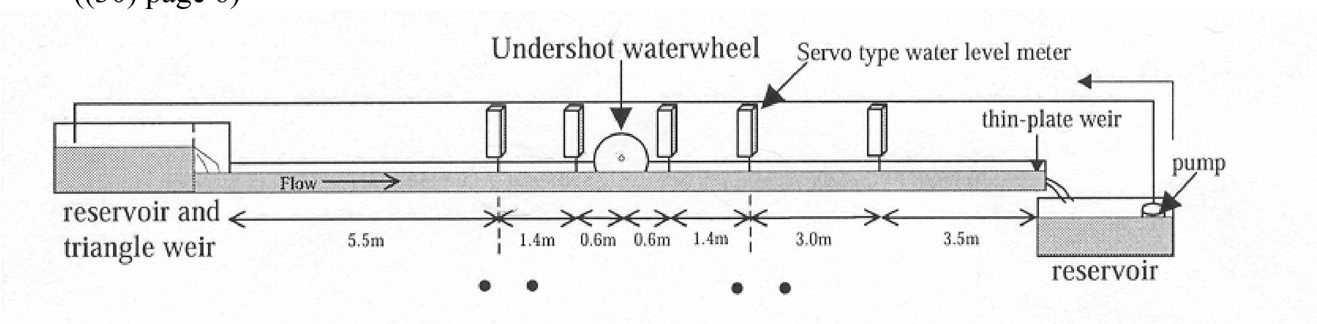


Fig. 16 Cross Section of wheel and sluice gate with water flow and energy lines marked on ((28) page 48)

A more conservative outlook was expressed by a Japanese paper (30) in 2001 where an undershot waterwheel was placed 3mm from the test bed and power output was measured by simply attaching weights to the wheel until it no longer turned. The tests were very comprehensive altering the upstream height, Froude number and blade heights finally concluding that 45% efficiency could be obtained provided the dimensions of the channel and flow lay within set parameters.

Fig.17 Cross section of rectangular channel experiment with undershot waterwheel ((30) page 6)

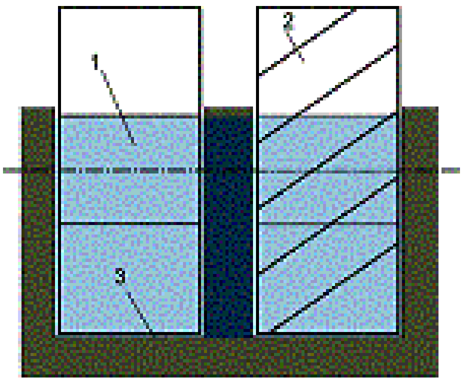


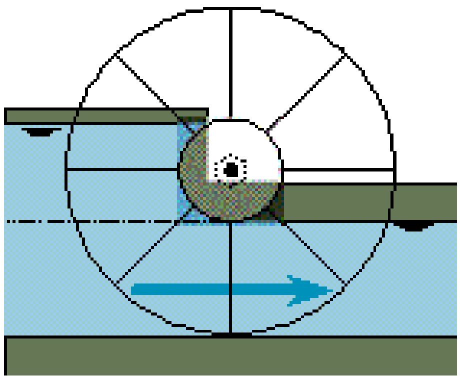
An Austrian engineer has shown interest in this style of wheel developing his own untested "Staudruckmaschine" (45). Roughly translated as "hydropressure machine" the wheel is shown below.

The development of these types of wheels is hampered by their strict requirements of narrow shallow streams with high flows, smooth beds and sides (reference 30 sites irrigation channels as being suitable for these wheels) , that are uniform enough to position a wheel in. The accuracy of construction and installation make them unsuitable for developing countries, and they would need a large trash rack to keep out debris. No study has been undertaken into how fish would navigate the wheel but with high velocities, and faster blade rotation this wheel could have a negative effect on aquatic life.

Fig. 18 (above) Photo of Staudruckmashine installed (45)

Fig. 19 and 20 (below) End view and cross section of wheel showing water (45)





2.7 Choice of waterwheel type

Seeing as the hypothetical site has no head difference a Zuppinger wheel would be unsuitable. The stream wheels fixed in rectangular channels require uniform channels with smooth beds and sides, which are not common in Scotland. So it was felt that a stream wheel would be most relevant to this dissertation requiring no head and being easy to fabricate.

2.8 Case Studies

Whilst researching this dissertation it became clear that waterwheel usage is not only held back by the lack of research and experimental evidence, high gearing ratios, etc. but also a perception amongst engineers and the public that an old technology could have no relevance to the present day. To help change this attitude the following case studies are included to illustrate scenarios where waterwheels have been installed or are used successfully.

2.81 **Heatherslaw Mill, Ford and Etal Estates, Northumberland**

Heatherslaw is a working mill grinding flour on the River Till in Ford and Etal Estate in the north east of England (see Fig…). The site has milled flour using hydropower since 1830 and is currently run by miller Mrs. Julia Nolan. The mill uses an undershot wheel 5m in diameter and 1.52m wide. Through a series of cogs, wheels and stones the rotating force of 20 horsepower (with sluice gate is fully open) transferred into mechanical power grinding grain into flour. The current wheel was constructed in the mid-1970's from wood partly because it is easier to replace and to also prevent sparks in the highly flammable mill.

Although the miller admitted that a Zuppinger-style wheel would be more efficient and produce more power the mill could not risk the loss of revenue in time spent waiting for specialist servicing or replacement part production.

So one and a half wheels were constructed locally (the half being for spare parts), and the wheel is maintained locally.

Fig. 21 right Heatherslaw Mill (Authors own, 2005)

The river Till is well known locally for having a fast flow, aided by its sand gravel river bed. The half dam on the mill side (seen in Fig. 22) was actually installed to hold water in reserve in case of low rainfall. Water is drawn just in front of the dam into a covered channel running under the bank. The channel turns about 90 degrees (causing a drop in velocity), passes the trash rack and then enters the mill.

Fig. 22 River bank and half dam
(Authors own, 2005)

Fig. 23 Entrance to waterwheel. The rectangular inlet channel can be seen, the sluice gate is just under the wooden walkway (Authors own, 2005)

Controlling the power output is important in order to mill different grains to different flour densities and so a sluice gate in front of the wheel controls the area of flow. There is also a sluice gate behind the wheel preventing back flow from storm surges jamming the wheel. At 10 rpm the wheels rotation is relatively slow allowing eels to pass through easily swimming to breeding grounds nearby.

Ford and Etal Estates receive a growing number of visitors each year to the mill and are keen to power the mills' lighting (it has no heating) with renewable energy. A research project with Newcastle University has concluded that using the wheel's outflow, and the considerable head drop there to turn a turbine would prevent altering the complex milling machinery and would raise visitor's awareness of another type of microhydropower. (37)

2.82 **Howsham Mill Project, Renewable Heritage Trust, North Yorkshire**

Sitting on an island in the river Derwent in Howsham this former watermill was been abandoned as a derelict shell (see Fig. 25 next page). The mill was built in about 1770 by the eminent architect John Carr of York in the Gothic Revival style of fussy details that was seldom used on functional buildings. This lead to Howsham mill being described as "a building of maximum historical interest" by an inspector for the Royal Commission of Historical Monuments some 40 years ago (31).

The mill closed in 1947 and despite its Grade II listing and presence on several buildings at risk registers the mill fell into disrepair and dereliction, the roof fell in and parts of the undershot waterwheel were stolen (31).

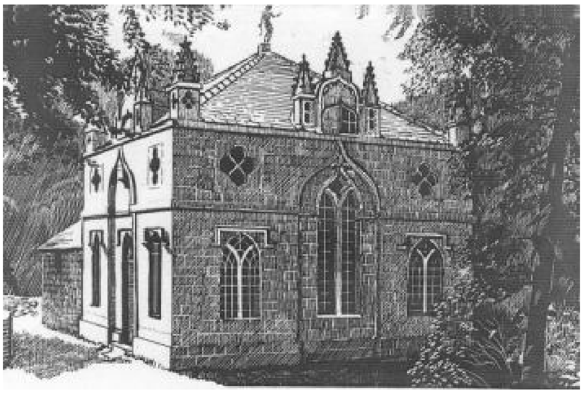


Fig. 24 Howsham Mill c.1945 (32)   Fig. 25 Howsham Mill before renovation (32)

A local charity called the Renewable Heritage Trust have formed to restore the mill, turning it into an educational centre about renewable energy and to reinstate a waterwheel to power the centre, giving the public a chance to see renewable energy close up (32). The Trust relies on grants and donations and the hard work of their volunteers to carry out much of the renovation work giving the project a feel of community involvement (See Fig. 26).

A research project with Gerald Muller at Queens University has been carried out over 2004-2005 to determine a suitable wheel design. From his tests into breast shot wheels and their efficiencies the Trust are hoping to install a breast shot wheel designed by him. The site has plenty of water and the centre only needs a small output (approx 0.5kW) and so feel that a breast shot wheel, being smaller in diameter than an undershot wheel, would rotate faster limiting gearing ratios and being cheaper to construct. There are also plans to install an Archimidian hydraulic screw in the sluice channel which would enable the Trust to sell some power back to the National Grid funding, further rebuilding. (38)

Fig. 26 Community renovation of Howsham Mill (46)

2.83 **Pedley Wheel Charitable Trust, Sri Lanka**

The Pedley Wheel Charitable Trust began in a similar grass-roots renewables vein as the Howsham Project. In 1991 the trust built a demonstration overshot waterwheel in Pedley Wood in Cheshire to interest young people in renewable energy.

After many attempts at solving the high gearing ratios the trust replaced the original tractor gearing systems they had with a chain configuration and an industrial gearbox (33). The trust also run their wheels as quickly as possible to minimise the torques and gearing ratios using about 65% of the power theoretically possible (34).

Fig. 27 An example of a Pedley Wheel (34)

In 1998 the trust visited Lower Amanawela in the high up Southern Uplands of Sri Lanka. Although situated at approximately 460 m above sea level (35) the village did have two discarded irrigation channels, one 5m below the other, providing an ideal low head site.

The trust then designed the 3.5m dia wheel in the UK, and had it built in Colombo whilst villagers built the powerhouse and civil works. To many critics surprise the wheel has worked well since installation producing 2.75 kW to light and heat 25 homes. There is also a stand alone gearbox allowing mechanical power for milling, woodwork, rice hulling etc. The electricity is administered by the Village Electricity Consumer's Society who maintain the system and issue each house with a time slot when they can exceed their allocated 100W usage (35).

Since this project the trust has completed four waterwheel projects in Sri Lanka and are planning four more (36) providing power to a diverse range of properties including houses, community centres and computer training facilities.

3 Technical Background

3.11 Waterwheel Design

Unlike overshot and breast shot wheels a stream wheel does not rely on a difference in head (See Fig. 28) but instead uses the potential energy of the water to generate power.

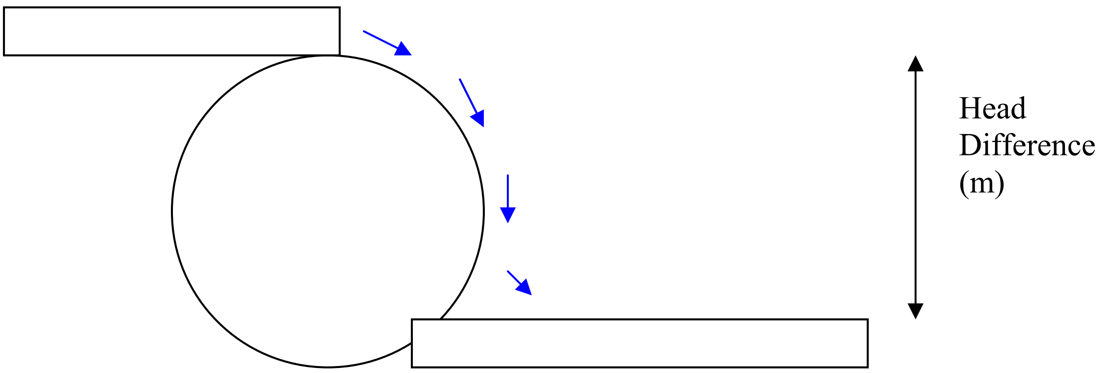


Fig. 28 Overshot wheel using potential energy and head difference to generate electricity

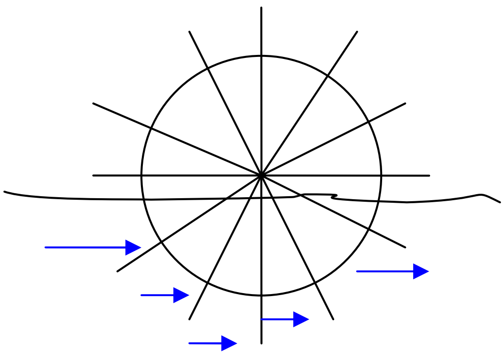


Fig. 29 Stream wheel using the kinetic energy of the flow to generate electricity

3.12 The Force on one paddle of a stream wheel is given by rearranging the drag equation to get : $F = 0.5 \times \rho \times Cd \times A \times V_r^2$ where ρ = Density of Water, Cd = Drag Coefficient, A = Area of blade in the water and Vr = velocity of the blade relative to the velocity of the water. MathCAD sheet can be viewed on enclosed CD

3.13 Determination of Cd

The drag coefficient for any object is influenced by its dimensions and journey through a fluid. From experimental data a variety of standard drag coefficients are known for set shapes. No experimental data exists on the drag coefficient of a waterwheel blade so here 1.5 has been used as an estimate, as this is the Cd value for a rectangular plate.

3.14 Determination of A

The area of the blade in the water obviously changes as the blade rotates about the central axis. To model this change mathematically the movement from a vertical position in the water to 90 degrees to the vertical is modelled. So if x = the angle at the wheels centre the blade's movement is modelled from x = 0 through x=x1 (when the blade begins to leave the water) to x = xL, (when the blade fully leaves the water). DraftT is the distance between the wheels centre and the top of the water, y is the distance from the wheels centre to the top of the blade and a(x) is the vertical distance from the wheel centre to the waterline.

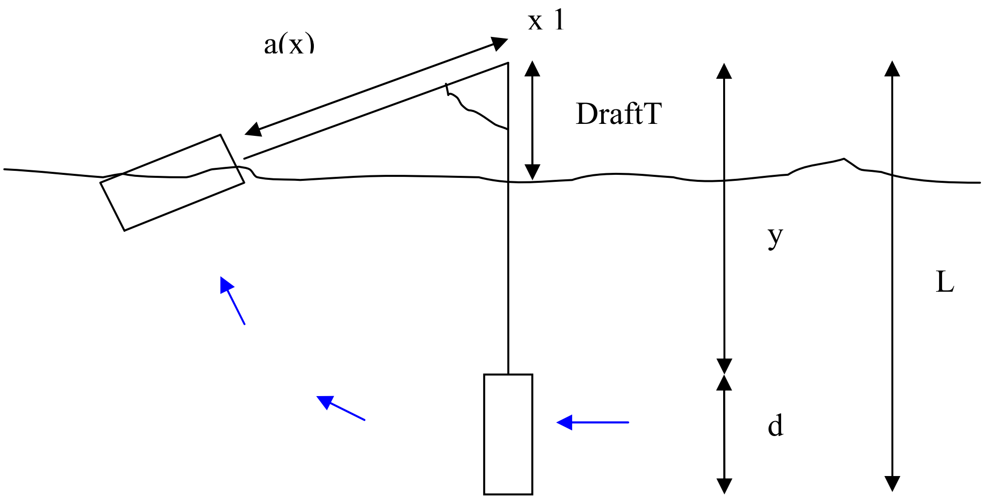
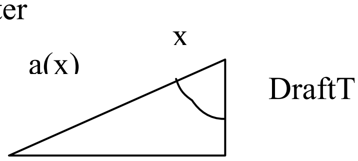


Fig. 30 Diagram showing the passage of one blade through the water

Using basic trigonometry a(x) can be expressed:



$$Cos\theta = adj \div hyp$$

$$Cosx = DraftT \div a(x)$$

Re-arranged to get: $a(x) = DraftT \div \cos x$

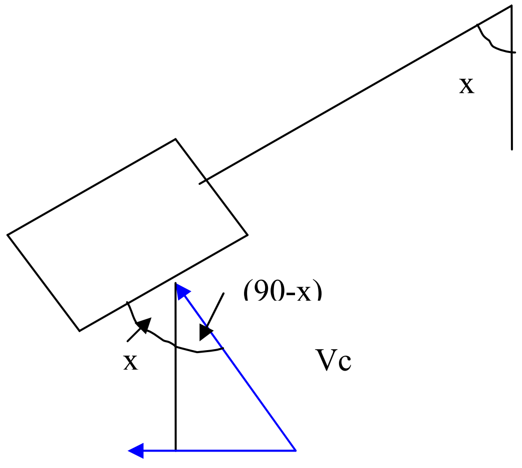
If a(x) is $\leq L - d$ then the paddle is still fully submerged and this area is given by: $A(x) = d \times f$ where f = width of the blade

However as the angle x increases towards x1 (when the blade begins to leave the water) a(x) increases and the submerged area decreases: $A(x) = (L - (a(x)) \times f$

3.14 Determination of Vr(x)

In order to express the relative velocity, Vr it is necessary to know the velocity of the blade, Vb and the velocity of the current in the direction of the blade, Vc. Vc is found by using trigonometry:

$$\sin\theta = opp \div hyp$$
$$\sin(90 - x) = Current \div V_c$$
$$V_c = Current \times \sin(90 - x)$$
$$V_c = Current \times \cos(x)$$



Fig… Diagram showing the current component in the blade's direction, Vc

The velocity of the blade, Vb is equal to the Current speed multiplied by a constant, p that represents the proportion of the Current the blade should absorb for maximum power output.

A way to think of p is to imagine the amount of work the water needs to do to turn a blade. If the wheel is allowed to rotate freely with the speed of the current then the water will easily push the blades around transferring no energy from water to blade- "freewheeling". If the wheels speed is less than the current then the water will have to push harder to turn the blades the same distance transferring energy to the blade.

If, however the wheels resistance increases too high then the water will find it too difficult to move the blades, instead flowing around and underneath them, leaving the wheel stationary.

This phenomenon was first described by Antoine Parent in 1704 who correctly identified that the optimum value for p was 1/3 (40).
This has been since proved by M.Denny in "The efficiency of overshot and undershot waterwheels" (8). In that paper the constant p is renamed c and the Power Output and Power Input are calculated. Efficiency = Power Ouput/Power Input and through cancelling out common terms, p is found to be 1/3 and the maximum efficiency, 33% (8).
Further evidence is provided in J. Wolfram's derivation in Appendix B and testing using the MathCAD worksheet on page…
In order to maintain this blade speed the generator would be calibrated to provide a set amount of resistance. Some waterwheel owners prefer to run the wheel faster than 1/3 of the current speed, in order to limit the gearing ratios involved, even at the expense of some efficiency (34)
So in conclusion,

$$V_c = V_c - V_b$$
$$V_c = (Current \times \cos x) - (p \times Current)$$

3.15 **Moment at the Waterwheel Centre**

The moment at the waterwheel centre is given by:
M (x) = F * LeverArm
Where LeverArm = distance from the centre of the wheel to the centre of the submerged areas of the blade
The work done between x = 0 and x = xL is equal to the integral of moment at the wheel centre. The work done by one blade from entering the water to leaving is given by:
WorkDone = 2 * ∫ M(x) dx
As the work done is measured in terms of radians not degrees a conversion factor of π/180 is used and the work done by the entire wheel is found using:
Total Work Done = N * Work Done * (π/180)

3.2 **Basic Barge Design**

In order for any object to float the downwards force that its weight supplies must equal the upward force supplied by the water. Archimedes found this upward force to be equal to ρgV where ρ is the density of the fluid displaced (usually water), g is the gravitational constant and V is the volume of the object underwater. This seems logical when considering cruise liners with hundreds of rooms, swimming pools, restaurants etc. that balance their large weight with many storeys being below the waterline.

However in order for a barge to float it also has to be stable. Too much swaying from side to side may lead to water entering and the barge sinking or capsizing.

The Centre of Gravity (CG), that the ships weight acts through must be in the same vertical line as the Centre of Buoyancy (CB), which is in the centre of the underwater volume. Swaying from side to side is can be caused by the CG moving if one side is heavier than another, this in turn will cause the CB to move to line up vertically with the new CG position. Both centres can be defined using 3 co-ordinates but for stability the Vertical Centre of Gravity (VCG) and Vertical Centre of Buoyancy (VCB) are used.

VCG represents the height that the entire weight of the ship acts through, normally measured from the base.

$VCG = \sum(MassesOfComponents \times IndividualCentreofGravity) \times \sum MassesOfComponents$

VCB is simply given by $DraftT \div 2$, and in this case the draft is decided by the waterwheel's requirements. In most barge design standard codes will be set as to the appropriate draft depth.

If a barge sways to one side due to wave action the Centre of Gravity will remain in the same place and the Centre of Buoyancy will move as the volume of barge underwater changes. If a vertical line is drawn from the original CB through CG upwards, intersecting a line from the new CB point they will meet at a theoretical point above the barge called the Metacentre, M. The distance BM is calculated using $Ixx \div V$, where Ixx is the second moment of area of the waterplane from the centre line axis (for a rectangular shape $= bd^3 \div 12$).

The Metacentric height, GM is given by $= VCB + BM - VCG$ and is the standard value used to determine a vessel's stability. An iterative process is used firstly inputting dimensions similar to previous designs, balancing the applied forces and then insuring GM is within acceptable limits.

4 MathCAD Model

The following flow chart describes the design process for the waterwheel model. Design is very much an iterative process of refining and feeding in new ideas as they come to light

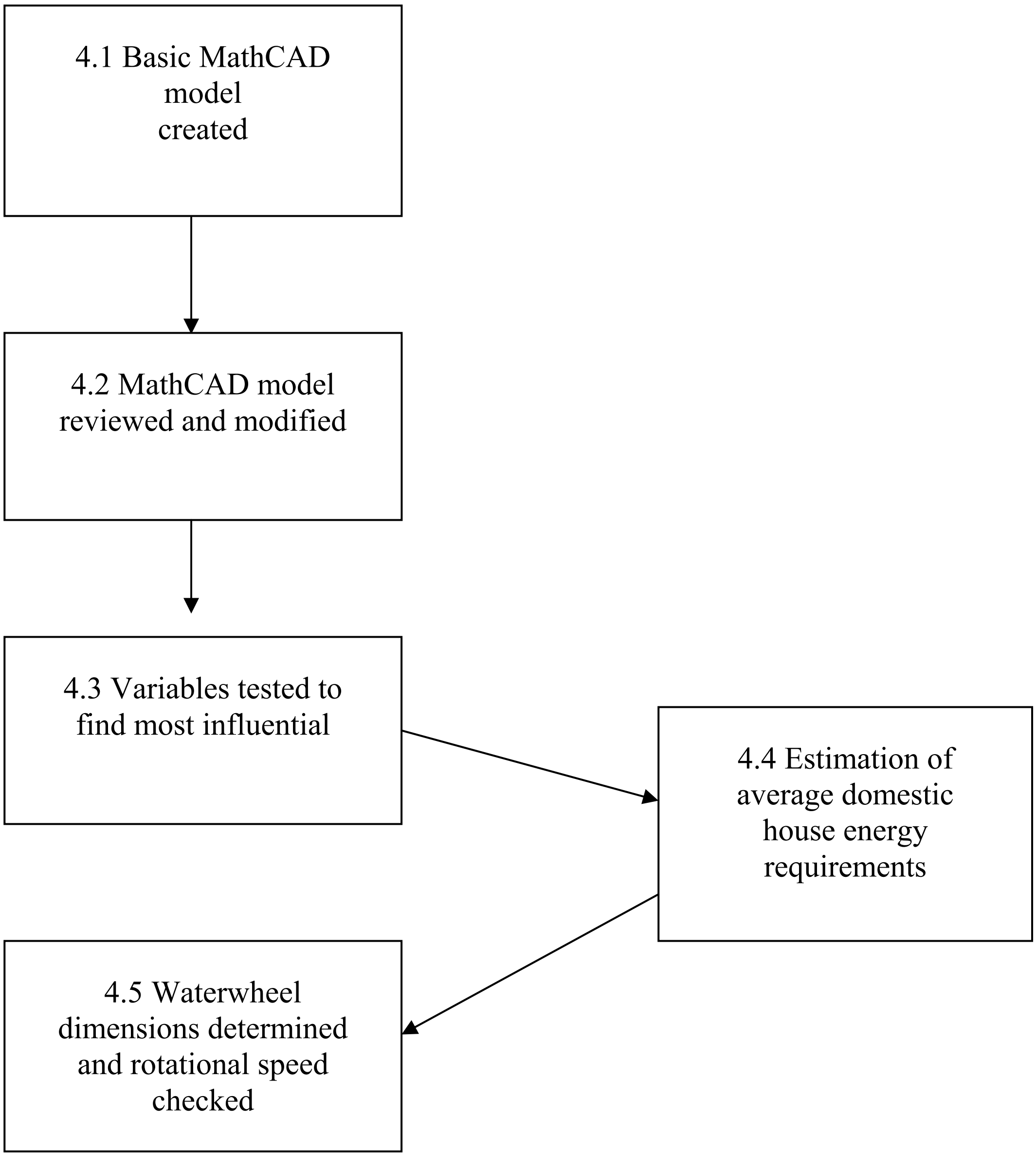


Fig. 32 Design process for waterwheel

4.1 Basic MathCAD Model

Using the theory explained in the Technical Background a basic MathCAD model was created describing the movement of a blade from the vertical position to the point when it leaves the water (see enclosed CD for copy). A copy of this early model appears on the next five pages with user inputs highlighted in red and observations highlighted in blue.

Initial assumptions included:

- The angle at which the blade leaves the water, xL is determined by the designer as 45 degrees.
- The area of the blade submerged decreases proportionally as the angle, x increases.
- The constant, p will be kept at the optimum 1/3 by later generator selection and design.
- The top of the blade sits at the waterline so once x is larger than zero the blade begins leaving the water (see fig. 33 below)
- The rate of rotation of the blade is constant; there is no acceleration or deceleration.
- The drag coefficient, Cd equals 1.5.
- The velocity of the Current is constant across the blades surface

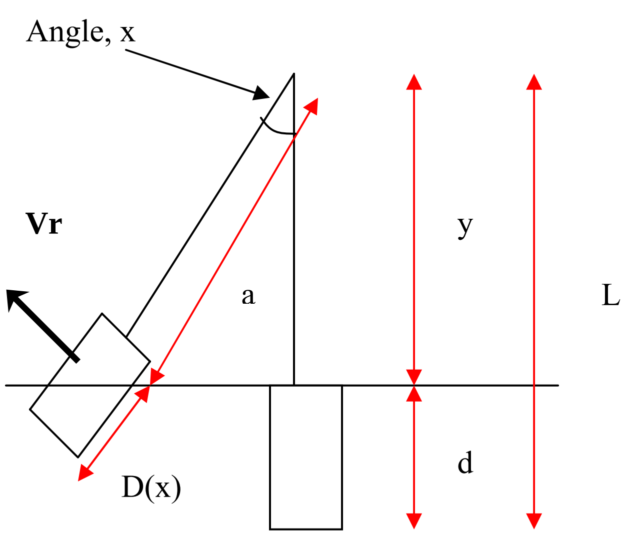


Fig. 33 Rotation of one blade through water

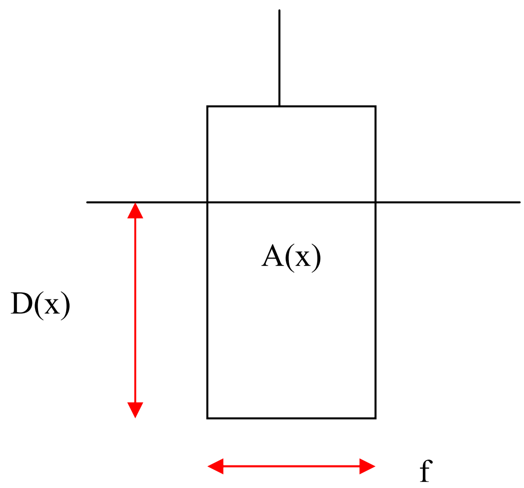


Fig. 34 Cross section of a blade

Fig. 35. Initial MathCAD design

Expressing Vb and Vc

$t := 1($ t= time taken for one blade to rotate 360 degrees

$Current := 2$ Current= velocity of the river

$x := 0..45$ x= angle at the centre of water wheel

$xL := 45$ xL= angle at the centre of the waterwheel as the blade leaves the water

$Cd := 1.2$ Cd= Drag Coefficient

$\rho := 100($ ρ=Density of Water

In this early design model it was decided that the user would specify the time for one blade to rotate 360 degrees. This was later changed to be derived from the blades velocity.
The angle that the blade would leave the water was also specified by the user regardless of the wheels dimensions. There is no DraftT value incorporated so the top of the blade is in line with the waterline, meaning the area decreases as x increases and so the power output was low.

Expressing Area, A

The dimensions of the wheel are decided by the user based on aesthetics, site conditions and minimising cost

$x := 0..45$

$f := 0.30$ f= width of the blade

$d := 0.5$ d= depth of blade

$L := 1.3$ L= distance from centre of waterwheel to bottom of blade

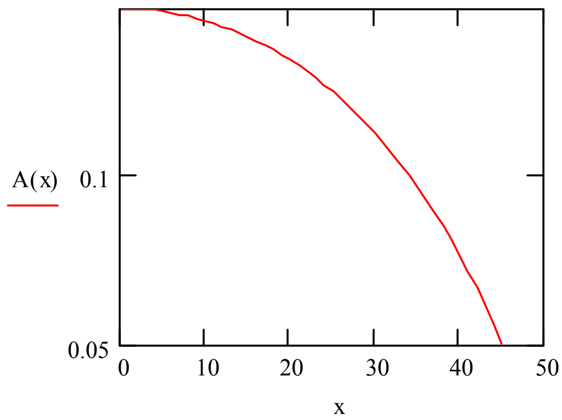
$y := L - d$ y= distance from centre of waterwheel to top of blade

$y = 0.8$

$a(x) := \dfrac{y}{\cos(x \cdot deg)}$ a=diagonal distance from the centre of the waterwheel to the water line

$D(x) := L - a(x)$ D(x)= depth of blade in water

$A(x) := D(x) \cdot f$ A(x)= surface area of blade in water



As the Force output is based on the Area this graph will give a low output as the area is decreasing with x. A more optimum design would have a constant area for as long as possible and then a quick drop in submerged area

Expressing Vr

The lever arm changes as the blade leaves the water and the submerged area decreases

$$\mathrm{Larm}(x) := L - \left(\frac{D(x)}{2}\right)$$

Larm is the lever arm from the centre of the waterwheel

$$p := \frac{1}{3}$$

p = the optimum proportion of velocity the blade absorbs from the current. Constant.

$$\mathrm{Vb}(x) := \mathrm{Current}\cdot p$$

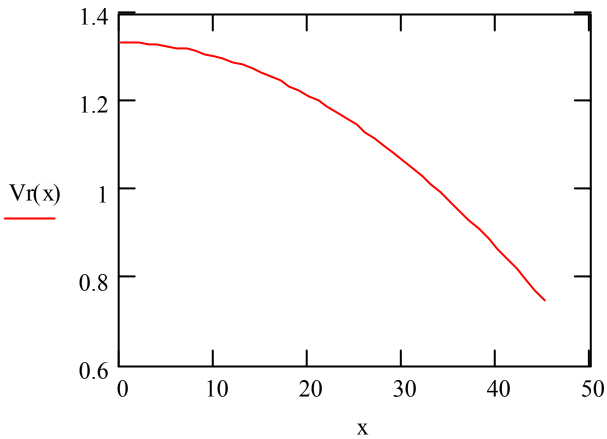
Vb(x)=rotational velocity of the blade

$$\mathrm{Vc}(x) := \mathrm{Current}\cdot\cos(x\cdot\deg)$$

Vc(x)= component of the current in the blade's direction

$$\mathrm{Vr}(x) := \mathrm{Vc}(x) - \mathrm{Vb}(x)$$

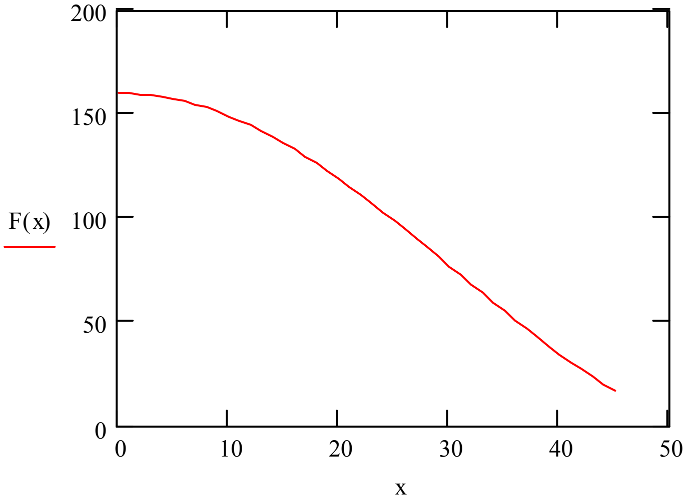
Vr= relative velocity



As expected the relative velocity drops as the blade begins to leave the water

Finding the force on the blade

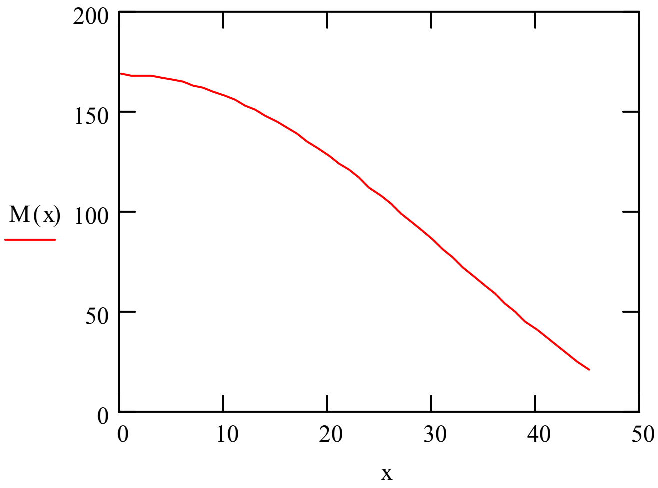
$F(x) := 0.5 \cdot \rho \cdot Cd \cdot (Vr(x))^2 \cdot A(x)$ F(x)= Force on the blade



Finding the Moment at the waterwheel centre

$M(x) := F(x) \cdot Larm(x)$ M(x)= Moment at the waterwheel centre

Both graphs seem consistent with less Force and Moment being produced as the blade leaves the water



Expressing the Work Done and Power Absorbed

Workdone in the 2nd half of travel = Work done in the first half of travel, so the Total Work Done on one paddle moving through the water= 2* the integral of the Moment at the centre of the wheel

$$\text{WorkDone} := 2\cdot\int_0^{xL} M(x)\,dx$$

WorkDone1= work done on 1 blade whilst moving through the water

$$N := 7$$

N= number of blades

$$\text{TotalWorkDone} := N\cdot\text{WorkDone}\cdot\left(\frac{\pi}{180}\right)$$

TotalWorkDone = total work done for the whole wheel. A conversion is needed to change degrees into radians

$$\text{PowerAbsorbed1} := \frac{\text{TotalWorkDone}}{t}$$

$$\text{PowerAbsorbed1} = 120.584$$

$$\text{PowerAbsorbed} := \frac{\text{PowerAbsorbed1}}{1000}$$

PowerAbsorbed (kW) = total amount of power absorbed by the wheel during one movement through the water.

$$\text{PowerAbsorbed} = 0.121$$

4.20 Reviewing and Modification of MathCAD Model

The basic model had a number of inaccurate features that could be improved; a copy of the improved worksheet can be seen over the next four pages. Improvements included:

- Introduction of a waterline and a DraftT value which then allows the designer to alter how low in the water the wheel is sitting.
- The exit angle, xL is now defined in terms of the draft and the diameter, L allowing a more integrated design.
- The change in submerged area is also linked to the draft and the model has been updated so the area is constant until the blade begins to leave the water.
- By rearranging time = distance/speed in terms of rotational distance/blade velocity a more accurate value for t is produced.

Fig. 36 Modified MathCAD design

Expressing Vb and Vc

$Current1 := 2$

$DraftT := 0.4$ DraftT= Distance between the axis of the waterwheel and the top of the water

$L := 4.5$ L= distance from centre of waterwheel to bottom of blade in metres

$$xL := \left(\operatorname{acos}\left(\frac{DraftT}{L}\right)\right)\cdot\left(\frac{360}{2\pi}\right)$$

$xL = 84.9$ xL= angle at the centre of the waterwheel as the blade leaves the water

$x := 0 .. xL$ x= angle at the centre of water wheel in degrees

$Cd := 1.5$ Cd= Drag Coefficient

$\rho := 1000$ ρ=Density of Water in kgm^-3

A more realistic design is created by inputting the waterline and noting the Force created from x = 0 to x = xL.

$f := 4$ f= width of the blade in metres

$d := 3.5$ d= length of blade in metres

$L := 4.5$ L= distance from centre of waterwheel to bottom of blade in metres

$y := L - d$

$y = 1$ y= distance from centre of waterwheel to top of blade in metres

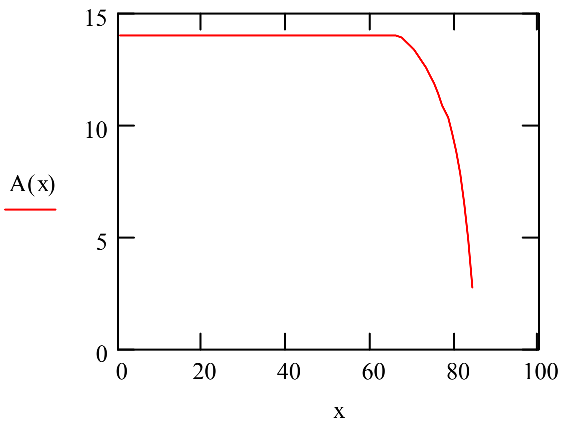
DraftT= Distance from centre of waterwheel to the water line when x=0

$a(x) := \frac{(\mathrm{DraftT})}{\cos(x \cdot \deg)}$ a(x) =diagonal distance from the centre of the waterwheel to the water line in metres

$$D(x) := \begin{vmatrix} d & \text{if } L - a(x) > d \\ (L - a(x)) & \text{otherwise} \end{vmatrix}$$

D(x)= depth of blade in water in metres

$A(x) := D(x) \cdot f$ A(x)= surface area of blade in water in m^2



The inclusion of a DraftT term allows the Area to be accurately calculated. The graph above shows the blade area being constant, as it moves through the water and then suddenly dropping as it leaves the water

Expressing Vr

p=proportion of speed blade takes up from river

$p := 0.33$

$$t := \frac{2 \cdot \pi \cdot L}{p \cdot Current1}$$

t= time taken for one blade to do one revolution (seconds)

Calculation of t is now linked to values of L and the Current speed

$t = 42.84$

$Vb := p \cdot Current1$

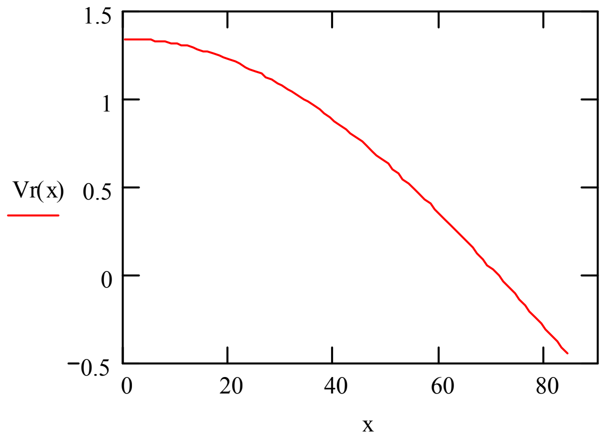
Vb(x)=rotational velocity of the blade in ms^-1

$Vc(x) := Current1 \cdot \cos(x \cdot deg)$

Vc(x)= component of the Current in the blade's direction in ms^-1

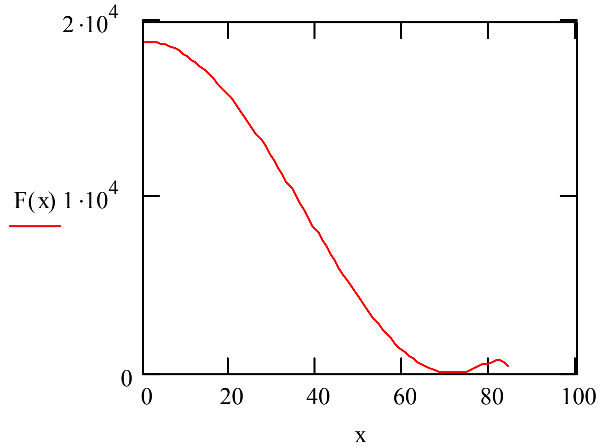
$Vr(x) := Vc(x) - Vb$

Vr= velocity of the blade relative to the speed of the Current in ms^-1



Finding the force on the blade

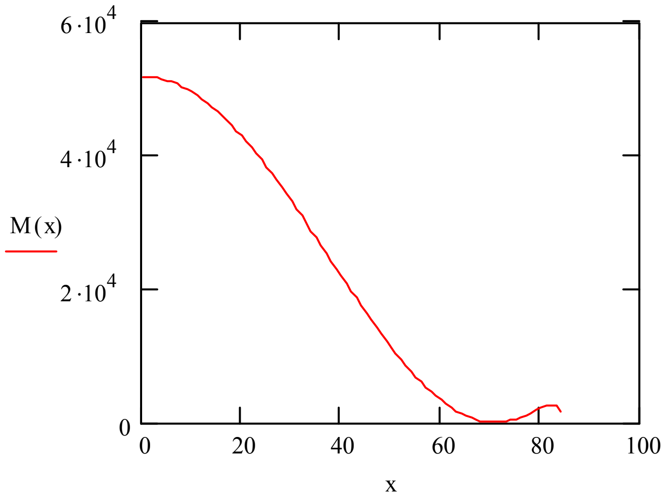
$F(x) := 0.5 \cdot \rho \cdot Cd \cdot (Vr(x))^2 \cdot A(x)$ F(x)= Force on the blade in kgms^-2 or Newtons



Finding the Moment at the waterwheel centre

$Larm(x) := (L) - \left(\frac{D(x)}{2}\right)$ Larm is the lever arm from the centre of the waterwheel in metres

$M(x) := F(x) \cdot Larm(x)$ M(x)= Moment at the waterwheel centre in kgm^2s^-2 or Nm



Expressing the Work Done and Power Absorbed

Workdone in the 2nd half of travel = Work done in the first half of travel, so the Total Work Done on one paddle moving through the water= 2* the integral of the Moment at the centre of the wheel

$$WorkDone := 2 \cdot \int_0^{xL} M(x)\,dx$$

WorkDone= work done on 1 blade whilst moving through the water in Nm

$$N := 16$$

N= number of blades

$$TotalWorkDone := N \cdot WorkDone \cdot \left(\frac{\pi}{180}\right)$$

TotalWorkDone = total work done for the whole wheel in Nm

$$PowerAbsorbed1 := \frac{TotalWorkDone}{t}$$

$$PowerAbsorbed1 = 2.477 \times 10^4$$

PowerAbsorbed1 = total amount of power absorbed by the wheel during one movement through the water in Nms^-1or Watts

$$PowerAbsorbed := \frac{PowerAbsorbed1}{1000}$$

PowerAbsorbed= total amount of power absorbed by the wheel during one movement through the water in kNms^-1 or kWatts

$$PowerAbsorbed = 24.771 \text{ kW}$$

4.21 **Further Modification**

It can be seen from the graph of Vr(x) against x that there comes a point when the velocity appears to be negative causing an upturn in the Force against x graph (below).

Considering Vr = Vc – Vb

= (Current * cos x) – (Current * p)

= (cos x) – p

So in that case when cos x = 1/3 then Vr will equal zero. This will occur regardless of the current at approximately 70.5 degrees (see graph below).

Fig. 37 Further Modification of MathCAD sheet

$p := 0.33$ p=proportion of speed blade takes up from river, proved in Appendix

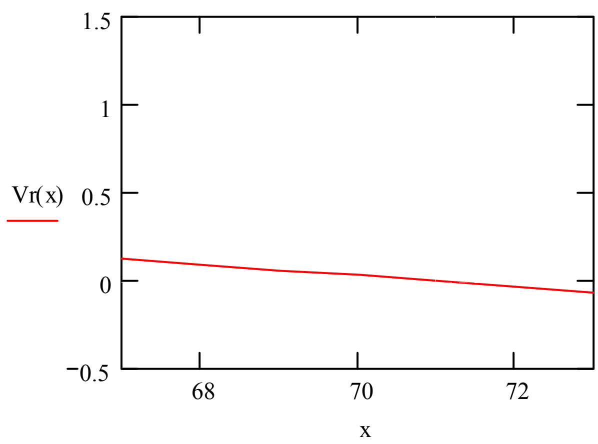
$Vb := p \cdot Current1$ Vb(x)=rotational velocity of the blade in ms^-1

$Vc(x) := Current1 \cdot \cos(x \cdot deg)$

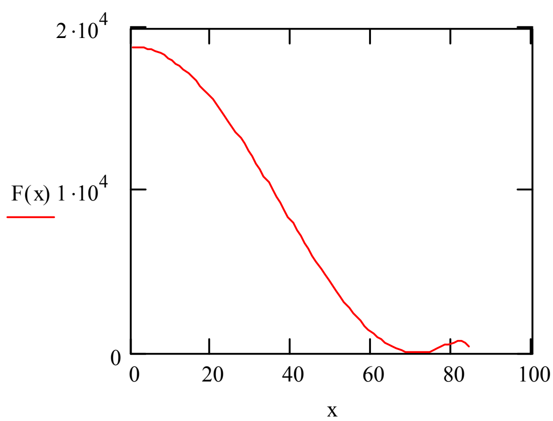
Vc(x)= component of the current in the blade's direction in ms^-1

$Vr(x) := Vc(x) - Vb$

Vr= velocity of the blade relative to the speed of the Current in ms^-1



$F(x) := 0.5 \cdot \rho \cdot Cd \cdot (Vr(x))^2 \cdot A(x)$



This can be physically reconciled by considering the blade's path through the water (See Fig. 38). When the blade is vertical in the water the Current velocity acts at right angles to the blade, generating the most force. As the blade rises the angle changes enabling more water to flow under the blade not passing on energy. At 70.5 degrees the current passes on no more energy to the blade managing to escape around the base and sides. Above 70.5 degrees its is suggested that the system may begin to lose energy as the blade push aside water above it to reach the surface.

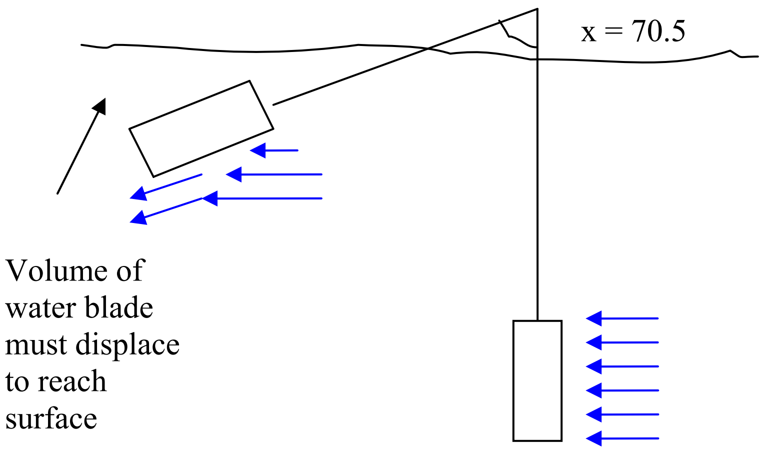


Fig. 38 Diagram showing blades at 0 degrees and 70.5 degrees.

This implies that the blade need not be in the water past 70.5 degrees, which in turn decides the optimum Draft for the system;

$L := 5.5$ L= distance from centre of waterwheel to bottom of blade in metres

$x1 := 70.5$ x1= the optimum angle for the blade to begin to leave the water

$y := 2$ y= the lever arm distance from the waterwheel centre to the top of the blade

$$DraftT := \cos(x1 \cdot deg) \cdot y$$

$DraftT = 0.668$ DraftT= Distance between the axis of the waterwheel and the top of the water

$$xL := \left(\operatorname{acos}\left(\frac{DraftT}{L}\right)\right) \cdot \left(\frac{360}{2\pi}\right)$$

$xL = 83.028$ xL= angle at the centre of the waterwheel as the blade leaves the water

$x := 0 .. xL$ x= angle at the centre of water wheel in degrees

By adjusting the Vr equation all negative values of Vr are ignored, removing the upturn in the Force graph;

$Vb := p \cdot Current1$

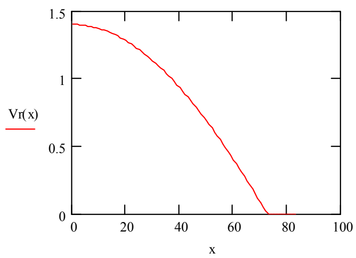
Vb(x)=rotational velocity of the blade in ms^-1

$Vc(x) := Current1 \cdot \cos(x \cdot deg)$

Vc(x)= component of the current in the blade's direction in ms^-1

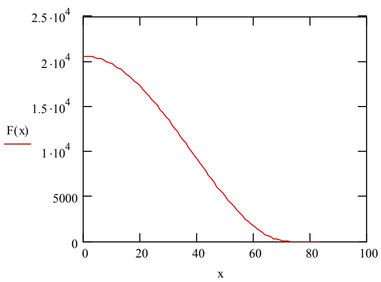
$Vc(30) = 1.732$

$$Vr(x) := \begin{cases} (Vc(x) - Vb) & \text{if } (Vc(x) - Vb) > 0 \\ 0 & \text{otherwise} \end{cases}$$



$$F(x) := 0.5 \cdot \rho \cdot Cd \cdot (Vr(x))^2 \cdot A(x)$$

F(x)= Force on the blade in kgms^-2 or Newtons



The only assumptions still in the final worksheet were:

- The drag coefficient, Cd equals 1.5.
- The rate of rotation stays constant, there is no acceleration or deceleration of the blades
- The velocity of the Current acts constantly across the blades surface
- The forces experienced by the blade are simplified not considering the effects of turbulence, currents or loss of energy through splashing.
- Blades are flat, rectangular blocks
- Constant p is kept at 1/3 by later generator selection and design

4.2 Variables Testing

By altering the input variables individually by 50% and noting the corresponding Power Absorbed the most influential variables can be found, leading to a more complete understanding of how they influence power output.

4.31 Initial Values- number of paddles, N =7

Current= 2 ms^-1

DraftT= 0.8m

Width of blade, f= 0.3m

Depth of blade, d= 0.5m

Length from axis to base of blade, L= 1.3m

Lever arm, y= 0.8m

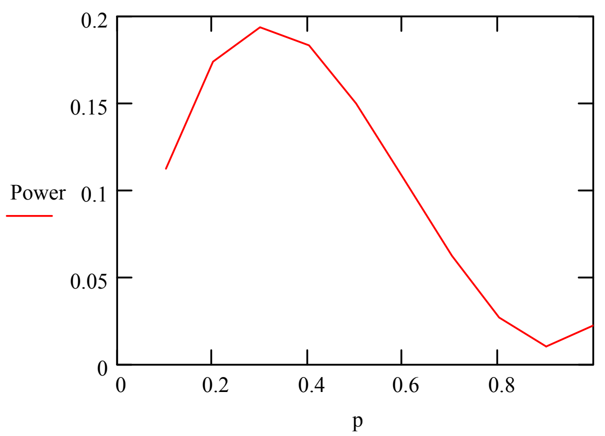
Constant, p= 0.33

Angle blade leaves the water, xL= 52.02 deg

This gives an initial Power Output of 0.12 kW

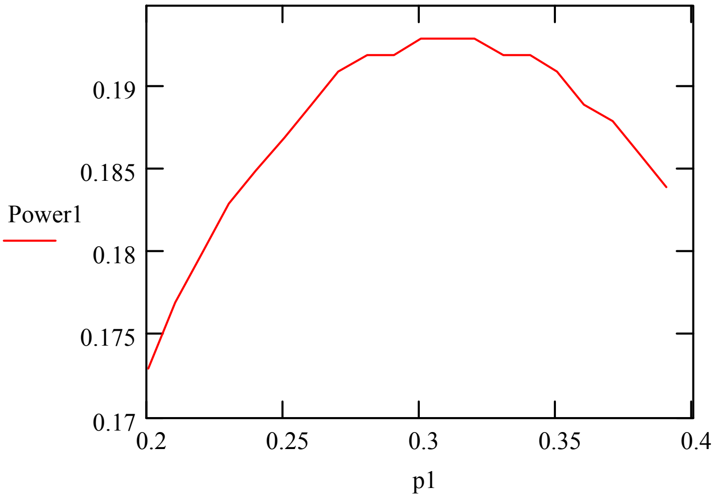
4.32 **Test 1.0- Confirmation of p=0.33 value**:

i) Values of p from 0-1.0 were plotted against power output and the maximum was found to lie between 0.2 and 0.4



$$p := \begin{pmatrix} 0.1 \\ 0.2 \\ 0.3 \\ 0.4 \\ 0.5 \\ 0.6 \\ 0.7 \\ 0.8 \\ 0.9 \\ 1.0 \end{pmatrix} \qquad Power := \begin{pmatrix} 0.112 \\ 0.173 \\ 0.193 \\ 0.183 \\ 0.15 \\ 0.107 \\ 0.062 \\ 0.027 \\ 9.896 \cdot 10^{-3} \\ 0.022 \end{pmatrix}$$

ii) Values of p from 0.20-0.39 were plotted against Power Output values (kW) with the maximum occurring at 0.3 confirming the proof work in the Technical Background chapter



$$p1 := \begin{pmatrix} 0.20 \\ 0.21 \\ 0.22 \\ 0.23 \\ 0.24 \\ 0.25 \\ 0.26 \\ 0.27 \\ 0.28 \\ 0.29 \\ 0.30 \\ 0.31 \\ 0.32 \\ 0.33 \\ 0.34 \\ 0.35 \\ 0.36 \\ 0.37 \\ 0.38 \\ 0.39 \end{pmatrix} \qquad Power1 := \begin{pmatrix} 0.173 \\ 0.177 \\ 0.18 \\ 0.183 \\ 0.185 \\ 0.187 \\ 0.189 \\ 0.191 \\ 0.192 \\ 0.192 \\ 0.193 \\ 0.193 \\ 0.193 \\ 0.192 \\ 0.192 \\ 0.191 \\ 0.189 \\ 0.188 \\ 0.186 \\ 0.184 \end{pmatrix}$$

Test 1.1- Change of blade area:

i) Increase in blade width, f by 50%. Power Output noted

ii) Increase in blade depth, d by 50% which led to increases in length, L and leaving angle of blade, xL. Power Output noted

Test 1.2- Change in number of blades:

i) Increase in number of blades by 57%. Power Output noted.

Test 1.3- Change in lever arm:

i) Increase in lever arm, y by 50% leading to an increase in length, L and leaving angle of blade, xL. Power Output noted.

Test 1.4- Change in current speed:

i) Current speed increased by 50%. Power Output noted

Test 1.5- Change in DraftT

i) Increase in DraftT, causing a decrease in leaving angle of blade, xL. Power Output noted.

ii) Decrease in DraftT, causing an increase in leaving angle of blade, xL. Power Output noted.

Fig. 39 Results from Tests 1.1-1.5. Blue variables have been changed; yellow shows other variables reacting to that change

Test	N	Current (ms^-1)	Draft T (m)	f (m)	d (m)	L (m)	y (m)	p	xL	Power Output (kW)
Initial Values	7	2	0.8	0.3	0.5	1.3	0.8	0.3	52.02	0.12
T1.1 i)	7	2	0.8	0.45 +50%	0.5	1.3	0.8	0.3	52.02	0.187 +156%
ii)	7	2	0.8	0.3	0.75 +50%	1.55 +117%	0.8	0.3	58.9 +113%	0.19 +158%
T1.2	11 +57%	2	0.8	0.3	0.5	1.3	0.8	0.3	52.02	0.196 +193%
T1.3	7	2	0.8	0.3	0.5	1.7 +31%	1.2 +50%	0.3	61.9 +119%	0.158 +132%
T1.4	7	3 +50%	0.8	0.3	0.5	1.3	0.8	0.3	52.02	0.421 +350%
T1.5 i)	7	2	0.12 +50%	0.3	0.5	1.3	0.8	0.3	22.62 -56.5%	0.015 -87.5%
ii)	7	2	0.4 -50%	0.3	0.5	1.3	0.8	0.3	72.08 +38.6%	0.152 +26.7%

From the table above can be seen the increases in the variables and the corresponding increases in Power Output. The most influential variable for a stream wheel is the Current velocity shown by Test 1.4 as with a 50% increase it produces a 350% increase in Power Output. This is consistent with the relative velocity, Vr value being to the power of 2 in the Force equation.

The second most influential variable, which can be controlled by the designer is the number of blades. However the MathCAD model does not take into account that too many blades will lead to the water not being able to flow freely around the blade and so less force being generated on each blade. It was felt between 12 and 16 blades was optimum.

The tests show that an increase in the lever arm, y is more influential than either an increase in the depth or width of the blade (which produce very similar Power Output increases) and any change in draft is surprisingly uninfluential.

4.4 Estimation of Typical Domestic Energy Requirements

There are no published figures for a typical 3-bedroom house's energy requirements so in order to estimate them a list of household appliances was drawn up, their power ratings and duration of usage estimated. Obviously not all of these appliances will be running all day every day, so the duration of usage (in a month) is estimated and multiplied by the rating to get the power usage in kWhr. This is then divided by the number of hours in a month to get an overall power requirement in kW.

Standard appliances have been used throughout as have some of the findings of the Energy Consumption in the UK Report (41) which shows statistically appliances that are widely owned. From the graph below the household appliances considered "typical" included- VCR, washing machine, microwave, fridge freezer, refrigerator.

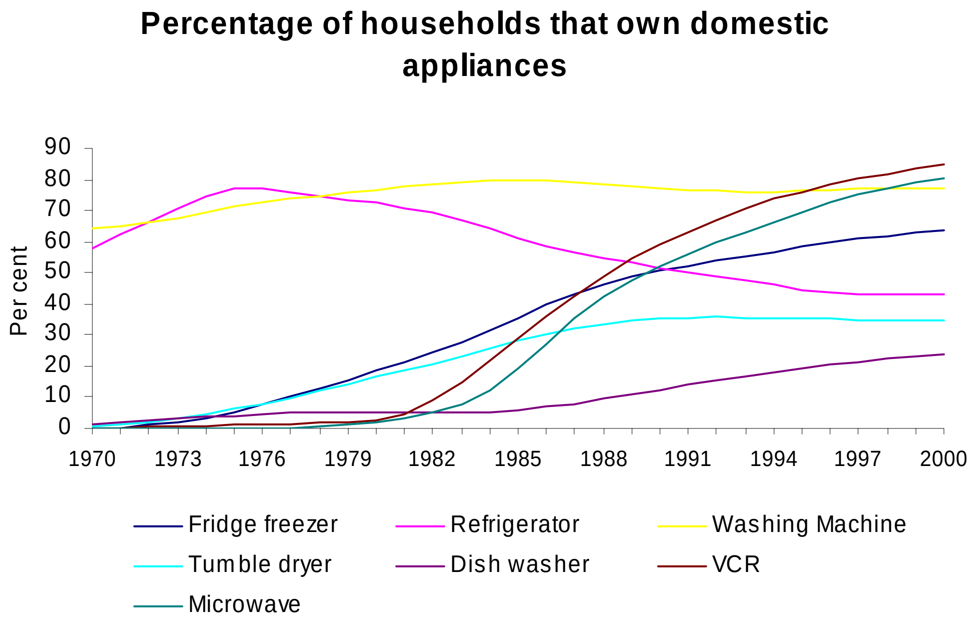


Fig. 40 Graph showing the percentage of household appliances owned in the UK, 1970-2000 (41)

From page 25 of the report it was noted that 59% of UK households have two or more colour televisions, 45% of UK houses own a home computer and that energy consumption for lighting had risen over the last decade due to multiple lights and table lamps (41). According to more recent DTI (42) data electric heating is the second most common form of heating, far behind gas so it is fairly realistic for the house in this dissertation to be heated using electricity.

Fig. 41 Table calculating the typical energy requirements of a three bedroom, four person family based on figures from (43).

Appliance	Quantity	Power rating in W	Average hours per month	Energy used (kWhr) per month
Blender	1	350	3	1
Coffee Maker	1	900	12	11
Electric Kettle	1	1500	15	10
Refrigerator freezer	1	500	300	150
Microwave Oven (0.5 ft)	1	900	10	9
Range and oven	1	3500	25	90
Toaster	1	1200	4	5
Washing machine (33 loads/month)	1	500	26	13
Electric water heaters Family of four	1	3800	140	532
Iron	1	1000	12	12
Water pump (1.2 hp)	1	1000	44	44
Electric Heating	1	1000	250	250
Hair Dryer	1	1000	5	5
60 W bulbs	15	900	120	108
Portable electric heater	1	1000	350	350
Telephone	1	6	720	4

Vacuum	1	800	10	8
Appliance	Quantity	Power Rating in Watts	Average hours per month	Energy used (kWhr)per month
Home computer	1	250	240	60
Radio	1	5	120	1
Stereo	1	120	120	14
Colour Television	1	100	125	13
Video Cassette Recorder	1	40	100	4
Total Energy Usage				1694 kWhr
Number of hours per month				720 hrs
Total Power Rating Required				**2.35 kW**

The average power rating is 2.35 kW but obviously there will be periods where there is a higher consumption e.g. morning breakfast and evenings, winter time.

For their electrical system the Pedley Wheel Trust estimates efficiencies in the region of generator- 81% and gearbox- 97% (49). Using these figures the overall power output will be 7.857 kW. By choosing 10 kW as the design output for the wheel enough energy can be insured and excess power may be stored in a battery or sold back to the national grid

4.3 Final Waterwheel Specification

The final waterwheel output is 10.5kW of power with specifications of:

L = 3.5m

y = 1m

d = 2.5m

f = 3m

DraftT = 0.42m

N = 12

4.41 Rotational speed check

One of the *"main disadvantages of water wheels for electricity generation is the slow shaft speed"* (24) as AC generation requires a high rotational speed. Ideally at the most a 1:100 gear ratio will be used needing a rotational speed of at least 6 rpm from the wheel. Using $Rotationalspeedofthewheel = (9 \times Current) \div Diameter$ (6) the rotational speed of the final waterwheel was calculated as being 2.75 rpm. Chapters 5 and 6 detail a barge and catamaran design to support the wheel and also increase this rotational speed.

5 Barge Design using Excel

Following the calculation of a low rotational speed of waterwheel one possible solution was suggested- a barge with two waterwheels either side. A breakdown of the tasks involved in the barge design is shown below:

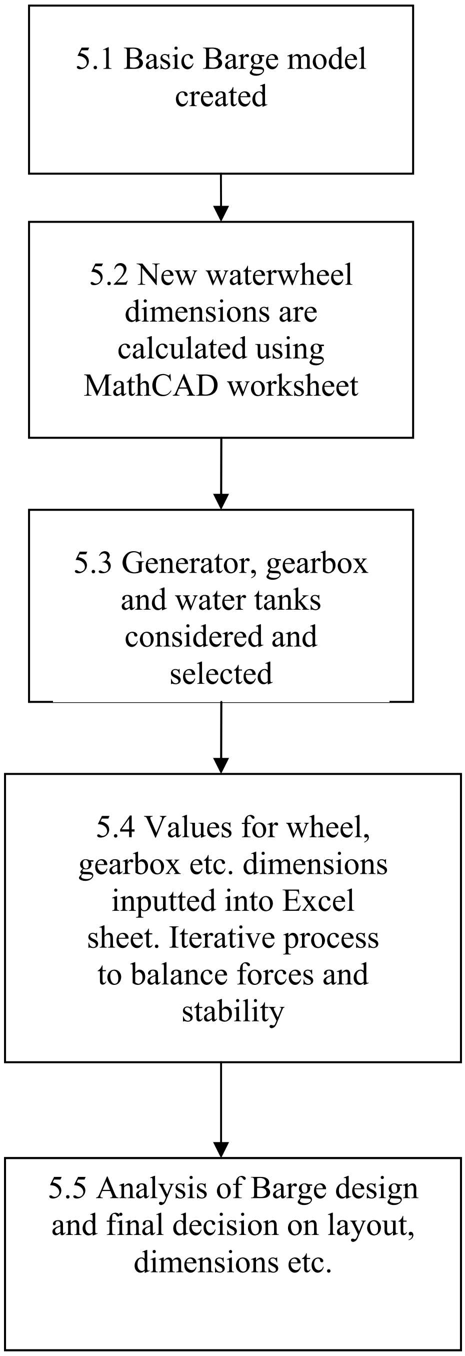


Fig. 42 Design process for barge

5.1 Basic Excel Model

Using the basic naval architecture given in the Technical Background section the basic barge design spreadsheet over pages 62-64 was generated (see enclosed CD for spreadsheet).The triangular barge ends pictured below prevent waves and turbulence at the barge ends. For this small scale design it was felt that the metacentric height, GM should be at least 30cm. Although drawn here the trash rack is not included in the spreadsheet.

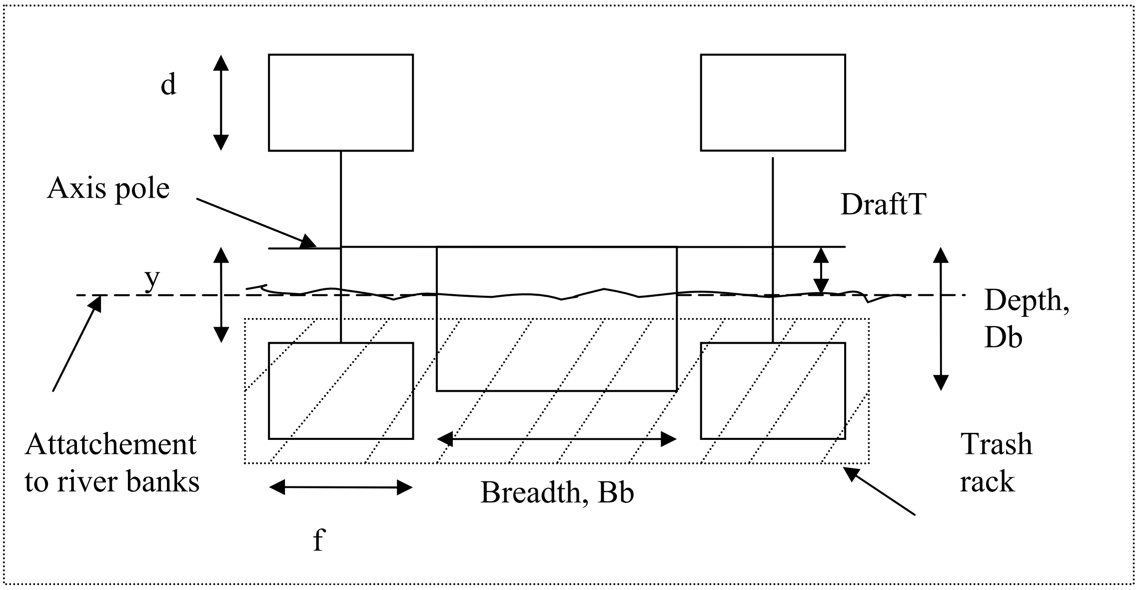


Fig. 43 Cross section showing barge with two waterwheels attached at either side

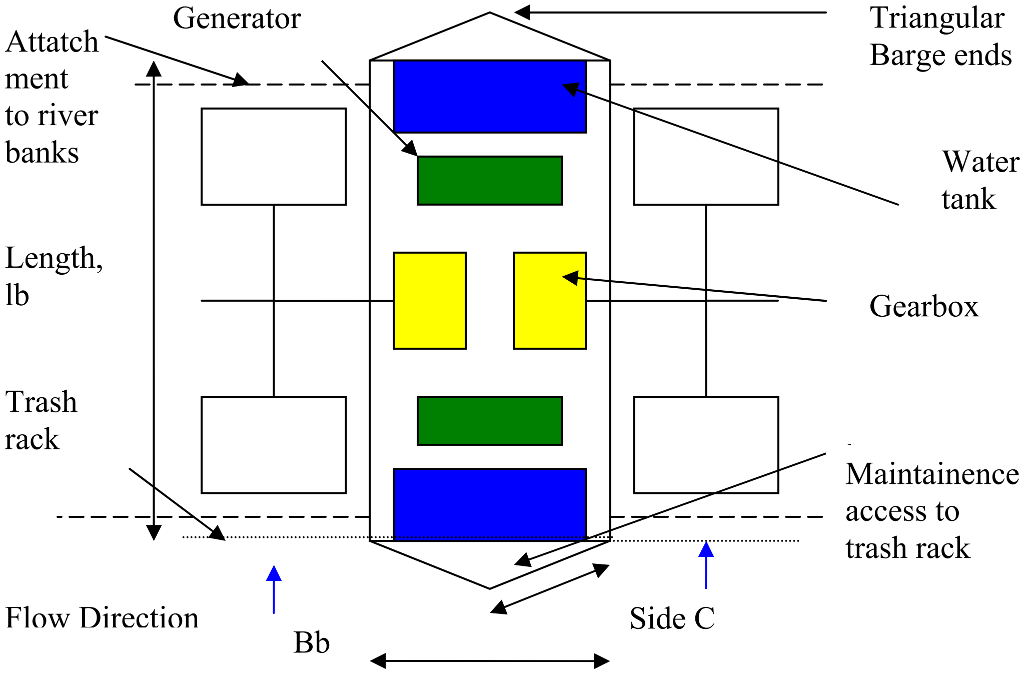


Fig. 44 Plan of barge showing approximate layout of generator, gearbox and watertanks

5.2 Waterwheel Re-design

The decision to have two 5.32 kW waterwheels producing the same 10.64 kW total power output as one wheel would have, leads to a lesser diameter for each wheel, increasing the rotational speed of the wheels, lessening the gear ratio.

Working in reverse from the optimisation of 4.32 now it becomes a matter of which dimensions can be lessened, whilst still keeping power output as high as possible. In changing the diameter, L the depth of the blades, d and the lever arm, y will lessen. Tests 1.1 and 1.3 showed that the depth of the blade is more influential in power output terms, so is kept as high as possible.

The final dimensions for each wheel are:

L = 1.5m

y = 0.4m

d = 1.1m

f = 3.5m

DraftT = 0.169m

N = 12

The rotational speed for each wheel is now 6rpm allowing a gear ratio of 1:100.

It is assumed that both wheels will experience the same velocity of current and so rotate at the same speed.

5.3 Generator, Gearbox and Watertank selection

5.31 Gearbox selection

A disadvantage often sited about waterwheels is their slow rotational speed. In the industrial revolution when they were often used to mill or lift water this posed no problem but to generate AC electricity a large gearing ratio is needed.

Waterwheel installers have sought to overcome this in a number of ways- using tractor gearboxes (6), modified car alternators (47), or even washing machine motors (48). Some groups advocate using integral geared motors, describing them as being available world wide and relatively cheap (49) whereas others claim using them as a generator is expensive (6).

The system shown below uses a specially designed Hydro-alternator that is ideal for low rotational speeds and is self-adjusting, and so useful for variable flows. The Canadian manufacturer supplies remote fishing cabins that generate their own electricity for domestic use and fishing boat battery recharging (50). Notice by building a sloped off-shoot stream to the main river the velocity can be increased.

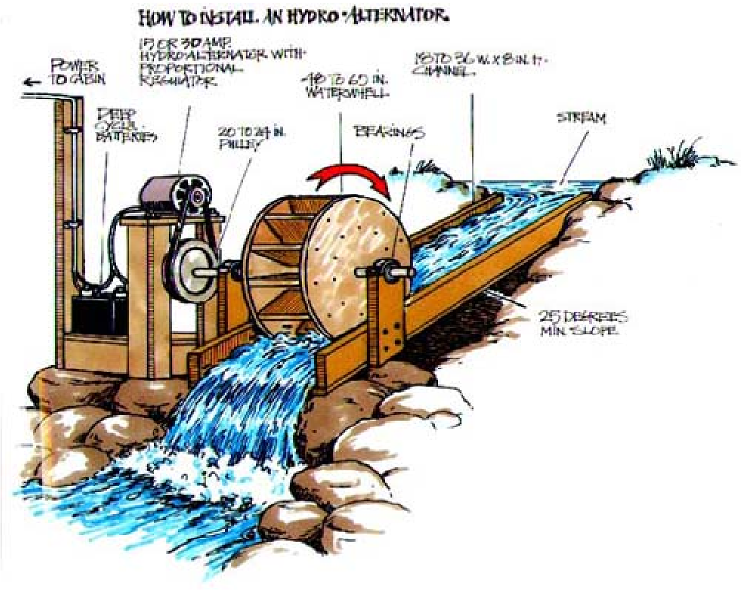


Fig. 45 Diagram showing Hydro-alternator wired to a waterwheel, Canada (50)

For the purposes of this dissertation a gearbox has been chosen to give a fairly accurate idea of the typical weight and dimensions involved, to more accurately calculate the downwards force on the barge and the space required. For a real-life design a trained mechanical engineer would be consulted.

Following advice from Dr. Davies of Heriot Watt University a type of reduction gearbox was chosen from suppliers, Boston Gear's (www.bostongear.com) non-flanged reducer 600 series range. Gearbox 623A-100 has been chosen with a ratio of 1:100 and weight of 36 lbs (≈16.3 kg) (51) with the overall dimensions in Fig…

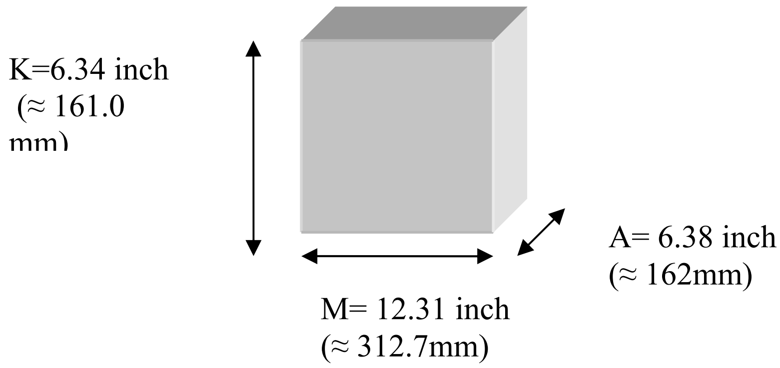
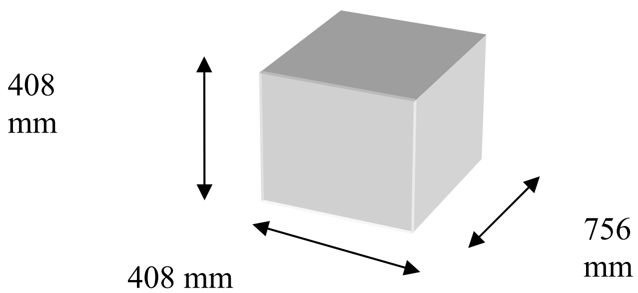


Fig. 46 Overall dimensions of gearbox from (51)

5.32 Generator Selection

As most houses require AC electricity it was decided to pick a Permanent Magnet Alternator that produces AC current. Other generation and storage systems could be selected but this is just a preliminary selection to give some indication of the weight and dimensions. After much searching a 2.5 kW Alternator was selected and its weight and overall dimensions multiplied by 1.5 to roughly equal a 5 kW Alternator, giving weight ≈ 60 kg (52)

Fig. 47 Overall dimensions of generator (52)



5.33 Watertank Design

In barge or catamaran design it is necessary to exactly balance the downwards force of the barge and its contents, with the upwards thrust of the volume of water displaced. To allow for future design changes that lead to an increase or decrease in weight (such as new, heavier gearbox or more blades on waterwheels, lighter generator etc.) two water tanks are included in this design. If further load is added later the equivalent weight of water can be removed from the tanks, keeping the vessel at the same draft depth.

5.34 Iterative barge design

There are two main components to designing a barge; on one hand the force downwards has to equal the force upwards and the stability criteria have to be met. The downwards force of the barge and its contents is balanced by the volume of water the barge is displacing. In that case it follows that a greater underwater volume will lead to a greater load supported.

However the metacentric height, GM is calculated using $GM = VCB + BM - VCG$ where VCB is the Vertical Centre of Buoyancy, BM is the distance between the centre of buoyancy and the metacentre and VCG is the Vertical Centre of Gravity. This can be rewritten:

$$GM = (Draft \div 2) + (2ndMomentofArea, Ixx \div Underwatervolume)$$
$$- ((\sum Mass \times DistToCentreofGravity) \div \sum Mass)$$

The draft value is given from the MathCAD sheet and ideally is not altered. So to maximise the GM value BM has to be at a maximum and VCG at a minimum.

$$BM = (bd^3/12) \div (l \times b \times (d - Draft))$$

It is difficult to maximise BM as increasing the top line containing breadth and depth will lead to a proportional increase in the bottom line, as it also contains breadth and depth.

Working with the Excel spreadsheet it seems as though maximising the length and minimising the depth and breadth slowly increases the metacentric height, however despite several design changes, such as lighter material for the wheel (wood instead of steel), a low 4 kN of watertank weight and the inclusion of triangular barge ends, contributing to the underwater volume the metacentric height still failed to rise above zero. Over the following three pages a copy of the Excel file can be seen with the two forces balancing but the metacentric height still being too low.

It was felt that this indicated the unsuitability of a barge to support two waterwheels, especially two wheels that had to be set low into the flow to generate high enough outputs. It was therefore decided to modify the Excel file to investigate a catamaran layout (Chapter 6). This would not need to be as rigorously analysed for stability as catamarans are inherently more stable, having two hulls rather than one.

Fig. 48 Copy of Barge design spreadsheet

Barge Design Spreadsheet

Page 1 of

Red= input values

Downwards Force

Waterwheel

Number of blades, N	12		
Draft required,T (m)	0.169		
Length of blade, d (m)	1.1		
Depth of blade, b (m)	0.1		
Width of blade, f (m)	3.5		
Total volume of 1 blade (m^3)	0.385		
Total volume of all blades (m^3)	4.62	Density of blade(kgm^-3	500
Radius of paddle, r (m)	0.05		
Length of 1 paddle,y (m)	0.4		
Total volume of all paddles (m^3)	0.0377	Density of paddle	500
Diameter of axis pole (m)	0.2	Length of axis pole (m)	4.5
Mass of axis pole (kg)	70.6858	Density of axis pole	500
Total mass of blades (kg)	2310		
Total mass of paddles (kg)	18.8496		
Total downwards acting Force from both wheels (kN)	47.0789		
Distance from base of barge to CoG for waterwheel (m)	1		

Gearbox

Mass of gearbox (see report for details) (kg)	16.3	Mass of two gearboxes (kg)	32.6
Total downwards acting Force (kN)	0.31981		
Depth of gearbox (m)	0.161		
Distance from base of barge to Centre of Gravity(m)	0.0805		

Generator

Mass of generator (see report for details) (kg)	60	Mass of two generators (kg)	120
Total downwards acting Force(kN)	2.3544		
Depth of generator (m)	0.408		
Distance from base of barge to Centre of Gravity(m)	0.204		

Barge Design Spreadsheet Page 2 of

Main body of Barge

Length, lb (m)	8		
Breadth, Bb (m)	2		
Depth, Db (m)	1		
Thickness (m)	0.2		
Volume of base (m^3)	3.2	Density (kgm^-3)	500
Mass of base (kg)	1600	Downwards force (kN)	15.7
Distance from base of barge to Centre of Gravity (m)	0		
Volume of sides A (m^3)	0.8		
Volume of Sides B (m^3)	3.2	Density (kgm^-3)	500
Mass of sides (kg)	2000	Downwards force (kN)	19.6
Distance from base of barge to Centre of Gravity (m)	0.5		
Triangular Barge Ends			
Length of side C (m)	1.6		
Volume of sides C (m^3)	1.28		
Mass of Sides C (kg)	2048	Downwards force (kN)	20.1
Distance from base of barge to centre of gravity (m)	0.5		
Distance q (m)	1.249		
Volume of base of triangular barge ends (m^3)	0.1249		
Mass of base of triangular barge ends (kg)	62.45	Downwards force (kN)	1.23

Barge Design Spreadsheet Page 3 of

Watertanks			
Length of Side F (m)	0.75		
Length of Side E (m)	0.4		
Depth of watertank, Dw (m)	1	Density of water (kgm^-3)	1000
Level of water in watertank, dw (m)	0.3		
Mass (kg) of watertanks with water at level dw (kg)	180		
Distance from base of barge to Centre of Gravity(m)	0.15		
Downwards force exerted by both watertanks(kN)	1.7658		
Total Downwards force (kN)	108.151		

Upwards Force

Volume water displaced by barge (m^3)	11.049
Total Upwards Force (kN)	108.39

Barge Design Spreadsheet Page 4 of

VCB (m)	0.0845
VCG	
Base and triangular base Moment (kgm)	0
Sides Moment (kgm)	1000
Watertank Moment (kgm)	27
Gearbox Moment (kgm)	2.6243
Generator Moment (kgm)	24.48
Waterwheel and axis pole Moment (kgm)	2399.535391
Side C Moment (kgm)	1024
Side D Moment (kgm)	0
Sum of Moments (kgm)	4477.639691
VCG (m)	0.530363567
I waterplane about Centre line (m^4)	0.166666667
BM (m)	0.015084354
Metacentric Height, GM (m) (should be less than 0.30m)	-0.430779213

6. Catamaran Design

Following the unsuccessful barge design a catamaran design was suggested. The original problem of a slow turning wheel was now solved by using the sides of the catamaran to restrict flow towards the wheel, increasing the speed. Test 1.4 earlier showed that an increase in Current of 50% could lead to an increase in power output of 350% allowing the wheel diameter to be lessened, increasing the rotational speed of the wheel.

Taking the modified MathCAD output the Excel model was altered to design a catamaran (see enclosed CD for catamaran file) and an iterative process was begun to balance forces in both directions.

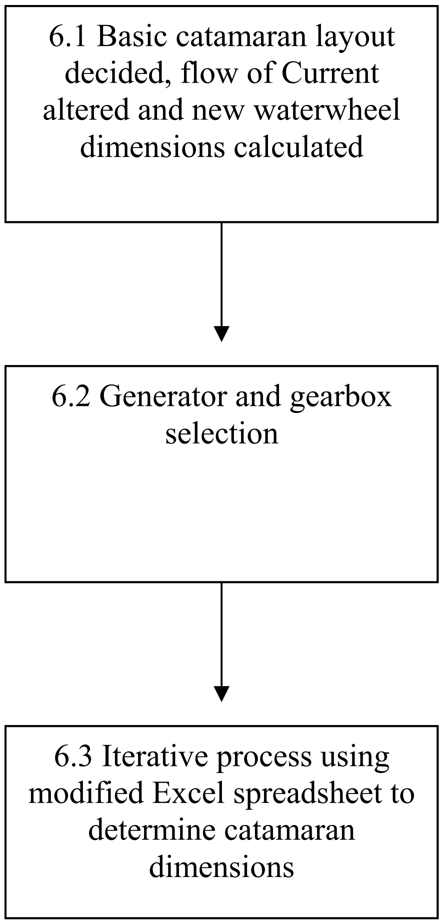


Fig. 49 Diagram showing design process followed for catamaran

6.1 Catamaran layout and Re-design of Waterwheel

6.11 Catamaran Layout

The catamaran design involves a wheel in between two hulls containing the gearbox, generator and water tanks as symmetrically as possible (See Figs 50 and 51 below). Although shown here neither the trash rack nor stability bars are considered in the Excel file, being fairly lightweight.

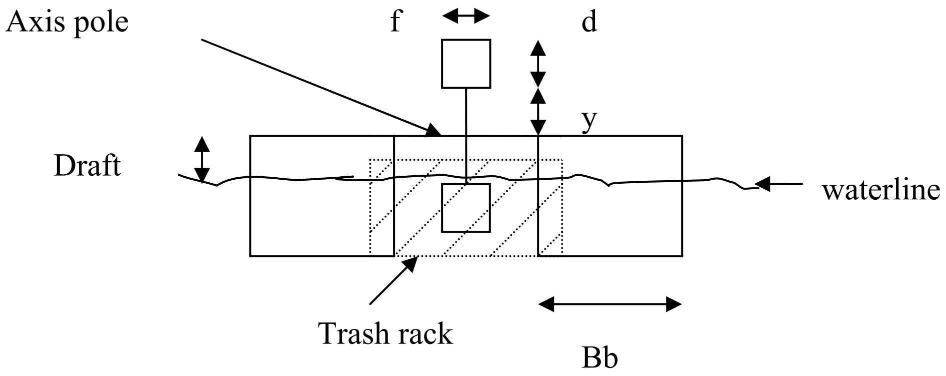


Fig. 50 Cross section of catamaran in direction of flow

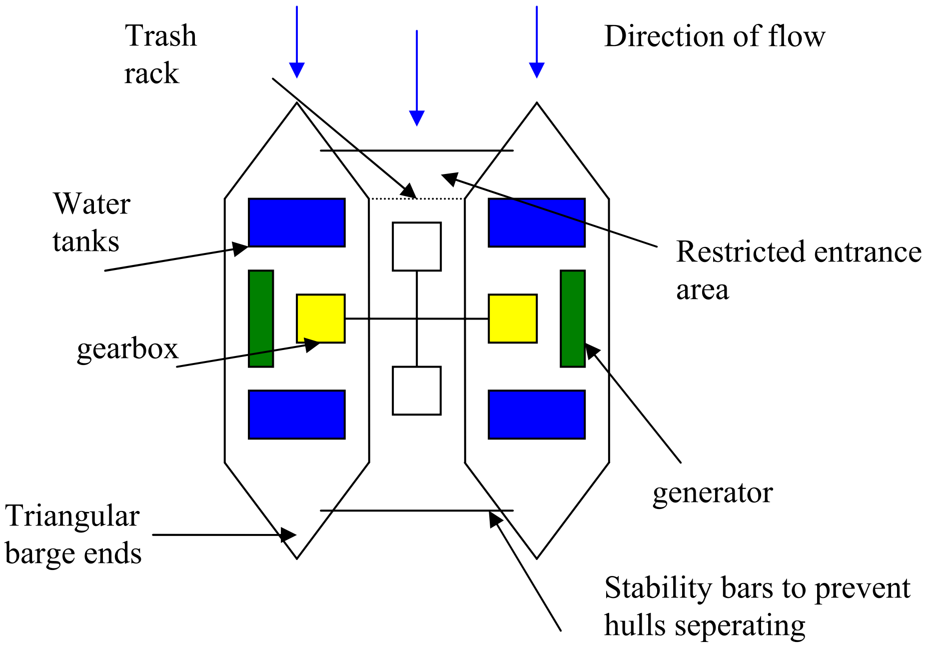


Fig. 51 Plan of catamaran

6.12 New waterwheel design

To evaluate the increase in Current caused by a restricted opening area to the wheel firstly the flow in the river has be inputted. As actual data is not available it is estimated that the channel is 4m wide and the distance from the waterline to the river bed is 3m. If site investigations showed otherwise these figures can be changed manually in the MathCAD program (see attached CD for program).

From this a flow rate of 24 m^3s^{-3} was calculated. The restricted area of the flow before the wheel is equal to the multiplication of the catamaran depth (kept fairly deep to ensure high velocities and water flowing into wheel rather than underneath catamaran) and the distance between the two hulls, set at 0.5m more than the width of the blades. In this example using this arrangement increases the Current from 2 ms^{-1} to 3.5 ms^{-1}

This increased speed allows for smaller wheel dimensions leading to an increase in the rotational speed of the wheel. The final wheel dimensions are:

L = 1.25m

y = 0.5m

d = 0.75m

f = 1.75m

DraftT = 0.211m

N = 12

This gives an output of 10.76kW turning at a speed of 12.6 rpm.

6.2 Generator and Gearbox Selection

Although in a real life application a different gearbox and generator would be selected the same pair are being kept here. The selection in this dissertation would probably not be actually used in reality; here it is an indicator of size and weight.

Instead of a 1:100 gear ratio 1:50 could be used here, and again two 5kW generators would be incorporated.

6.3 Catamaran Design

The Excel sheet for the barge was altered to include the two hulls of the catamaran (see enclosed CD for program). The new dimensions of the wheel were then added and the barge dimensions and water level in the tank were altered to balance the downwards and upwards forces. As catamarans are more stable than barges it was considered unnecessary to calculate the metacentric height of the system. The following 72-73 pages show the Catamaran Excel sheet.

The significant drop in the waterwheel weight means that the wheel can now be manufactured out of steel (coated against corrosion) with a density of 7700 kgm^{-3} rather than the wood used for the barge design with a density of 500 kgm^{-3}. The catamaran is still made from wood, however.

The increased displacement volume of two hulls allows for 34 kN of water to be stored on the catamaran allowing large weight changes to be made in the future. This could help with future maintenance e.g several people standing on the catamaran to clear the trash rack or bringing on heavy tools to repair or replace parts. The dimensions of the catamaran and water tanks have been kept uniform to ease construction, although a scale would be needed on the water tank for 1.78m of water to be measured out easily.

Fig. 52 Copy of Catamaran Spreadsheet

Catamaran Design Spreadsheet

Page 1 of 3

Red= input values

Downwards Force
Waterwheel

Number of blades, N	12		
Draft required,T (m)	0.211		
Length of blade, d (m)	0.75		
Depth of blade, b (m)	0.1		
Width of blade, f (m)	1.75		
Total volume of 1 blade (m^3)	0.13125		
Total volume of all blades (m^3)	1.575	Density of blade(kgm^-3	7700
Radius of paddle, r (m)	0.05		
Length of 1 paddle,y (m)	0.5		
Total volume of all paddles (m^3)	0.04712389	Density of paddle	7700
Diameter of axis pole (m)	0.2	Length of axis pole (m)	2.75
Mass of axis pole (kg)	665.2322444	Density of axis pole	7700
Total mass of blades (kg)	12127.5		
Total mass of paddles (kg)	362.8539515		
Total downwards acting Force from wheel (kN)	129.0563006		
Gearbox			
Mass of gearbox (see report for details) (kg)	16.3	Mass of two gearboxes	32.6
Total downwards acting Force (kN)	0.319806		
Depth of gearbox (m)	0.161		
Generator			
Mass of generator (see report for details) (kg)	60	Mass of two generators	120
Total downwards acting Force(kN)	2.3544		
Depth of generator (m)	0.408		

Catamaran Design Spreadsheet Page 2 of 3

Main body of Catamaran

Length, lb (m)	2.5		
Breadth, Bb (m)	1.5		
Depth, Db (m)	3		
Thickness (m)	0.2		
Volume of base (m^3)	0.75	Density (kgm^-3)	500
Mass of base (kg)	375	Downwards force (kN)	3.68
Volume of sides A (m^3)	1.8		
Volume of Sides B (m^3)	3	Density (kgm^-3)	500
Mass of sides (kg)	2400	Downwards force (kN)	23.5

Triangular Barge Ends

Length of side C (m)	1.6		
Volume of sides C (m^3)	3.84		
Mass of Sides C (kg)	1440	Downwards force (kN)	14.1
Distance q (m)	1.413329403		
Volume of base of triangular barge ends (m^3)	0.105999705		
Mass of base of triangular barge ends (kg)	52.99985259	Downwards force (kN)	1.04

Catamaran Design Spreadsheet Page 3 of 3

Watertanks

Length of Side F (m)	1		
Length of Side E (m)	1		
Depth of watertank, Dw (m)	3	Density of water (kgm^-3)	1000
Level of water in watertank, dw (m)	1.78		
Mass (kg) of watertanks with water at level dw (kg)	3560		
Downwards force exerted by all watertanks(kN)	34.9236		
Total Downwards force (kN)	286.3557208		

Upwards Force

Volume water displaced by catamaran (m^3)	29.16945375
Total Upwards Force (kN)	286.1523413

7 Conclusion

7.10 The waterwheel has played an important part in energy production over the last 2000 years, reaching its peak in usage in the early 19th century, but in the end losing out to steam and motor power. The waterwheel is today experiencing renewed interest as engineers, scientists and now energy suppliers (due to the Renewables Obligation) look for new renewable energies to combat climate change.

7.11 However progress is hampered by a lack of government support, high gearing ratios, little recent research and a perception amongst engineers, scientists and the public that waterwheels are old fashioned and irrelevant.

7.12 Waterwheel progress is most apparent on the internet with many sites written by home enthusiasts and small charities. This dissertation includes three case studies as a homage to this group and an example of how interested and keen the public are on interactive renewable energy.

7.13 Scotland has been chosen for this dissertation because of it plentiful supply of small rivers and streams, the high yearly rainfall and a rural population who could benefit from decentralised power. After briefly evaluating different types of wheel an undershot wheel was chosen, as it utilised the lowest heads. From this group the stream wheel was picked for this project as the site had zero head, and a fairly fast flow. Additionally stream wheels are simple to manufacture and considered “fish friendly”. Experiments currently running at Berlin Technical University will be able to shed new light onto their flow and power output capabilities.

7.14 A mathematical model of the flow of a stream wheel and its corresponding power output was created. The program was then used to estimate the dimensions for a typical domestic house’s energy requirements. The slow rotational speed of this design and the consequent large gearing ratio led to an investigation in fixing the wheel to a barge, and later a catamaran.

7.15 A basic barge layout complete with water tanks, generators and gearboxes was modelled in Excel and from the output it was concluded that the stability of a barge with two waterwheels either side was not satisfactory. So the spreadsheet was altered to describe a catamaran design with two hulls either side of a waterwheel. The benefit of this was that the entrance area of the water between the hulls was restricted, leading to a higher Current and so a smaller wheel for the same power output. Further advantages to this system included a high rotational speed, low gearing ratios, and a large amount of on board stored water, which allows later design alterations.

7.16 In conclusion it can be said that a waterwheel is a viable, aesthetically pleasing option for domestic energy generation that would benefit from further investigation.

8 Suggestions for future work

8.10 Being so previously unresearched waterwheels now present a large variety of potential projects. Fundamentally there has been little work into the characteristics of the flow of waterwheels. Understanding exactly how energy is transferred, how the blade interacts with the water and how these characteristics can be optimised is crucial to the advancement of all wheels especially the stream wheel, and the stream wheel in a rectangular channel

8.11 From this predominately experimental and case study based work designs can be generated for the blade shape and size (with the drag coefficient being researched experimentally), optimum number of blades, and external works. An evaluation into the relative merits of fixed waterwheels perhaps sited in their own diversion channel or moored waterwheels on barges and catamarans is needed. Ecological studies are needed into the effects of waterwheels as is the simulation and minimisation of the impact noise so as to prevent wheels becoming unpopular with their neighbours.

8.12 In the wider context an evaluation is needed of the low head (‹ 25 kW) sites in the UK with more accurate estimates of the potential of microhydropower. A lack of knowledge of previous waterwheel installations also hampers progress so an in-depth investigation into case studies worldwide would enable a type of guide to be drawn up detailing successful practice. When combined with experimental and scientific theory this guide could slowly replace the wealth of intuitive knowledge lost at the end of the industrial revolution. Further afield the application of waterwheels to developing world sites is virtually unknown.

8.13 Any form of renewable energy will rely on generators, gearboxes, alternators or batteries to gain and store power and waterwheels are no exception. Further research is needed into gearing methods that are both efficient and easy to construct, and relatively inexpensive.

References

1. Muller,G, Kauppert, K and Mach, R (2002) Back to the future, International Water Power and Dam Construction, August 2002, pgs 30-33

2. Goring, O (2004) Potential power?, International Water Power and Dam Construction, 7th September 2004
http://www.waterpowermagazine.com/story.asp?storyCode=2024514

3. The Pedley Wheel Charitable Trust (2001) The Pedley Wheel: a waterwheel-driven electricity generator, International Journal on Hydropower and Dams, 2001, Issue 2

4. Klunne, W (2000) Microhydropower basics- Introduction to micro hydro,
http://www.microhydropower.net/into.html

5. Muller,G, Himmelsbach,G, Frohle,P, Carmer, C.v, Small River Re-Naturalization and Cultural Heritage,
http://www.muehlen-deutschland.de/Allerlei-Ubersicht/Artikel-Wasser/Allerlei-Verschiedenes/Allerlei-Protestaktion/1_-_Small_River_re-naturalization.html

6. British Hydropower Association, Waterwheels
http://www.british-hydro.org/infopage.asp?infoid=185

7. Bjorling, P R, (1894) Chapters 1-7. In:Water or Hydraulic Motors, London: E.and F.N Spon

8. Denny, M (2004), The efficiency of overshot and undershot waterwheels, European Journal of Physics, March 2004, Issue 2

9. Schafer, W (2002) Paddle-Steamer DS "Gallia" on the Lake Lucerne: High Tech of 1913 or 2002?, In: 3rd International Euro Conference on High-Performance Marine Vehicles, "HIPER 2002", pgs 329-344, 14-17 September 2002, Bergen

10. Muller, G, Kauppert, K (2002) Old watermills- Britain's new source of energy?, Proceedings of ICE, Civil Engineering 150, November 2002, pgs 178-186

11. Scottish Executive (2004) Review of Scottish Climate Change Programme- A Consultation

12. Energy Saving Trust, Renewable Energy Explained.
http://www.saveenergy.co.uk/renewables/explained/

13. Klunne, Wim (2003) Micro and Small Hydropower For Africa, ESI Africa Issue 4
Also found at http://www.microhydropower.net/intro.html

14. DTI Renewables Explained (2005) Hydroelectric Current use in the Uk
http://www.dti.gov.uk/renewables/renew_1.7.2.htm

15. Renewable Power Association (2002-2003) Map of Hydroelectric Plant in the UK.
http://www.r-p-a.org.uk/article_flat.fcm?articleid=14

16. DTI Renewables Explained (2005) Introduction
http://www.dti.gov.uk/renewables/renew_1.1.htm

17. DTI Policy (2005) UK national policies- Scotland
http://www.dti.gov.uk/renewables/renew_2.3.2.htm

18. Intergovernmental Panel on Climate Change (IPCC), Edited by Watson, R.T, Zinyowera, M.C, Moss, R.H (November 1996) Technologies, Policies and Measures for Mitigating Climate Change- IPCC Technical Paper I, pg 49
http://www.ipcc.ch/pub/techrep.htm

19. Scottish Executive (2004) Renewable Energy Hydro
http://www.scottishexecutive.gov.uk/Topics/Business-Industry/infrastructure/19185/17851

20. Scottish Renewables, Map of Renewable Energy Sites
http://www.scottishrenewables.com/renewable_map.asp

21. Penche, Celso, Commission of the European Communities (1997) Layman's guide on how to develop a small hydro site, Directorate-General for Energy by European Small Hydropower Association (ESHA)
http://europa.eu.int/comm/energy/library/hydro/layman2.pdf

22. British Hydropower Association, Water Wheels,
http://www.british-hydro.org/infopage.asp?infoid=185

23. Muller, G, Wolter, C (2004) The breastshot waterwheel: design and model tests, ICE Proceedings- Engineering Sustainability, December 2004, Issue ES4, pgs 203-211

24. Muller, G, Kauppert, K (2004) Performance characteristics of waterwheels, Journal of Hydraulic Research, Vol 42, No 5, pgs 451-460

25. The Sustainable Energy Research Group, Southampton University, Micro Hydropower,
http://www.energy.soton.ac.uk/ then click on Current Research Work followed by Micro Hydropower- More about the Programme

26. Hydrowatt, (2002) Aussenansichten (project literature about installed waterwheels)
http://www.hydrowatt.de/bildseite10.htm

27. Hydrowatt (2002) Einbauzeichnung
http://www.hydrowatt.de/bildseite15.htm

28. Bagdhadi, A.H.A, Mikhail, S. (1985) Design of momentum water wheels used for mini hydropower, International Water Power and Dam Construction, Spt 1985, pgs 47-52

29. Ritz Atro, Gallery Of Pictures- Archimedian Screw Pumps, http://www.ritz-atro.de/english/5_service/5_3_1_schneckenpumpen.htm

30. Gotoh, Masahiro, Kowata, Hisashi, Okuyama, Takehiko, Katayama, Shyusaku, (2001) Characteristics Of A Current Water Wheel Set In A Rectangular Channel, Proceedings of FEDSM'01:2001 ASME Fluids Engineering Division Summer Meeting, May 29-June 1, 2001 New Orleans, USA
Available at http://cse.cryo.affrc.go.jp/griese/neworleanspaper.pdf

31. John Woodcock,Yorkshire Post (2005) Island of Hidden Treasures, Yorkshire Post, 07 February 2005
Available at
http://www.yorkshiretoday.co.uk/viewarticle2.aspx?ArticleID=938310&SectionID=1472&Search=howsham&Searchtype=any&SearchSection=1472&DateFrom=011995&DateTo=022005&Page=1&ReturnPage=Results.aspx

32. Renewable Heritage Trust (2004) The Howsham Mill Project
http://www.rht.greenisp.org/HowshamMill.htm

33. Bromley, Paul (1999) Re-inventing the waterwheel, International Water Power and Dam Construction, 10 February 1999
http://www.waterpowermagazine.com/story.asp?storyCode=1406

34. The Pedley Wheel Trust (2000), The Pedley Wheel- A Water Wheel Driven Electricity Generator, World Renewable Energy Congress VI (WREC2000), July 2000
http://www.pedleywheel.com/pw/articles/worldrenewableenergycongressvi.htm

35. The Pedley Wheel Trust (2001) The Pedley Wheel: a water wheel- driven electricity generator, International Journal on HydroPower and Dams, Issue 2, 2001
http://www.pedleywheel.com/pw/articles/intljournalonhydropowerdams.htm

36. The Pedley Wheel Trust, Home Pedley Wheel Programme to date,
http://www.pedleywheel.com/index.htm

37. Julia Nolan (2005) Interview at Heatherslaw Mill, Ford and Etal Estate, Northumberland

38. David Mann (2005) Personal Communication

39. Gerald Muller (2005) Personal Communication

40. Lockett, W.G, (2001) Jacob Leupold's anaylsis of the overshot water-wheel, Proceedings of the ICE- Water and Maritime Engineering, Dec 2001, Issue 4, pgs 211-218
Available at http://www.extenza-eps.com/extenza/loadPDF?objectIDValue=40571

41. Department of Trade and Industry (2002) Energy Consumption in the UK, http://www.dti.gov.uk/energy/inform/energy_consumption/ecuk.pdf

42. Department of Trade and Industry (2004) Table 3.14- Ownership of Central Heating by type in Great Britain 1970 to 2002 http://www.dti.gov.uk/energy/inform/energy_consumption/table3_14.xls

43. Canadian Renewable Energy Network, CanREN (2004) Publications and Software- Appendix C- Typical Household Appliance Loads http://www.canren.gc.ca/prod_serv/index.asp?CaId=196&PgId=1317

44. Rupprecht Gymnasium, Ph09 Turbinen, http://www.physik.uni-muenchen.de/leifiphysik/web_ph09/umwelt_technik/06wasser/turbine.htm

45. Pressentin ek, Wasserkraft- Staudruckmaschine (SDM), http://www.staudruckmaschine.de/

46. Tony Bartholomew,Renewable Heritage Trust (2004), Picture Gallery http://www.rht.greenisp.org/PictureGallery.htm

47. Harris, Mick, Waterwheel-powered house, http://www.ata.org.au/articles/51hydro.htm

48. Lawley, Michael (2002) Getting Smart With A SmartDrive, Eco Living New Zealand, Issue 14, http://www.ecoinn.co.nz/pdf/getting_smart.pdf

49. The Pedley Wheel Trust, Background The Pedley Wheel, http://www.pedleywheel.com/pw/pedleywheel/background.htm

50. EcoVent (1998) How To Install An Hydro-Alternator http://www.electrovent.com/#engpp

51. Boston Gears (2003) 600 Series Helical Gear Speed Reducers, http://bostongear.com/pdf/product_sections/600_series_helical.pdf

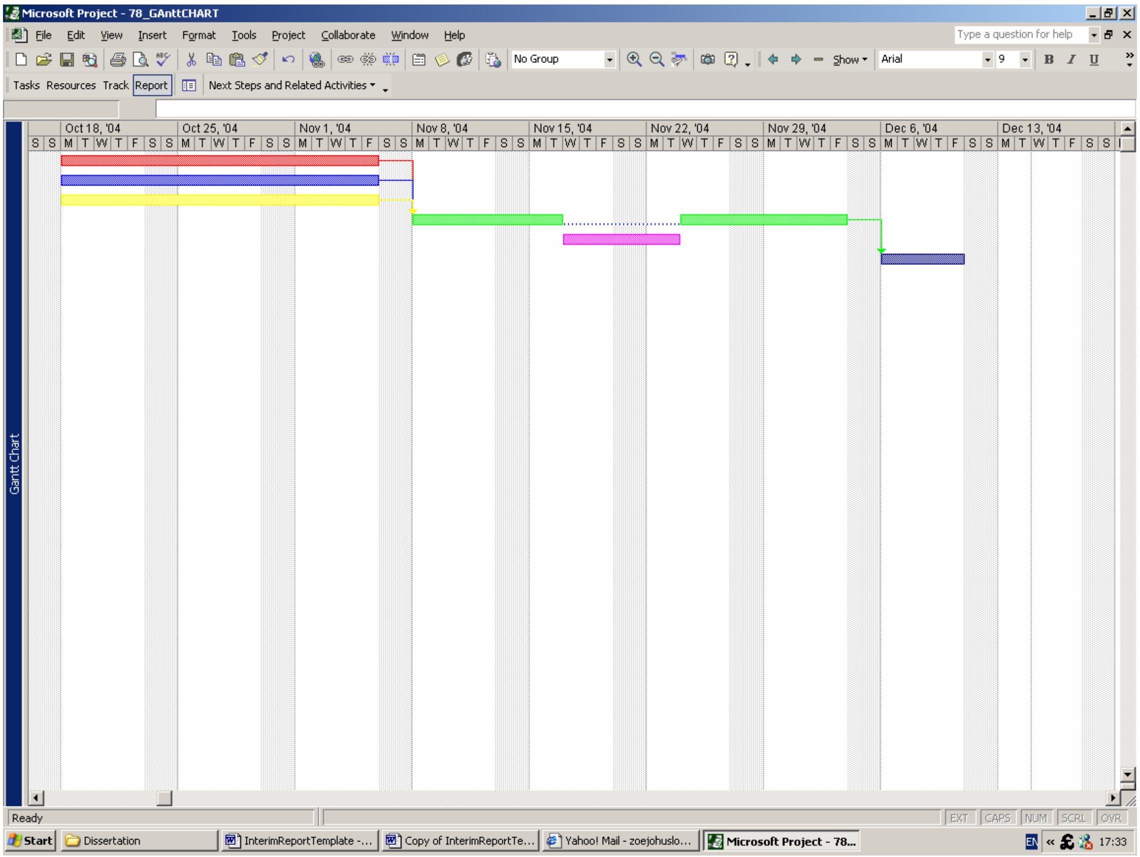
52. WindStreamPower (2002) 445255 2.5kW Permanent Magnet Alternator Tech http://www.windstreampower.com/generators/445255.html

53. Wissenzentrum Wasserrader (2005), Forschung- Hydraulische Modellversuche http://www.projekte.arteng.de/wasserraeder

Appendix A

Project Planning Documents

Project Gantt chart for Term 1



Task A- Background Reading

Task B- Understand Fluid Mechanics involved in waterwheels

Task C- Research and make contact with industrial contacts

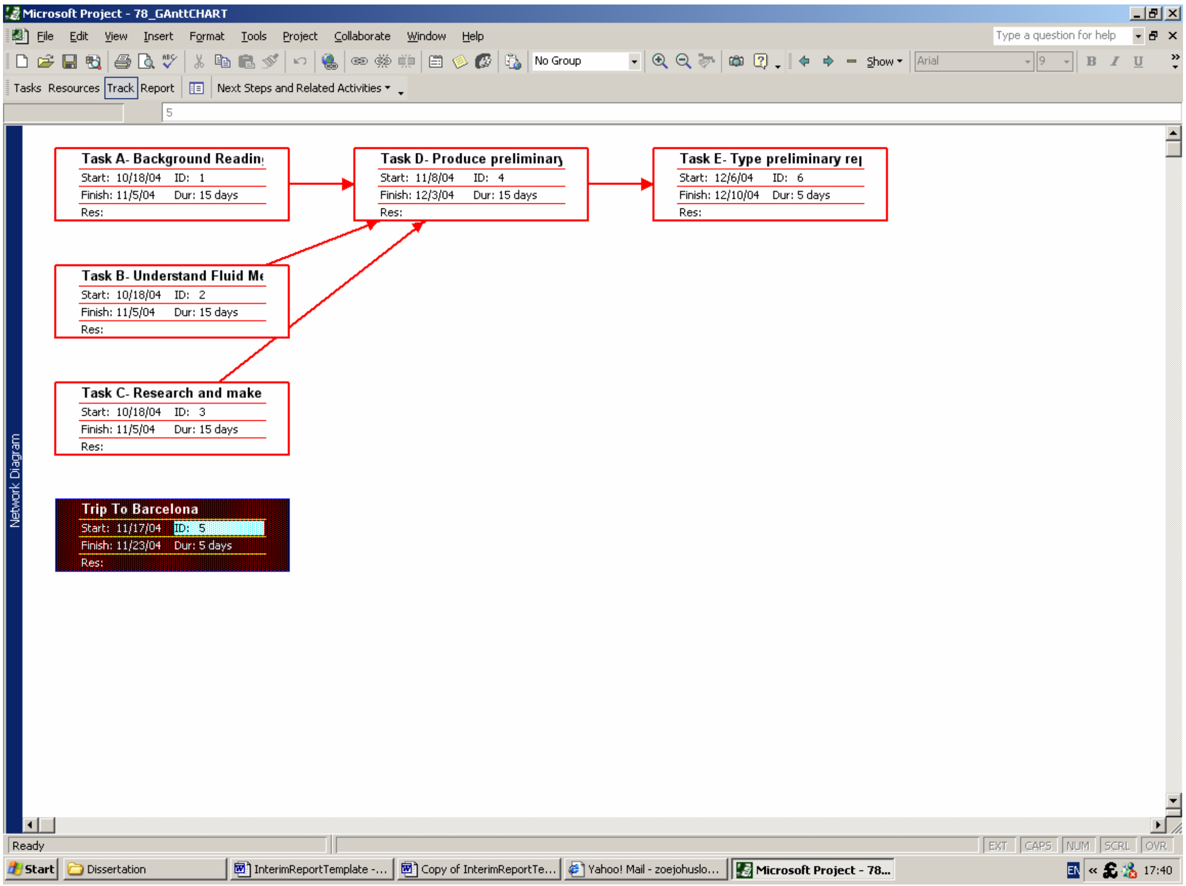
Task D- Produce preliminary waterwheel design

Trip to Barcelona

Task E- Type preliminary report. Send report to J.Wolfram and any industrial contacts

Project Planning Documents

Project PERT Chart for Term 1



HERIOT WATT UNIVERSITY	**School of the Built Environment** **Civil Engineering Programme**	**General Risk Assessment Form**
Project Title:- Dissertation- Domestic electricity generation using waterwheels on moored barge.	**Name:** **Zoë Jones**	**Date:** **June 2004- May 2005**

Work Activities:
1. Analysis of waterwheel carried out using computer and computer software, MathCAD.
2. Research and background reading carried out using computer and Internet and library resources.

Hazards (List significant hazards)	**Who might be harmed?** (List groups of people at significant risk)	**Existing controls** (List existing controls)	**Is risk adequately controlled? (Y/N)**	**Further action to control (List the risks which are not adequately controlled and the action necessary)**
1. Repetitive Strain Injury (RSI) due to typing and using computer mouse for prolonged periods.	**Student**	**Health and Safety (Display Screen Equipment) Regulations 1992. Health advice to take regular breaks**	**Y**	
2. Eyestrain due to prolonged periods sitting at a flickering screen.	**Student**	**Regular breaks and eye exercises**	**Y**	
3. Sore joints and back due to hunched posture at a computer or desk	**Student**	**Adjustable seats**	**N**	**Stretching and strengthening exercise to improve overall posture**
4. Eyestrain due to reading in poor lighting	**Student**	**Rooms allow natural light in and all have electric light**	**Y**	

Appendix B

Proof of "p" by J. Wolfram

$$Power = Force \times Bladevelocity$$
$$Power = (0.5 \times \rho \times Cd \times A \times V_r^2) \times V_b$$

$$V_r = V_c - V_b$$

$$Power = (0.5 \times \rho \times Cd \times A \times (V_c - V_b)^2) \times V_b$$
$$Power = (0.5 \times \rho \times Cd \times A \times (V_c^2 V_b - 2V_b^2 V_c + V_b^3))$$

To find the maximum Power this expression is differentiated and set equal to zero

$$dPower / dV_b = (0.5 \times \rho \times Cd \times A) \times (V_c^2 - 4V_c V_c + 3V_b^2)$$

$$0.5 \times \rho \times Cd \times A \neq 0$$
$$V_c^2 - 4V_b V_c + 3V_b^2 = 0$$
$$V_b = 1 \times V_c$$
$$V_b = 1/3 \times V_c$$

By solving the quadratic two values of Vb are found- one when Vb = Vc (the wheel is freewheeling at this point) and one when Vb=1/3 Vc, which has been used in these calculations.

Domestic Roofwater Harvesting
Research Programme

NEW TECHNOLOGY FOR VERY-LOW-COST DOMESTIC ROOFWATER HARVESTING

DFID KaR Contract R7833

April 2002

Development Technology Unit
School of Engineering, University of Warwick

CONTENTS

1 INTRODUCTION

This report is an output from a 27 month research contract (R7833) for the UK Department for International Development and describes work done during the 4 month prototyping phase of that project. The prototyping phase has developed out of the practice descried in the report to the DFID “R1, Very-Low-Cost Domestic Roofwater Harvesting in the Humid Tropics: Existing Practice” and the constraints identified in “R2, Very-Low-Cost Domestic Roofwater Harvesting in the Humid Tropics: Constraints and Problems”. The work itself was carried out by the DTU along with members of the Lanka Rainwater Harvesting Forum in Sri Lanka, Water Action in Ethiopia and ACORD in Uganda.

The Sri Lankan portion of this work would not have been possible without the facilities and kind assistance of the Nation Builders Association. In particular we would like to thank Mr Adikarum, Mr Amarasinge, Mr Widasecara and Mr Siripala for their help, insight and patience during our time there.

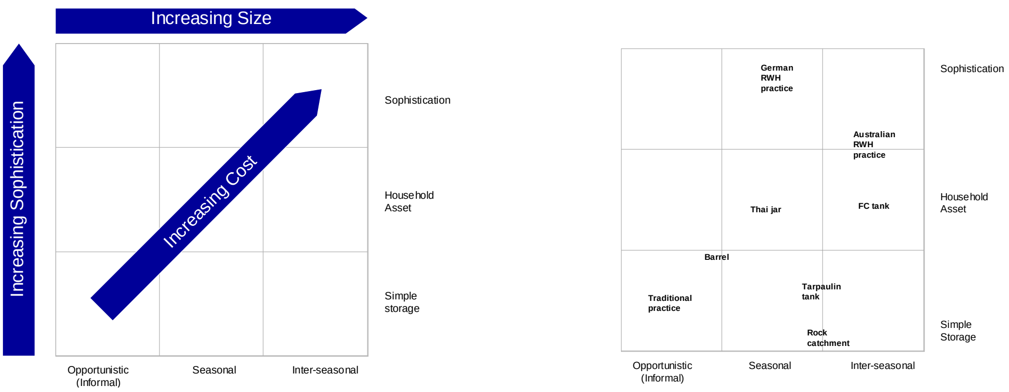
1.1 The need

Surveys carried out in the initial phase of the project have pointed to cost as the number one constraint facing users adopting roofwater harvesting so if it is to become an affordable option for poor people in developing countries, the cost of systems must be reduced.

In R2 a methodology for cost reduction of roofwater harvesting systems was discussed. The method revolved around the building of a rainwater harvesting service framework as shown in Figure 1.1 focusing on the quality of structure as well as the quantity of water delivered. By reducing the structural quality to a level closer to the quality of housing found in poor areas, large savings can result.

Figure 1.1: Service framework for rainwater harvesting systems

a. Framework b. How current systems map onto the framework



In reducing the quality, however, there are a number of critical functional constraints that should be regarded as a minimum specification:

- Gutters should deliver a good fraction of the water falling on the roof – dependent on the local rainfall, roof size storage size and demand pattern
- The tank should not have excessive loss through seepage or evaporation – <5% of the water drawn from it

- The tank should not present an excessive danger to its users, either by their falling in or by the tank failing violently
- The water must be of a quality consummate with its intended use – water that is used for drinking requires a certain care in transport and storage:
  - The catchment area should be smooth and free from accumulated debris
  - The water should be filtered to remove gross impurities or the first flush removed
  - The tank should be covered to prevent entry of light, and sealed against intrusion by vermin
  - The tank should be ventilated to prevent anaerobic decomposition of any washed-in matter

1.2 Methods of cost reduction

The methods used to reduce the cost of rainwater harvesting systems are covered in report R2, the methods described are:

- Material reduction (Improved formwork, Shape optimisation, Function separation)
- Material substitution (Using cheaper materials, use of "free" materials)
- Mass production
- Use of existing containers

1.3 Basis for cost comparison

Costing "rainwater harvesting systems for the poor" is more complex than simply adding up component costs on a bill of materials. Some materials are only available at a cost through purchasing channels, while others are available at no cost if one is willing to gather them. This report will group materials and labour into the following categories:

a. *Economic materials*; materials relying on industrial processes – in their purchase cash will be removed from the area

b. *Local economic materials*; materials which must be bought but where production is from local sources – the cash will remain in the community

c. *Skilled labour*; labour that must be paid for

d. *Local materials*; materials that can be gathered from the land – they may or may not have an economic value but tend to be freely available

e. *Householder labour*; labour that can be provided by the household – it may be skilled, but in locally used techniques (such as wattle and daub construction).

For the poor the latter two costs may be heavily discounted when compared to any cash outgoings, and consequently they will be are separated from the first three 3 in costings presented in this report.

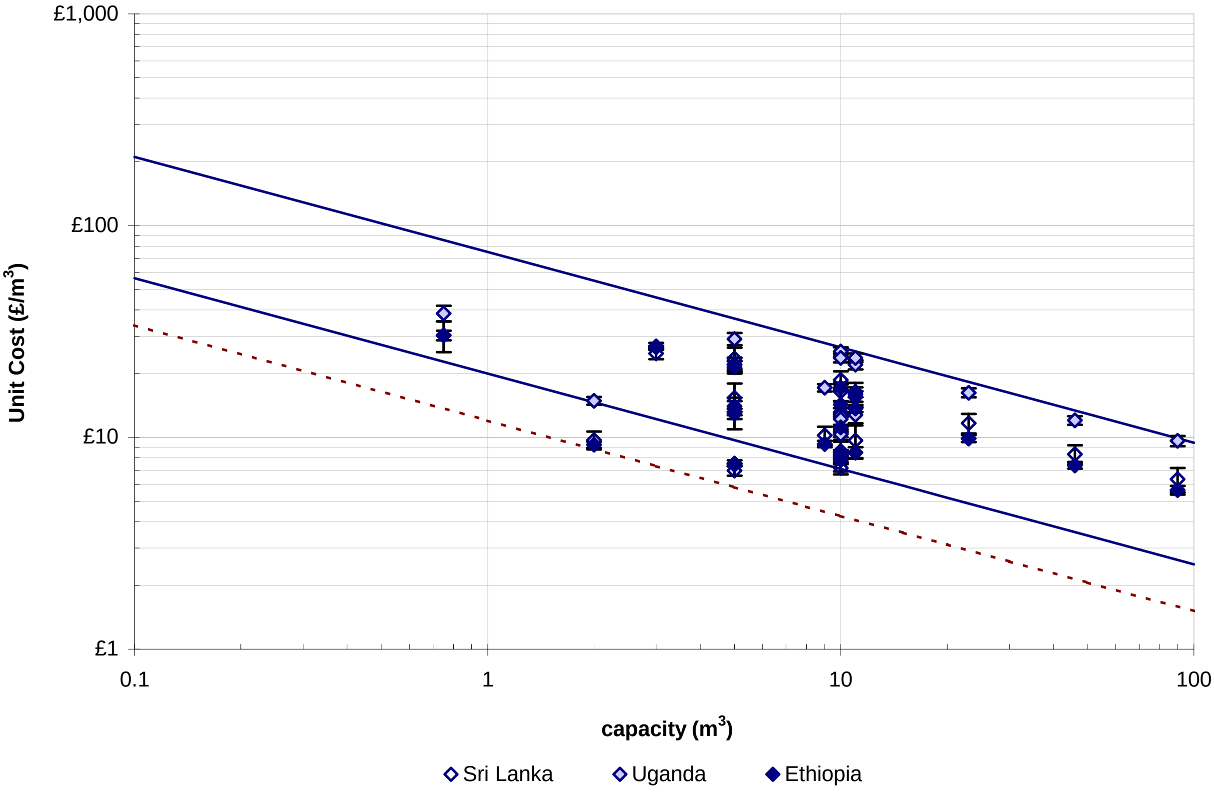
Household labour is often ignored in costing programmes aimed at the poor, however this may become unrealistic if the household labour contribution becomes burdensome. Another approach is to allocate an opportunity cost of 50% of the unskilled labour rate to such a contribution to reflect the loss of time to the household. The real value of their time to a householder should lie between these two figures, so both will be quoted in this report They are referred to as "HH labour discounted" cost and "HH labour ignored" cost. A total cost with no discounting is also included for reference.

2 TANKS

2.1 Current state-of-the-art

Current practice is comprehensively described in report R1 so only a brief summary of current best practice will be given here as a baseline from which to judge new developments. Water storage forms the largest single cost component of a RWH system and so is the most likely candidate for cost reduction. Current state-of-the-art for storage tanks in developing countries is ferrocement construction, either based on a fixed solid mould or on an open framework. Generally, a solid mould results in a cheaper tank, as the wall thickness of the tank can be more tightly controlled and work is done against an inflexible surface. Open frameworks have the advantage of allowing greater flexibility in tank size but at the cost of plastering work taking place against an elastic backing which lets some mortar through resulting in greater and more variable wall thickness and a higher cost per tank. Tanks are also made from bricks which can reduce costs by material substitution but the savings are often not great although more of the money remains in the community. A graph of current costs of various techniques in the three study countries is given in Figure 2.1.

Figure 2.1: Tank costs



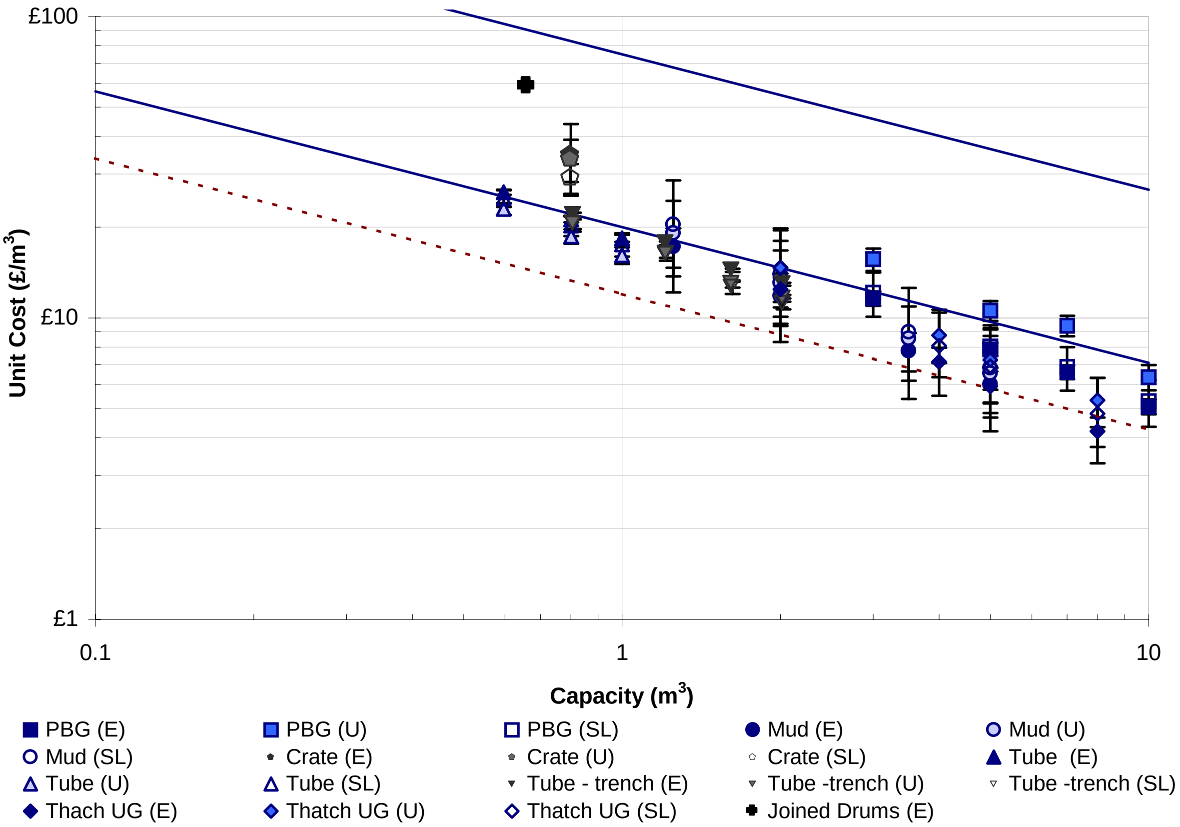
Notes: Points represent costs with household labour discounted, error bars represent the extremities when household labour is included in full or ignored altogether

Lines slopes correspond to a cost:capacity sensitivity of 0.6

A cost sensitivity to volume of 0.6 – 0.7 is the norm, so designs can be usefully compared over a wide range of capacities. The Solid blue lines represent the bounds of normal state-of-the-art. the dotted red line represents the lower limit of a region populated by “exceptional” designs such as the Thai jar and the Ugandan tarpaulin tank. It is this region that forms the target cost range. Tanks developed during the prototyping phase of the programme fall mainly into the benchmark range as shown in Figure 2.2.

Most of the designs also have a greater reliance on household labour and local materials so compulsory cash costs are kept lower still (as indicated by the bottom extent of the error bars).

Figure 2.2: cost comparison of tanks developed during the prototyping phase



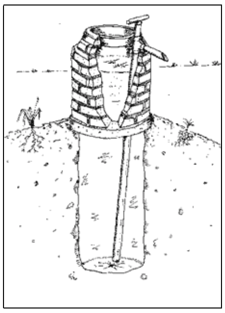
2.2 Below-ground designs

Below ground designs are the mainstay of low cost tank design. If built in stable soil, the ground itself will act as a stiff spring taking much of the load, reducing the role of the lining material to waterproofing. In these circumstances, tank wall thicknesses can be very small and materials that have good waterproofing properties but suffer from a low structural strength (such as polyethylene sheet) can be used.

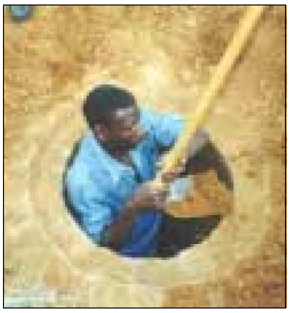
Tube tank

The tube tank was originally developed by the DTU in July 2000 (Rees & Whitehead, 2000), as part of a technology development project funded in part by the Morris Laing Foundation. The design was based around a widely available plastic tube of about 500 microns thickness sold by the metre on a roll of 3ft width. The cost of this tube is £0.38 in Sri Lanka, £0.54 in Ethiopia and £0.40 in Uganda. When opened, the tube forms a cylinder of Ø54cm resulting in a volume of 0.23m^3 per metre length. The cost of storage, is therefore only £1.66, £2.33, £1.75 (SL, Et, Ug) per m^3 of storage for the tube itself. The original design (see Figure 2.3) was based on the partially below ground principle where most of the tank was below ground, but about 1m protruded as a brick parapet, avoiding stormwater ingress and providing a visual presence for the tank. A hole was dug to accommodate the tube and a bag was formed by folding and tying off the end. The bag itself was fixed between two courses of bricks and an overflow pipe was fitted in a hole in the bag and sealed with a small amount of cement.

Figure 2.3: Original tube tank design. (all pictures from (Rees & Whitehead, 2000))



General arrangement of tube tank Mk 1



Digging the hole

Bag tying

Overflow arrangement and fixing of the bag between brick courses

The design was fairly successful and several of the tanks are in service however it has some problems, most which have become apparent after a year of use:

- The cost of the parapet wall dominates the overall tank cost and forces the design to compete directly with more robust and desirable designs such as the ferrocement jar
- The hole is difficult to dig
- The tied joint at the bottom can leak
- The liner is extremely difficult to remove if it is punctured – those that do leak have not been repaired
- The overflow can leak resulting in accumulation of water between the bag and the excavation walls

The new design (see Figure 2.4)addresses these problems by including the following changes:

- A more classic underground design using a precast concrete cover reduces the cost. The cover itself is similar to the arrangements found on some handpumps and so should be familiar to use. It is made using similar casting techniques to pit latrine covers (sanplats).
- The tube has been folded in two, eliminating the tied joint, making the hole 40% larger and consequently, easier to dig
- The tube itself is easily replaceable by using a retaining ring and binding wire to hold the tube in place.
- A variant folds the tank into a “L” shape reducing the depth of the excavation and replacing the vertical hole with a trench

Figure 2.4: Modified tube tank design.

General arrangement of tube tank Mk 2 | **Retaining ring** | **Overflow** | **Horizontal folded tube tank being filled**

With these modifications, the design is extremely inexpensive at less than £18 for a 700l tank and £26 for a 2m^3 folded tank. Although the prototypes were made using in-situ casting in the ground, the slab has designed with pre-casting in mind and so is well suited to factory production where savings of up to 50% can be made by drastically reducing labour content.

The tank design is also ideally suited to rapid implementation projects such as refugee camps; if the excavation is done by the householders, an agency can simply transport a number of prefabricated parts and each tank can be assembled within an hour.

Tank size is determined by hole depth so the deeper a household digs, the larger the store, extra storage is relatively cheap as the cost of the tank is dominated by the concrete slab. The alternative horizontal folded design is more suited to less stable soils as the excavation itself is about 60cm deep and can be tapered if necessary, however the cost is higher due to the need to cover the trench over. Costs of several capacities of each type of tube tank are shown against benchmarks in Table 2.1.

Table 2.1: costs of tube tanks

a. vertical

Capacity (m^3)	0.6	0.8	1.0
Total	£16 – 17	£16 – 17	£17 – 19
Total (HH labour discounted)	£14 – 16	£15 – 17	£16 – 18
Total (HH labour ignored)	£13 – 15	£14 – 17	£16 – 18
Target range	£9 –15	£11 –18	£12 –20

b. horizontal

Capacity (m^3)	0.8	1.2	1.6	2.0
Total	£17 – 18	£20 – 22	£21 – 24	£23 – 26
Total (HH labour discounted)	£17 – 18	£20 – 21	£20 – 23	£23 – 26
Total (HH labour ignored)	£16 – 17	£19 – 21	£19 – 23	£22 – 26
Target range	£11 –18	£13 – 22	£16 – 26	£18 – 29

Thin shell cement slurry lining

Neat cement is about 40% stronger and twice as flexible as mortar. It is also almost completely impermeable, making the material ideal for the construction of very thin shells to line pits. Initial experiments on a 1m depth pit of Ø1.5m revealed leakage of less than 10 litres/day and the surface showed no cracks. A larger pit of 3m depth was also tried, however the cement was applied without due care and cracks developed. The leakage from this pit is 80 litres/day, however no additional cracking has been observed as a result of the significant head of water acting on the shell.

The thin shell cement lining technique can result in material economies of up to 50% when compared to conventional mortaring, however the care with which it must be applied to the soil may make it unsuitable to unsupervised (free market) construction at a local level.

Direct application of mortar

It is common practice to put mesh in an underground tank when lining with mortar. Typically this takes the form of a layer or two of chicken wire. The given reasons are that it:

- adds strength
- provides a surface upon which to plaster (lath)
- Reduces cracking

Looking at the first of these reasons, a single layer of chicken mesh will have an in-line volume fraction less than 0.2% when compared to the mortar (assuming a 1cm mortar layer), so its contribution to the strength of the composite is fairly minimal (about 20% using the law of mixtures). Of course, the strength of an underground tank is mostly from the reaction of the soil itself.

The second may be true, but the initial layer of mortar is usually applied to the hole *adding* the mesh (indeed placing mesh directly on the soil is risky as it will certainly corrode from contact with moist soil). Further layers can be built up by scoring the original thus providing keys in the mortar itself. Such scoring is also simple to add to a hard soil and should provide a good keying for mortaring.

Crack reduction can be a useful property, particularly in tanks with large surface areas, however in smaller, domestic tanks cracking is not big problem as witness the popularity of mortar tanks in Thailand.

Thus the supposed advantages of using mesh may be insubstantial in this application and are certainly small compared to the advantages of using the strength of the soil wall.

Conversely, the mesh itself will cause problems when added to a concave surface such as the inside of a hole. The problem is basically one of overconstraint. If one considers, for the sake of argument a flexible sheet placed inside a cylinder. It can be either just the right size, too small or too big. If it is too big, the loose mesh forms bubbles, losing the benefit of the earth wall's support, if it is too tight it will stretch tight across the wall rather than following the curve, resulting in voids. Adding fixing points is the usual solution to these difficulties but this often simply turns one large problem into several small ones. The more fixings, the better the fit but also the greater likelihood of human error – the result can even be areas of too tight mesh next to bubbles of too loose mesh.

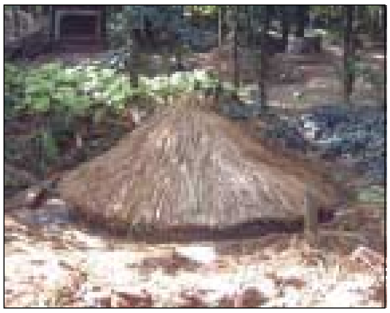
Directly applying mortar to the walls has proven to be a simple technique to apply in the field. A thin layer of 1cm can be applied with ease and with good quality control. The mortar itself has no need for high strength so can be as lean as 1:8 (cement:sand) in a good soil. Waterproofing is provided by a thin cement slurry applied while the mortar is wet. Several tanks have been built using this method to depths of up to 2.5 meters with no visible cracking and lower leakage than evaporation. The direct application of lean mortar with a slurry coat is the basis for two interrelated designs of tank, the below-ground cement tank with a cheap roof based on organic material and the partially below ground tank with a ferrocement dome.

Below-ground cement tank with organic roof


Ring beam with trench


Shuttering for pump and overflow mounting


low-cost tank cover under construction


Finished tank with cover in place

As the cost of the below-ground waterproof tank is reduced, the cover of the tank becomes the dominant cost. Organic roofs are used on many buildings in poor households and so the skills to build them are common. The materials themselves also tend to fall into the "gatherable" class. To put an organic roof onto a water tank, however, a number of precautions must be taken.

- The organic material must not fall into the tank and contaminate the water
- Runoff from the organic roof will be of low quality and so must not be allowed to enter the tank
- The roof must provide a good barrier to vermin entry, especially as some creatures make their homes in thatch.
- The wooden supports must not be exposed to the humid atmosphere inside the tank which will make them liable to rot
- To aid bacteriological die off, the roof must provide a good barrier against sunlight entering the tank

A polyethylene barrier fulfils the need to protect the organic matter from moisture and also to protect the water from falling debris. If the joining is handled well, it can also act as an excellent seal – completed by the use of inner tubes around the rim. Prevention of water entry can be afforded by the use of a sloped ring beam which will divert the water away from the tank and into a drainage channel.

Below-ground tanks also need care with avoiding floodwater ingress and also with overflow arrangements. The new design uses a syphonic overflow by employing an upwardly facing elbow connected to an outflow pipe which leads either to a nearby slope or to an infiltration pit. Stormwater ingress is handled by digging a channel around the ring beam to a width and depth determined from the runoff.

The overall combination of direct mortar application and low cost roof yields a tank that uses very little material but is quite householder labour intensive. The costs for the tank are shown in Table 2.2.

Table 2.2: Cost of underground mortar tank with organic roof

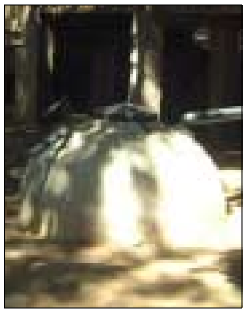
Capacity (m^3)	2	4	5	8
Total	£28 – 40	£32 – 43	£33 – 46	£37 – 51
Total (HH labour discounted)	£25 – 29	£29 – 35	£30 – 36	£34 – 43
Total (HH labour ignored)	£19 – 23	£22 – 29	£23 – 29	£26 – 3`5
Target range	£18 – 29	£26 – 43	£26 – 48	£38 – 63

Partially below-ground tank with ferrocement dome

Lining the pit

Fixing the frame for the dome

first layer of mortar for the dome



Finished tank

A solution to the overflow and floodwater problems of an underground tank is a partially below ground design where most of the tank is underground takes advantage of the economies to be found from soil support but with some of the tank protruding. Partially below-ground tanks form a bridge between underground tanks and above-ground tanks. They are somewhat more expensive than purely underground tanks, as the above-ground section can cost 3-4 time more per unit volume than the underground section. The Below-ground cement tank can be upgraded to form partially below-ground tank by simply adding a ferrocement domed cover. The dome uses a removable frame that leaves behind only wire mesh as reinforcement. The mortar can either be applied without any other formwork by using one person outside to apply the mortar and one person inside to provide a backing (the addition of a small amount of sacking fibres to the mortar was found to help this process) or by making a temporary formwork from cardboard. The dome can be built when the tank is first commissioned or added later when more funds are available. The finished tank is slightly more expensive per litre than the organically roofed over tank but will require less maintenance. Costs for the tank are shown in Table 2.3.

Table 2.3: Cost of partially below-ground tank

Capacity (m^3)	3	5	7	10
Total	£37 – 43	£41 – 51	£49 – 62	£54 – 71
Total (HH labour discounted)	£35 – 39	£39 – 47	£46 – 57	£51 – 65
Total (HH labour ignored)	£30 – 35	£34 – 43	£40 – 52	£43 – 59
Target range	£22 – 37	£29 – 48	£35 – 58	£43 – 71

2.3 Above-ground designs

Above ground designs are generally more popular than below ground solutions, however the cost is often also higher as the tank must now cope with the full force of the water pressure acting on it. The principle of functional separation allows some scope for cost reduction. Waterproofing can be done by either mortar or a liner.

Fabric tank


end of fabric tank with bracket


top sheet showing rubber seal

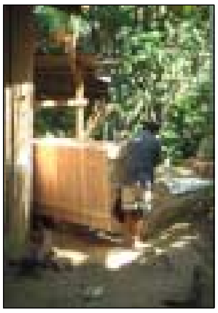

completed fabric tank

An above-ground tank is almost essential in poor crowded urban areas as the ground can be very contaminated. Ideally, a tank should also be fairly portable as tenure in such communities can often be insecure and many squatter communities live under constant threat of being moved on. The fabric tank goes some way to fulfilling these needs by providing a tank with a small footprint, protruding only 45cm from the dwelling. The tank can also be collapsed down into a long, thin package for transport. A polyethylene tube is placed within a fabric sleeve which is hung on a framework. The fabric takes the pressure load while the polyethylene provides waterproofing.

Unfortunately, the tank has proved problematic as the fabric itself stretches and puts high loads on its fixings which then fail. The fixings could be made stronger and more expensive fabrics should stretch less, however they will make the tank unaffordable.

Crate tank


lid with hole for inlet


delivering the crate to the household


Internal configuration


Finished tank in use

A similar design to the fabric tank has a wooden crate forming the load-bearing structure. The design has the same small footprint as the fabric tank but is not as portable as it can only easily be knocked down into its component walls. It is, however, much better protected from accidental damage than the fabric tank. The design is similar in concept to the tube tank with a retaining ring holding the top of the tube to the top of the tank providing an inlet. The tube than folds around and the other end is attached to the overflow. A tap is attached at the bottom of the “U” and sealed with bitumen. The outlet and overflow can be on any of three sides of the tank to help it fit in with its location. The total cost of the tank is slightly higher than the target range, however the need for a slender profile and portability may make the tank usable in areas where cheaper alternatives will be inappropriate. The manufacture of the tank employs only skilled labour but is very portable (deliverable) so it lends itself to mass production at a central location which could reduce the cost.

Table 2.4: Cost of crate tank

Capacity (m^3)	0.8
Total	£23 – 28
Target range	£11 – 18

Wattle and daub tank

Bamboo frame

Detail of manhole

Mud blocks used for testing

Finished tank

A simple way of producing an above-ground tank with the economy of a below ground tank is to bring the ground up. Several earth technologies have been used in building for millennia and such techniques are often the mainstay of housing for the poor. Wattle and daub is a widespread practice for building from earth, particularly when householders their own homes. The technique uses unmodified mud to fill a frame structure made from roundwood. The materials necessary for this type of constructions are all in the "gatherable" class so cash costs are extremely low, being limited to the liner and plumbing.

A small sample of mud blocks from different sources were tested for tensile strength and it was found that they lay in the fairly narrow range of 730kPa to 900kPa with ant-hill mud generally at the higher end. This compares poorly with 2 – 5MPa for Portland cement mortar so walls have to me made quite thick – typically 15 to 20cm. Initial tests used cement as a liner, however the mud structure expands slightly under load and the lining cracked resulting in leakage and damage to the mud walls. The use of a plastic liner has proved much more satisfactory.

Table 2.5: Cost of wattle and daub tank

Capacity (m^3)	1.25	2	3.5	5
Total	£25 – 37	£27 – 40	£31 – 45	£34 – 48
Total (HH labour discounted)	£22 – 26	£24 – 29	£27 – 32	£30 – 35
Total (HH labour ignored)	£16 – 18	£17 – 20	£20 – 23	£22 – 26
Target range	£14 – 23	£18 – 29	£24 – 40	£29 – 48

Rammed earth

The use of rammed earth provides a stiffer structure than wattle and daub and can be rendered internally with mortar. The earth can also be stabilised with Portland cement to provide a better wet strength than with mud construction. The DTU has done some previous work with this technique to build cylindrical tanks (Rees, 2000) however they proved difficult to construct due to movement of the round mould during manufacture. To combat this, trials have taken place to develop a rectangular tank using traditional corner jointing methods and reinforcing. The trials are thus far incomplete but show some promise for a controllable product that can be used in sandier soils where mud construction is impossible.

Connected drums

Drums form an easily accessible form of storage and many households already have at least one drum performing various duties, among them water storage and even opportunistic rainwater collection. Very few households employ more than one such drum however despite the extra water this could provide. Drums also have a small footprint and are light enough to be portable. They do suffer from problems such as the possibility of residual toxicity and solar heating of the water through the metal.

Several configurations of 2 or 3 drums have been tried, including two forms of vertical connection and one horizontal one. The pros and cons of these are detailed in Table 2.6. Initial Cleaning of the drums is usually done by using a locally produced caustic soda, however there is still some uncertainty about its effectiveness.

Table 2.6: pros and cons of different drum configurations

Type	Pros	Cons
1. Vertically stacked drums connected by means of pipefitting.	• Doesn't require welding or electricity. • The piped outlet allows incorporation of a slow-sand filter • Small footprint	• There is some leakage in the lower drum between the lid and circular clamp.
2. Vertically stacked drums connected by means of welding.	• The number of fittings is reduced • There is no leakage in the system • Small footprint	• Depends on the availability of a welder.
3. Horizontally placed drums connected by means of pipefitting	• An unlimited number of drums can be connected without leakage from the top • Repeated filtering provides different grades of water. • Doesn't require welding or electricity	• The required space is relatively higher compared to others and thus the material for the plinth is greater

The final design uses two vertically stacked drums with welded seams to prevent leakage and an internal slow sand filter. As the slow flow through the filter could reduce the storage, particularly in heavy storms, an optional separate tank has been added to catch this overflow. The overall cost is quite high by very-low-cost standards but still within state-of-the-art boundaries especially considering the design incorporates quite sophisticated filtering.

Table 2.7: Cost of drum configurations[1]

Type	1	2	3	final
Number of drums	2	2	3	3
Capacity (litres)	340	420	620	540
Total	£30	£31	£36	£38
Target range	£7 – 11	£7 – 12	£9 – 15	£9 – 14
Upper limit of state-of-the-art	£41	$47	£58	£53

Note: Costs are based on Ethiopian experience

Several connected jerry cans

Many households already have a fair storage in the form of several jerrycans. The cans themselves are inherently portable and can be distributed throughout the dwelling to save space and so are a good solution for crowded urban areas. The requirement is for a system of plumbing to connect these cans together to form a continuous storage. The cans then fill up in turn with the water quality improving in each successive can. A system of siphons empty the plumbing into the last can or to an overflow so there will be no leakage when a can is removed for use, however the problem of accidentally leaving a can out of the chain (with disastrous consequences when it rains) is still unsolved. The cost of the cans themselves is also a problem and could make the system uneconomic unless the household already has a reasonable number available.

Cascade of water jars

Many households use water jars for water collection. These jars are well suited to stacking in an array. The water runs down the outsides of one jar into the mouth of another with quality improving as it moves down the cascade. The rounded jars do allow the water to progress unhindered to the next, however the overall storage is quite low at 75 litres. The framework necessary is also quite expensive and puts the entire array into the higher end of the state of the art at about £16. the jars themselves are fairly low in price, however and so a cheaper frame (e.g. a suspended chain) could make the system more economic but it will never be a very-low-cost option.

3 ROOFS

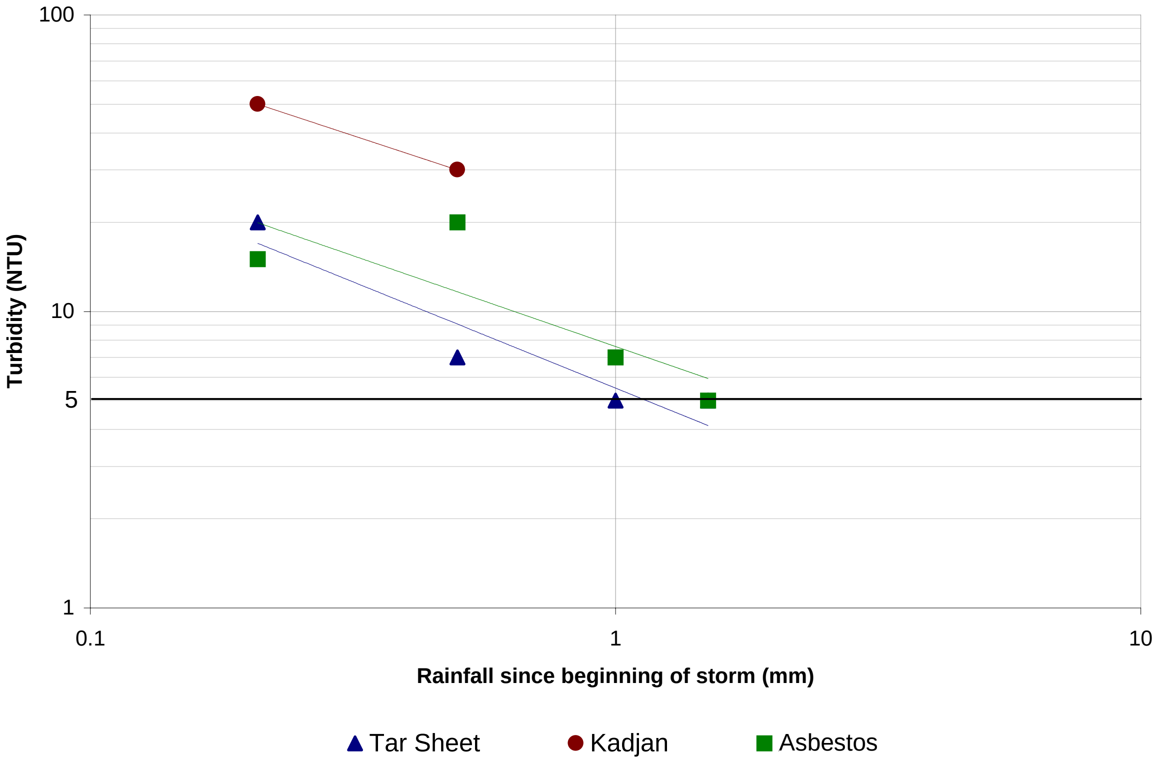
Another difficulty with roofwater harvesting is its reliance on an impermeable roof. Penetration of corrugated iron sheet roofs is growing rapidly and the costs are becoming competitive with professionally thatched roofs, however many poor houses in developing countries still have organic roofs. The reasons for this include cost, particularly where the roof is made by the householder, insulation and ventilation of the smoke from cooking fires.

There are five strategies for utilising organic roofs for rainwater harvesting;

1. Use the water for secondary processes
2. Treat the water
3. Cover the roof with an impermeable cover
4. Build a separate roof structure in the household compound
5. Use a nearby public building

Covering the roof and roof structures are the most obvious technical solutions, however, it was found that covering an organic roof with an impermeable barrier can cause problems to the roof itself, as water cannot evaporate presenting ideal conditions for mould growth, and the ultimate rotting of the structure. The runoff water quality from a number of materials for that might be employed to either to cover a roof or on a separate structure was measured. The materials tried were corrugated iron sheet, corrugated asbestos sheet, polypropylene sacking, tar sheet and palm-leaf organic roofing (*cadjan*).. A typical set of runoff quality results is shown in Figure 3.1. The characteristic runoff quality follows a power law decay with different materials having different starting points. Organic roofs predictably having the worst water quality while tar sheet, which is readily available and inexpensive has a good runoff characteristic, in line with asbestos but the water does contain petro-chemicals whose levels and rate of decay has not yet been quantified. Turbidity for the GI sheet is below the minimum level that can be discerned with the Del Agua turbidity meter used for this measurement and is therefore below the 5NTU level for turbidity set by the WHO (WHO, 1997)

Figure 3.1: Runoff quality of different roof types



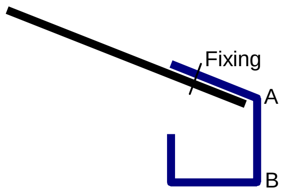
4 GUTTERS

Guttering on a very-low-cost roofwater harvesting system can take up a substantial amount of the cost so its optimisation is important here. Typical state-of-the-art gutters in developing countries tend to be quite expensive with a typical 10m length costing from £10 to £25. Some work has been done in East Africa with vee shaped gutters which have a typical cost of £8 for a similar length. Research at Warwick on optimising gutter size based on carrying capacity suggests that a vee shaped gutter of only 7.5 cm width is sufficient to carry water from all but the most severe downpours and will deliver more than 90% of the water it catches. Such a small gutter should cost less than £3 for a 10m run.

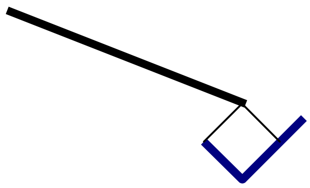
Water interception is a slightly more difficult issue. Water often has to fall some distance from the roof to the gutter and is thrown from the roof different distances depending on the intensity of the downpour. It can also be blown by wind in unexpected directions. Two solutions for this have been tried.

Figure 4.1: Gutters configurations

a. G-shaped gutter



b. extended vee



The first (Figure 4.1a) is a complete solution that captures the water at the end of the roof and directs it into the gutter below. The gutters are also very quick to install as the slope is determined by a variable manufactured length of vertical support (between A and B) so no adjustment is necessary. Cleaning is also simple as the inside edge is open for a brush all the way along its length. Problems with the gutter appear when the length to be guttered is longer than 5m or when thick roofs need to be accommodated. Under these circumstances the vertical support becomes very long and can flex causing the gutter to spill. This can be alleviated by using support wires with the loss of some ease of cleaning, however as the vertical support can use a substantial amount of material, the gutter starts to become expensive at over £9 for 10m.

The second uses the concept of a “upstand”, where one side of the gutter stands proud of the other, effectively raising the catchment height of the gutter. In the design the usual square gutter has been simplified to a vee and the upstand is merely an extension of one arm of the vee. This extends the catchment of the gutter upwards and moves the centre of the catchment out from the roof edge better matching the profile of water flowing from a roof. The gutter is extremely cheap (less than £3.50 for a 10m run) and can be applied to any sized roof without the need for a facia board. Like all suspended gutters, the design does need adjustment to maintain the slope and the suffers from guy wires obstructing cleaning.

5 CONCLUSION

The cost of rainwater harvesting systems can be reduced by a number of methods several of which have been presented here. The most promising are:

- The use of thin shell cement lining as found in the underground cement tank with an organic roof and the partially below-ground tank with a ferrocement dome
- The use of free materials and local techniques such as wattle and daub construction and organic roofs
- Earth technologies such as wattle and daub and rammed earth
- Mass production methods and the use of plastic linings as used in the tube tank and crate tank
- Incorporation of existing structures such as drums
- Smaller profile, sheet-metal gutters which can incorporate features such as water guides or extended catchment surfaces

Less successful were such techniques as:

- Distributed storage in jerrycans and multiple pots
- Very portable, yet fragile tanks such as the fabric tank

- Over-thin plastering techniques that rely on excessive quality control such as Thin shell cement slurry lining

Special circumstances can also be incorporated at little cost

- Tall thin structures such as the crate tank and vertically stacked drums can be built for crowded urban areas
- Auxiliary roofs can be constructed at little cost as an alternative to poor quality organic roof catchments

But

- Very portable, yet fragile tanks such as the fabric tank
- Roof covering treatments that are built onto an existing roof

Are problematic and cannot be recommended.

6 BIBLIOGRAPHY

Rees, D. (2000) *Stabilised Soil Tanks for Rainwater Storage*, (Report No. A3). Rainwater Harvesting Research Group.

Rees, D., & Whitehead, V. (2000) *Plastic Tube Tanks : Instructions for Manufacture,* (Report No. TR-RWH08). DTU, UK.

WHO (1997) *Guidelines for drinking-water quality. - Vol. 3 : Surveillance and control of community supplies,* World Health Organization Geneva.

APPENDIX

Bills of materials and costings

Material	Unit Quantity	unit costs			Quantity Used			Ethiopia			Uganda			Sri lanka		
		Ethiopia	Uganda	Sri Lanka	0.6m3	0.8m3	1.0m3	0.6m3	0.8m3	1.0m3	0.6m3	0.8m3	1.0m3	0.6m3	0.8m3	1.0m3
Tank																
cement	bag	£2.67	£6.10	£2.69	0.4	0.4	0.4	£1.03	£1.03	£1.03	£2.36	£2.36	£2.36	£1.04	£1.04	£1.04
sand	m3	£3.57	£3.05	£3.32	0.037	0.037	0.037	£0.13	£0.13	£0.13	£0.11	£0.11	£0.11	£0.12	£0.12	£0.12
aggregate	m3	£15.25	£2.98	£11.07	0.061	0.061	0.061	£0.94	£0.94	£0.94	£0.18	£0.18	£0.18	£0.68	£0.68	£0.68
Plastic tube	m	£0.58	£0.41	£0.37	3.6	4.66	5.7	£2.10	£2.72	£3.33	£1.46	£1.89	£2.32	£1.35	£1.74	£2.13
Bracket	each	£0.70	£0.70	£1.39	1	1	1	£0.70	£0.70	£0.70	£0.70	£0.70	£0.70	£1.39	£1.39	£1.39
Basin 20"	each	£1.33	£0.41	£0.52	1	1	1	£1.33	£1.33	£1.33	£0.41	£0.41	£0.41	£0.52	£0.52	£0.52
Inner Tube 20"	each	£1.08	£1.08	£0.93	1	1	1	£1.08	£1.08	£1.08	£1.08	£1.08	£1.08	£0.93	£0.93	£0.93
Labour (skilled)	day	£2.50	£2.03	£2.80	1	1	1	£2.50	£2.50	£2.50	£2.03	£2.03	£2.03	£2.80	£2.80	£2.80
Labour (unskilled)	day	£0.58	£1.22	£1.87	1	1	1	£0.58	£0.58	£0.58	£1.22	£1.22	£1.22	£1.87	£1.87	£1.87
Sub																
Pump																
1 1/2" pipe	m	£0.70	£0.83	£0.17	1.9	2.4	2.9	£1.32	£1.67	£2.02	£1.58	£2.00	£2.42	£0.32	£0.41	£0.49
1 1/4" pipe	m	£0.88	£0.73	£1.74	1.575	2.075	2.575	£1.39	£1.83	£2.28	£1.16	£1.52	£1.89	£2.75	£3.62	£4.49
1 1/2" tee	each	£1.50	£1.02	£0.50	1	1	1	£1.50	£1.50	£1.50	£1.02	£1.02	£1.02	£0.50	£0.50	£0.50
1/2"pipe	m	£0.00	£0.00	£0.47	0.2	0.2	0.2	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.09	£0.09	£0.09
wood (2X2)	m	£0.00	£0.00	£0.00	0.1	0.1	0.1	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00
wood screw	each	£0.00	£0.00	£0.01	2	2	2	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.01	£0.01	£0.01
Labour (skilled)	day	£2.50	£2.03	£2.80	0.5	0.5	0.5	£1.25	£1.25	£1.25	£1.02	£1.02	£1.02	£1.40	£1.40	£1.40
sub								£5.47	£6.26	£7.05	£4.77	£5.56	£6.34	£5.08	£6.04	£7.00
Total								£5.47	£6.26	£7.05	£4.77	£5.56	£6.34	£5.08	£6.04	£7.00
Total (discounted)								£5.47	£6.26	£7.05	£4.77	£5.56	£6.34	£5.08	£6.04	£7.00
Total (ignored)								£5.47	£6.26	£7.05	£4.77	£5.56	£6.34	£5.08	£6.04	£7.00

Underground tank with organic roof

Material	Unit Quantity	unit costs			Quantity Used				Ethiopiu				Uganda				Sri lanka			
		Ethiopia	Uganda	Sri Lanka	2m3	4m3	5m3	8m3	2m3	4m3	5m3	8m3	2m3	4m3	5m3	8m3	2m3	4m3	5m3	8m3
Tank																				
cement	bag	£2.67	£6.10	£2.69	0.9	1.5	1.6	2.2	£2.53	£4.10	£4.14	£5.90	£5.77	£9.38	£9.47	£13.47	£2.55	£4.14	£4.18	£5.94
sand	m3	£3.57	£3.05	£3.32	0.12	0.19	0.19	0.27	£0.43	£0.68	£0.70	£0.98	£0.36	£0.58	£0.59	£0.84	£0.40	£0.63	£0.65	£0.91
aggregate	m3	£15.25	£2.98	£11.07	0.06	0.06	0.07	0.07	£0.84	£0.84	£1.11	£1.11	£0.16	£0.16	£0.22	£0.22	£0.61	£0.61	£0.80	£0.80
6mm re-bar	m	£0.08	£0.16	£0.07	5	5	5	5	£0.39	£0.39	£0.39	£0.39	£0.78	£0.78	£0.78	£0.78	£0.33	£0.33	£0.33	£0.33
Binding wire	kg	£0.67	£0.81	£0.34	2	2	2	2	£1.33	£1.33	£1.33	£1.33	£1.63	£1.63	£1.63	£1.63	£0.67	£0.67	£0.67	£0.67
Bamboo	m																			
Thatch																				
Motorcycle tubes	each	£0.00	£0.00	£0.00	7	7	10	10	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00
Polythene	m2	£0.08	£0.14	£0.07	6	6	10	10	£0.50	£0.50	£0.83	£0.83	£0.81	£0.81	£1.36	£1.36	£0.45	£0.45	£0.75	£0.75
Basin	each	£1.50	£1.08	£1.08	1	1	1	1	£1.50	£1.50	£1.50	£1.50	£1.08	£1.08	£1.08	£1.08	£1.08	£1.08	£1.08	£1.08
24" Inner tube	each	£1.00	£0.93	£0.93	1	1	1	1	£1.00	£1.00	£1.00	£1.00	£0.93	£0.93	£0.93	£0.93	£0.93	£0.93	£0.93	£0.93
Labour (skilled)	day	£2.50	£2.03	£2.80	3	3	3.5	3.5	£7.51	£7.51	£8.76	£8.76	£6.10	£6.10	£7.12	£7.12	£8.41	£8.41	£9.81	£9.81
Labour (unskilled)	day	£0.58	£1.22	£1.87	11	11	12	13	£6.42	£6.42	£7.01	£7.59	£13.42	£13.42	£14.64	£15.86	£20.55	£20.55	£22.42	£24.29
Sub									£22.46	£24.29	£26.78	£29.40	£31.06	£34.88	£37.81	£43.28	£35.98	£37.81	£41.62	£45.52
Pump																				
1 1/2" pipe	m	£0.70	£0.83	£0.50	2	3.2	2.5	3.5	£1.39	£2.22	£1.74	£2.43	£1.67	£2.67	£2.08	£2.92	£1.01	£1.61	£1.26	£1.77
1 1/4" pipe	m	£0.88	£0.73	£0.43	1.675	2.875	2.175	3.175	£1.48	£2.54	£1.92	£2.81	£1.23	£2.11	£1.60	£2.33	£0.72	£1.24	£0.93	£1.36
1 1/2" tee	each	£1.50	£1.02	£0.45	1	1	1	1	£1.50	£1.50	£1.50	£1.50	£1.02	£1.02	£1.02	£1.02	£0.45	£0.45	£0.45	£0.45
1/2"pipe	m	£0.00	£0.00	£0.17	0.2	0.2	0.2	0.2	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.03	£0.03	£0.03	£0.03
wood (2X2)	m	£0.00	£0.00	£0.45	0.1	0.1	0.1	0.1	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.04	£0.04	£0.04	£0.04
wood screw	each	£0.00	£0.00	£0.01	2	2	2	2	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.01	£0.01	£0.01	£0.01
Labour (skilled)		£2.50	£2.03	£2.80	0.5	0.5	0.5	0.5	£1.25	£1.25	£1.25	£1.25	£1.02	£1.02	£1.02	£1.02	£1.40	£1.40	£1.40	£1.40
sub									£5.62	£7.52	£6.41	£7.99	£4.93	£6.81	£5.71	£7.28	£3.67	£4.79	£4.14	£5.07
Total									£28.08	£31.81	£33.19	£37.39	£35.99	£41.69	£43.53	£50.56	£39.66	£42.61	£45.76	£50.60
Total (discounted)									£24.87	£28.60	£29.69	£33.59	£29.28	£34.98	£36.21	£42.63	£29.38	£32.33	£34.55	£38.45
Total (ignored)									£21.66	£25.39	£26.18	£29.80	£22.57	£28.27	£28.89	£34.70	£19.10	£22.05	£23.34	£26.31

Partially below ground tank

Material	Unit Quantity	Unit costs			Quantity Used				Ethiopia				Uganda				Sri lanka			
		Ethiopia	Uganda	Sri Lanka	3m3	5m3	7m3	10m3	3m3	5m3	7m3	10m3	3m3	5m3	7m3	10m3	3m3	5m3	7m3	10m3
Tank																				
cement	bag	£2.67	£6.10	£2.69	1.9	2.8	3.5	4.3	£4.99	£7.39	£9.31	£11.48	£11.40	£16.89	£21.28	£26.24	£5.03	£7.45	£9.39	£11.57
sand	m3	£3.57	£3.05	£3.32	0.2	0.3	0.4	0.5	£0.79	£1.06	£1.34	£1.63	£0.67	£0.90	£1.14	£1.39	£0.73	£0.98	£1.24	£1.52
aggregate	m3	£15.25	£2.98	£11.07	0.1	0.1	0.1	0.1	£0.84	£0.84	£1.11	£1.11	£0.16	£0.16	£0.22	£0.22	£0.61	£0.61	£0.80	£0.80
mesh	m3	£0.56	£0.75	£0.42	5.8	5.8	10.4	10.4	£3.24	£3.24	£5.77	£5.77	£4.39	£4.39	£7.81	£7.81	£2.42	£2.42	£4.30	£4.30
bolts	each	£0.07	£0.03	£0.03	10	10	16	16	£0.70	£0.70	£1.12	£1.12	£0.35	£0.35	£0.56	£0.56	£0.35	£0.35	£0.56	£0.56
Basin	each	£1.50	£0.41	£1.08	1	1	1	1	£1.50	£1.50	£1.50	£1.50	£0.41	£0.41	£0.41	£0.41	£1.08	£1.08	£1.08	£1.08
24" Inner tube	each	£1.00	£0.61	£0.93	1	1	1	1	£1.00	£1.00	£1.00	£1.00	£0.61	£0.61	£0.61	£0.61	£0.93	£0.93	£0.93	£0.93
Labour (skilled)	day	£2.50	£2.03	£2.80	5	5	5.5	5.5	£12.51	£12.51	£13.76	£13.76	£10.17	£10.17	£11.18	£11.18	£14.01	£14.01	£15.42	£15.42
Labour (unskilled)	day	£0.58	£1.22	£1.87	6.5	6.5	8.5	10	£3.80	£3.80	£4.96	£5.84	£7.93	£7.93	£10.37	£12.20	£12.15	£12.15	£15.88	£18.69
sub									£29.37	£32.04	£39.87	£43.21	£36.09	£41.81	£53.58	£60.62	£37.31	£39.98	£49.60	£54.87
Tooling																				
Frame		£0.56	£0.56	£0.56	10	10	16	16	£5.63	£5.63	£9.01	£9.01	£5.63	£5.63	£9.01	£9.01	£5.63	£5.63	£9.01	£9.01
6mm bar		£0.08	£0.16	£0.16	20	20	37	37	£1.57	£1.57	£2.91	£2.91	£3.13	£3.13	£5.79	£5.79	£3.13	£3.13	£5.79	£5.79
sub									£7.21	£7.21	£11.92	£11.92	£8.76	£8.76	£14.80	£14.80	£8.76	£8.76	£14.80	£14.80
sub/10									£0.72	£0.72	£1.19	£1.19	£0.88	£0.88	£1.48	£1.48	£0.88	£0.88	£1.48	£1.48
Pump																				
1 1/2" pipe	m	£0.70	£0.83	£0.50	2.2	3.4	2.9	3.9	£1.81	£2.64	£2.29	£2.99	£2.17	£3.17	£2.75	£3.59	£1.31	£1.92	£1.66	£2.17
1 1/4" pipe	m	£0.88	£0.73	£0.43	2.6	3.8	3.3	4.3	£2.01	£3.07	£2.63	£3.51	£1.67	£2.55	£2.18	£2.92	£0.98	£1.49	£1.28	£1.71
1 1/2" tee	each	£1.50	£1.02	£0.45	2.275	3.475	2.975	3.975	£1.50	£1.50	£1.50	£1.50	£1.02	£1.02	£1.02	£1.02	£0.45	£0.45	£0.45	£0.45
1/2"pipe	m	£0.00	£0.50	£0.17	1	1	1	1	£0.00	£0.00	£0.00	£0.00	£0.10	£0.10	£0.10	£0.10	£0.03	£0.03	£0.03	£0.03
wood (2X2)	m	£0.00	£0.00	£0.45	0.2	0.2	0.2	0.2	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.04	£0.04	£0.04	£0.04
wood screw	each	£0.00	£0.00	£0.01	0.1	0.1	0.1	0.1	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.01	£0.01	£0.01	£0.01
Labour (skilled)		£2.50	£2.03	£2.80	0.5	0.5	0.5	0.5	£1.25	£1.25	£1.25	£1.25	£1.02	£1.02	£1.02	£1.02	£1.40	£1.40	£1.40	£1.40
sub									£6.57	£8.47	£7.68	£9.26	£5.97	£7.85	£7.07	£8.63	£4.23	£5.35	£4.89	£5.82
Total									£7.29	£9.19	£8.87	£10.45	£6.85	£8.73	£8.55	£10.11	£5.11	£6.23	£6.37	£7.30
Total (discounted)									£7.29	£9.19	£8.87	£10.45	£6.85	£8.73	£8.55	£10.11	£5.11	£6.23	£6.37	£7.30
Total (ignored)									£7.29	£9.19	£8.87	£10.45	£6.85	£8.73	£8.55	£10.11	£5.11	£6.23	£6.37	£7.30

Crate tank

Material	Unit Quantity	unit costs			Quantity Used	Cost		
		Ethiopia	Uganda	Sri Lanka		Ethiopia	Uganda	Sri lanka
8 x 1 planks	m	£0.46	£0.45	£0.37	32.5	£15.06	£14.54	£12.15
tap	each	£0.92	£2.03	£0.70	1.00	£0.92	£2.03	£0.70
1/2" pipe	m	£0.25	£1.71	£0.17	0.20	£0.05	£0.34	£0.03
tap socket	m3	£0.21	£0.61	£0.21	2.0	£0.42	£1.22	£0.42
tap plug (?)	each	£0.21	£0.61	£0.21	1	£0.21	£0.61	£0.21
1" GI pipe	m	£0.25	£1.71	£0.70	0.2	£0.05	£0.34	£0.14
plastic tube	m	£0.58	£0.41	£0.37	5	£2.92	£2.03	£1.87
20" Basin	each	£1.33	£0.41	£0.52	1	£1.33	£0.41	£0.52
20" Inner tube	each	£1.08	£0.61	£0.93	1	£1.08	£0.61	£0.93
bracket	each	£0.70	£0.70	£0.70	1	£0.70	£0.70	£0.70
Labour (skilled)	day	£2.50	£2.03	£2.80	2	£5.00	£4.07	£5.61
Labour (unskilled)	day	£0.58	£1.22	£1.87	0	£0.00	£0.00	£0.00
Sub						$27.75	$26.90	$23.27
Sub (discounted)						$27.75	$26.90	$23.27
Sum (ignored)						$27.75	$26.90	$23.27

Wattle and daub tank

Material	Unit Quantity				Quantity Used				Ethiopia				Uganda				Sri Lanka			
		Ethiopia	Uganda	Sri Lanka	1.25m	2m	3.5m	5m	1.25m	2m	3.5m	5m	1.25m	2m	3.5m	5m	1.25m	2m	3.5m	5m
cement	bag	£2.67	£6.10	£2.69	0.4	0.5	0.6	0.8	£1.10	£1.36	£1.63	£2.07	£2.51	£3.12	£3.72	£4.73	£1.11	£1.38	£1.64	£2.09
sand	m3	£3.57	£3.05	£3.32	0.06	0.07	0.09	0.11	£0.21	£0.26	£0.31	£0.40	£0.18	£0.22	£0.27	£0.34	£0.20	£0.24	£0.29	£0.37
aggregate	m3	£15.25	£2.98	£11.07	0.10	0.12	0.15	0.18	£1.50	£1.86	£2.22	£2.82	£0.29	£0.36	£0.43	£0.55	£1.09	£1.35	£1.61	£2.04
Barbed wire	m	£0.03	£0.03	£0.03	31	41	50	66	£0.88	£1.14	£1.40	£1.84	£0.96	£1.25	£1.53	£2.01	£0.98	£1.27	£1.57	£2.05
Binding wire	kg	£0.67	£0.81	£0.34	2	2	3	3	£1.33	£1.33	£2.00	£2.00	£1.63	£1.63	£2.44	£2.44	£0.67	£0.67	£1.01	£1.01
Plastic	m2	£0.39	£0.27	£0.25	7	10	13	17	£2.83	£3.73	£5.22	£6.42	£1.97	£2.59	£3.64	£4.47	£1.81	£2.38	£3.34	£4.11
String	m	£0.00	£0.02	£0.01	0	0	0	0	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00	£0.00
Mud	m3																			
Bamboo	m																			
Thatch																				
Basin	each	£1.50	£0.41	£1.08	1	1	1	1	£1.50	£1.50	£1.50	£1.50	£0.41	£0.41	£0.41	£0.41	£1.08	£1.08	£1.08	£1.08
24" Inner tube	each	£1.00	£0.93	£0.93	1	1	1	1	£1.00	£1.00	£1.00	£1.00	£0.93	£0.93	£0.93	£0.93	£0.93	£0.93	£0.93	£0.93
1 1/4" pipe	m	£0.88	£0.73	£0.43	1	1	1	1	£0.88	£0.88	£0.88	£0.88	£0.73	£0.73	£0.73	£0.73	£0.43	£0.43	£0.43	£0.43
1 1/4" elbow	each	£0.21	£0.21	£0.21	1	1	1	1	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21
1/2"pipe	m	£0.25	£0.83	£0.50	0.5	0.5	0.5	0.5	£0.13	£0.13	£0.13	£0.13	£0.42	£0.42	£0.42	£0.42	£0.25	£0.25	£0.25	£0.25
1/2" elbow	each	£0.21	£0.21	£0.21	1	1	1	1	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21	£0.21
1/2" socket	each	£0.21	£0.21	£0.21	3	3	3	3	£0.63	£0.63	£0.63	£0.63	£0.63	£0.63	£0.63	£0.63	£0.63	£0.63	£0.63	£0.63
Tap	each	£0.92	£2.03	£0.82	1	1	1	1	£0.92	£0.92	£0.92	£0.92	£2.03	£2.03	£2.03	£2.03	£0.82	£0.82	£0.82	£0.82
Labour (skilled)	day	£2.50	£2.03	£2.80	2	2	2	2	£5.00	£5.00	£5.00	£5.00	£4.07	£4.07	£4.07	£4.07	£5.61	£5.61	£5.61	£5.61
Labour (unskilled)	day	£0.58	£1.22	£1.87	11	12	13.5	14	£6.42	£7.01	£7.88	£8.17	£13.42	£14.64	£16.47	£17.08	£20.55	£22.42	£25.23	£26.16
Sub									£24.75	£27.17	£31.14	£34.20	£30.59	£33.44	£38.13	£41.25	£36.58	£39.89	£44.85	£48.00
Sub (discounted)									£21.53	£23.66	£27.20	£30.11	£23.88	£26.12	£29.90	£32.71	£26.30	£28.68	£32.24	£34.92
Sum (ignored)									£18.32	£20.16	£23.26	£26.02	£17.17	£18.80	£21.67	£24.17	£16.02	£17.47	£19.63	£21.84

A collaboration between the DTU and Lanka Rainwater Harvesting Forum, ACORD Uganda and Water Action Ethiopia.

This document is an output from a project funded by the UK Department for International Development (DFID) for the benefit of developing countries. The views expressed are not necessarily those of the DFID

RN – RWH04

RWH Performance Predictor for Use with Coarse (i.e. Monthly) Rainfall Data

by

Terry Thomas

This document is an output from a project funded by the UK Department for International Development (DFID) for the benefit of developing countries. The views expressed are not necessarily those of the DFID.

CONTENTS

1. THE NEED TO MODEL RWH SYSTEM PERFORMANCE

In every roofwater harvesting system there is a trade-off obtainable between increasing system performance and increasing system cost. The medium of this trade-off is normally tank size and hence at the centre of RWH design is the optimisation of that size. Many methodologies for tank sizing have been published, several are reviewed by (Gould & Nissen-Petersen, 1999). Fortunately, as with all optimisation, the plot of performance (e.g. cost:benefit) against tank size is 'flat-topped' in the area of interest, so that a ±10% variation in size there has little influence on economic performance. The cost of a tank of given size can usually be readily assessed, but the performance of the system containing that tank cannot. We therefore seek a methodology for predicting performance over a system's expected life: one that suits both the surrounding constraints (e.g. of data availability and access to computing facilities) and the RWH system use envisaged.

It is only possible to *roughly* predict the performance of a RWH system. Performance depends upon many factors, so its prediction can only be as good as knowledge of those driving factors. Some of these like tank size and roof size, once selected, remain conveniently constant. By contrast future user-demand behaviour and climate are uncertain. Water demand may vary widely with house occupancy, social calendar and season. Rainfall is hard to forecast more than two days ahead. To model system performance we therefore essentially average such variables in some way, assuming for example uniform water demand or a climatically 'typical' year. Normally we use the past as a template for the future. Since the critical factors can be estimated but crudely, we will do well to predict performance measures within 5% accuracy.

There are many such measures, including *reliability* (fraction of days that demand is met), *satisfaction* (fraction of demand volume that is met), *efficiency* (fraction of run-off water that is used) and *water value*. These measures can be applied to a typical year, to a typical wet or dry season or to an exceptional year/season, such as the driest in the last decade. They can be expressed for a representative location or for a particular one – for which meteorological data must then be available.

Suitable meteorological information is actually, at least in developing countries, rarely available and affordable in the right form and for the exact location of interest. This note addresses the specific problem of inadequately rainfall data.

RWH performance models are generally based on 'mass' balances. At each time-step, the roof run-off belonging to that step is added to the volume (mass) in the tank and the user's draw-off is subtracted. Tests and corrections are applied to cover the three cases 'tank overflows', 'tank runs dry' and 'demand exceeds the water available'. The time-step may be 1 day or 1 month: which of these is appropriate is discussed below. There are further fine modelling details to be decided, for example whether inflow is assumed to *precede* draw-off within any one time-step or to follow it.

A distinction needs to be made between RWH systems with respectively *no* storage, *some* storage and *very large* storage, since modelling them has quite different data needs. Three flows are to be modelled - input, usage and overflow - normal interest is in usage and occasionally in overflow. Modelling a 'no storage' system is simple: all daily inputs in excess of daily demand are spilled. Both *volumetric demand satisfaction* and *reliability* can be simply calculated directly from the rainfall record. Modelling a 'very-large-storage' system is even simpler. If annual roof run-off R_a exceeds annual water demand D_a then *satisfaction* and *reliability* will be 100% and *efficiency* will equal D_a/R_a. Conversely if R_a is less than D_a then *satisfaction* and *reliability* will fall to R_a/D_a , while *efficiency* will rise to 100%.

The situation however becomes more complex when the storage volume can be approximated neither to zero nor to infinity. This situation is addressed by the rest of this Note.

2. DATA NEEDS OF RWH PERFORMANCE MODELS

The list of data required to run a performance-prediction model comprises

- roof area and a roofwater run-off coefficient (for the latter, a constant such as 0.85 is often used to approximate the very variable behaviour found in practice)
- nominal ('standard') daily water demand *D*
- the management strategy the user proposes to use for selecting demand on a particular day as a multiple, greater or less than 1, of nominal demand
- a past rainfall record long enough to act as a reliable guide to future precipitation patterns
- proposed tank size *V*

We may express the tank size via an associated water residence time *T*, defined as tank volume divided by nominal daily demand ($=V/D$). Generally the rainfall data to accurately drive any model of that tank needs to be expressed in time steps shorter than *T*. Low-cost (and thus low-security) RWH systems having tanks of under say 15 day's capacity (T<15) need daily data sets, whereas high-security RWH systems with '3-month' tanks may be modelled with monthly data.

The length of the requisite rainfall record depends mainly on the level of supply reliability sought. Large RWH systems that constitute a supply of last resort in arid areas must be modelled (Vyas V., 1999)with long data sequences (say 25 years) in order to pick up extreme climatic events. Low-cost systems can usefully be modelled with 5 or 10-year sequences or with incomplete records where corresponding portions of previous years' data may have to be pasted into gaps.

The desirability of using *daily* rainfall data in RWH modelling has been noted (Heggen, 1993). Table 1 explores the error introduced by employing monthly instead of daily data to drive a RWH performance model. For four climate types the model was driven firstly with daily rainfall and then with rainfall produced by uniformly spreading each month's rainfall uniformly across the days of that month. In both cases the average *reliability* over 10 years was computed. The 'error' tabulated is the difference between the two reliability estimates.

For the large tank, both with a low demand (D = 0.6 x mean daily roof run-off) and an unmeetably high demand (D = 1.2 x mean runoff), the use of monthly data introduces negligible error. An even larger tank would raise the *reliability* with the high demand towards its theoretical maximum of 83% and get an even close agreement between forecasts using actual and 'uniform' daily data. For the medium and small tank sizes however, there is a significant error introduced by using synthetic uniform ('monthly') data instead of actual daily rainfall.

Table 1. Computed *reliability* of RWH supply as a function of rainfall data used in model

Mean run-off = constant 100 l/day.
Demand = constant.
Measures are averaged over 10 years.
Forecasts using *actual* daily data compared with those using *uniform* daily rainfall = monthly / 30

Location, annual rainfall and climate type	Rainfall data used for model	Small tank, 700 l		Medium tank, 3000 l		Large tank, 12000 l	
		Nominal daily demand as percentage of mean daily roof run-off					
		60%	120%	60%	120%	60%	120%
Saiya, W Kenya	actual daily	0.79	0.51	0.97	0.69	1.00	0.76
1507 mm/year	uniform	0.83	0.39	0.99	0.51	1.00	0.76
Double rains	*error %*	*4%*	*-12%*	*2%*	*-18%*	*0%*	*0%*
Bangkok	actual daily	0.58	0.42	0.73	0.57	1.00	0.76
1500 mm/year	uniform	0.58	0.45	0.71	0.56	1.00	0.76
Monsoon climate	*error %*	*0%*	*3%*	*-1%*	*-2%*	*0%*	*0%*
Panama	actual daily	0.71	0.50	0.83	0.66	1.00	0.76
1500 mm/year	uniform	0.71	0.47	0.83	0.59	1.00	0.76
Long single rains	*error %*	*0%*	*-3%*	*0%*	*-7%*	*0%*	*0%*
Petrolina, Brazil	actual daily	0.45	0.27	0.65	0.47	0.96	0.76
mm/year	uniform	0.55	0.38	0.71	0.48	0.98	0.76
Semi-arid	*error %*	*11%*	*11%*	*6%*	*0%*	*2%*	*0%*

Notes: *(a) Reliability is the fraction of days in 10 years that demand was met.*

(b) The error measures the bias in the estimates of supply reliability introduced by using coarse (monthly) rainfall data instead of fine (actual daily) data.

(c) Shaded cells indicate reliability less than 50% and therefore not very suitable for RWH

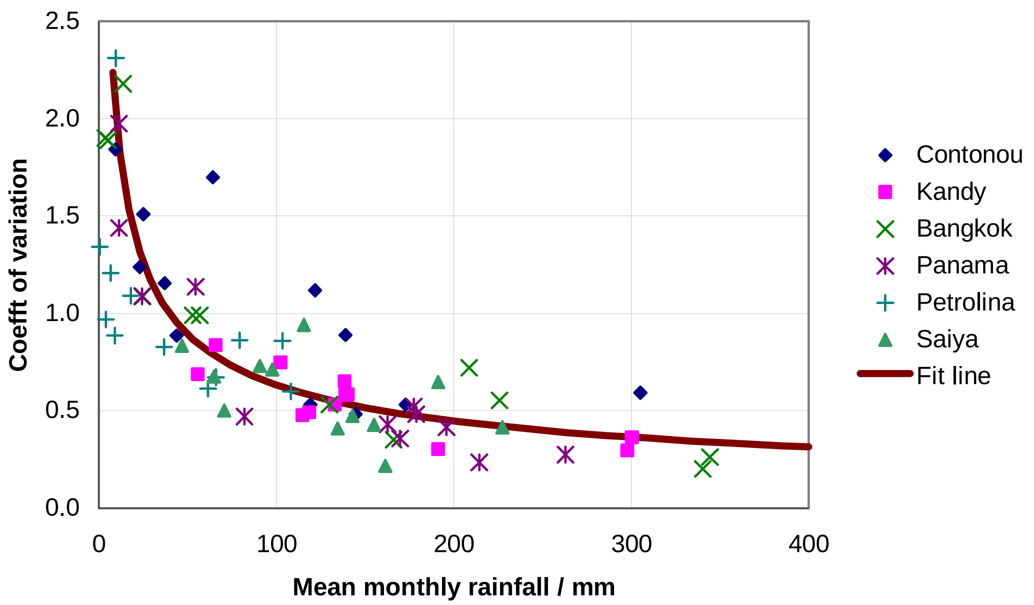
Tables similar to Table1 have also been produced to show not *reliability* but *satisfaction*. The values in such tables are of the order of 10% bigger than in Table 1 but the error introduced by employing only monthly data is similar. Other management strategies, such as rationing when the tank content is low, have also been modelled and show similar levels of error.

3. AVAILABILITY OF SUITABLE DATA

Meteorological data is rarely detailed, reliable and free. In many countries a 10-year daily rainfall record costs more than a small RWH system to buy. Rainfall varies with location, season and year. Its spatial variability is strongly influenced by topology and factors like distance from a coast. Its temporal variability becomes proportionally greater the lower the mean rainfall. The rough 'Fit line' in Figure 1 below shows that the variability, as measured by a coefficient of variation, may be expressed by $CoV = (R_m/40)^{-0.5}$ where R_m is the mean rainfall for that month in that location. The data is for two tropical sites from each of the Asian, African and American Continents.

In terms of geographical location, it may be possible to interpolate between neighbouring meteorological stations, or to use annual records to obtain a rainfall multiplication factor to apply to data from one place to make it better suit another.

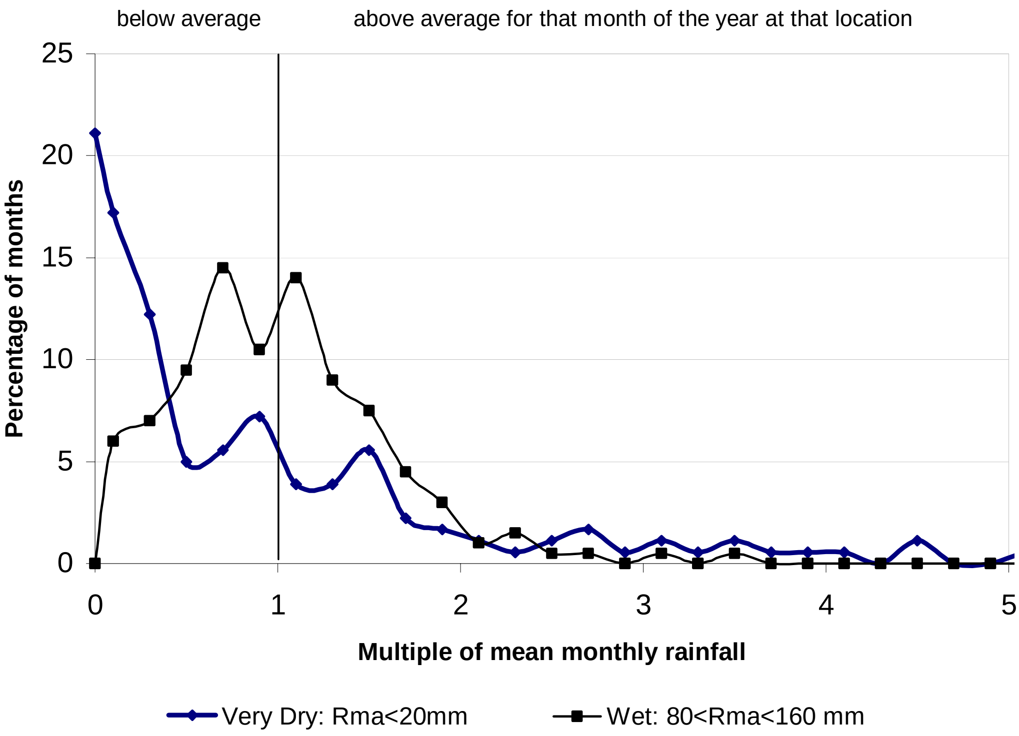
Figure 1. Variation in monthly rainfall (over 10 years & 6 sites)



In terms of time resolution, rainfall data is most readily available in ‘mean annual’ form, then as ‘mean monthly’, then as ‘actual monthly’ and least readily as ‘daily’. Even half-hourly data is available in richer countries with automatic recording equipment. Unfortunately ‘mean annual’ rainfall is little use for RWH modelling unless the modeller also possesses a library of representative seasonal distributions for the relevant region. ‘Mean monthly’ data is both too coarse and too time-averaged, but it is of some use and it is quite widely available (Pearce E & Smith C G, 1998). (Figure 2 shows amplitude distributions for 180 ‘dry-month’ tropical rainfalls and 200 ‘wet-month’ rainfalls normalised to the mean rainfall for the relevant calendar month and location. It confirms the greater variability and much higher chance of zero rain in months with a low average rainfall.) ‘Actual monthly’ data may be sufficient for quite accurate modelling as will be discussed below. Daily rainfall records may be regarded as effectively unavailable or unaffordable for most tropical sites.

Figure 2. Amplitude probability distributions

Monthly rainfalls placed in bands of width 20% mean monthly, or as zero rain



4. GENERATING PSEUDO DAILY RAINFALL DATA FROM ACTUAL MONTHLY RAINFALL DATA

In using monthly data directly we are effectively assuming constant daily conditions throughout that month – this is a very poor approximation and we could do better. From daily records for a climatic zone we could statistically characterise the rainfall distribution (in amplitude and sequence). We can then use these statistics to generate random rainfall data with the correct distribution and monthly totals.

In the normal collection and generation of rainfall data, daily records are condensed into monthly or annual records by the process below ($\rightarrow^{S}$ denotes summation and $\rightarrow^{A}$ denotes averaging).

$$\textit{Actual daily rainfall } (r) \rightarrow^{S} \textit{Actual monthly } (R_m) \rightarrow^{S} \textit{Actual annual}(R_a) \rightarrow^{A} \textit{Mean annual } (R_{aa})$$

$$\downarrow^{A}$$

$$\textit{Mean monthly } (R_{ma})$$

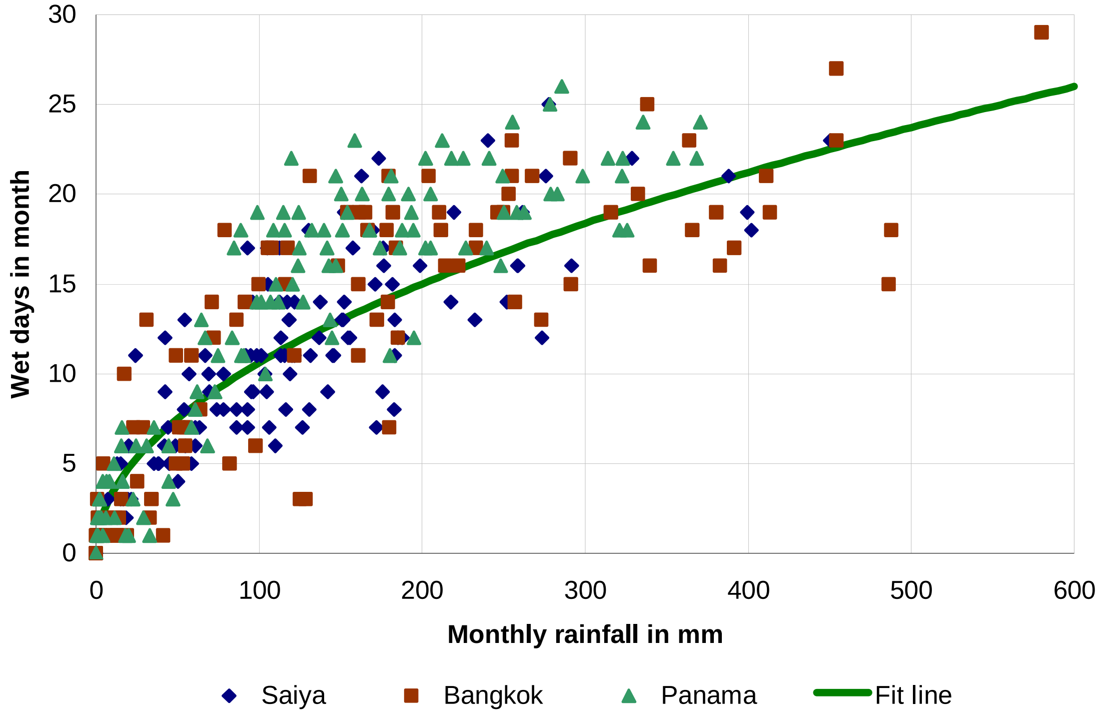
In generating pseudo data we reverse the process ($\rightarrow^{R}$ denotes randomisation)

$$\textit{Mean monthly } (R_{ma}) \rightarrow^{R} \textit{Pseudo monthly } (R'_m)$$

$$\textit{Actual } (R_m) \textit{ or pseudo } (R'_m) \textit{ monthly} \rightarrow \textit{Probability of wet day } (P) \rightarrow^{R} \textit{Pseudo daily } (r')$$

The amplitude distribution for daily rainfall we can treat as having two parts, an impulse at the origin corresponding to no-rain days and a generally falling curve away from the origin (reflecting that the lighter the rainfall the higher its probability of occurring). It is practical to replace these parts by a probability P (of a day being wet) and an amplitude distribution applicable only if the day is wet. The latter should give the correct wet day average R_w which in the humid tropics varies surprisingly little with variation in monthly rainfall (i.e. in wetter months there are primarily more wet days rather than wetter wet days).

Figure 3. Wet days per month (i.e. 30 P) versus monthly rainfall



The number of wet days in a standard month will be $n_w = 30\ x\ P$. Figure 3 shows a relationship between wet days and monthly rainfall and a crude fit line suggesting the number of them (and hence P) is proportional to square root R_m. In subsequent modelling we have therefore used $P = \mathrm{SQRT}(R_m/A)$ where $A = 800$ mm generally gives a good fit in the tropics. To maintain the correct monthly totals we have a mean wet-day rainfall $R_w = R_m / n_w$ and this ranges from around 6mm in a dry month to around 16mm in a very wet one.

The distribution of these wet days is open to debate. Table 2 (column 4) compares the actual probability of getting rain on successive days with that derived from a random (Poisson) rain-event sequence having the same mean wet-day probability P. ‘Rain yesterday’ clearly raises the probability of ‘rain today’ above its long-term value, indicating some bunching of rainy days. The table suggests that there is a higher than expectedly high chance of rain on successive days and a lower than expected chance of rain on alternate days. The auto-correlation of a long record of daily rainfall (with a 1-day shift) is also typically positive and of size 0.1 to 0.2. Seasonality explains some, but not all, of this apparent bunching

Table 2. Spacing between rainy days, actual *v* random (same average spacing)

Location	**Basis of prob-abilities**	**Probability of rain today**			
		In General $P = 1/ d_a$	**yesterday was wet**	**last rained 2 days ago**	**last rained >14 days ago**
Saiya, W Kenya	Actual	0.353	0.509	0.338	0.145
	Poisson	*0.353*	*0.353*	*0.353*	*0.353*
Bangkok	Actual	0.338	0.566	0.414	0.043
	Poisson	*0.338*	*0.338*	*0.338*	*0.338*
Petrolina NE Brazil	Actual	0.157	0.407	0.178	0.054
	Poisson	*0.157*	*0.157*	*0.157*	*0.157*

In computational terms, producing a suitable Markov process, whereby yesterday's turn-out affects today's probability, is complex and therefore to be avoided unless proven really beneficial. Initially, therefore, all modelling will be done assuming no such preceding-day influence – i.e. each day will be considered independent and only influenced by monthly rainfall. Thus wet days are taken to be randomly spaced and can be assigned using a random number generator ("wet if $X<P$" where X is a random number that is uniformly distributed between 0 and 1).

We next meet the task of assigning pseudo rainfall to each wet day. Examination of graphs suggest a roughly falling exponential amplitude distribution for wet day rainfall: $p_r = e^{-r/R}/R$ where p_r is the probability density in units of mm^{-1}. However it can be shown that the probability of very low and of very high rainfall is higher than this distribution would suggest. The modeller has therefore the options of using a straight exponential amplitude distribution – which is fairly easy to implement, or of 'tweaking' such a model with an additional tuning adjustment or of using some other distribution difficult to mimic. The second option was employed in this study. It can be shown that transforming a uniformly distributed random variable X (range 0 to 1) by the equation $r' = -R \ln(X)$ gives the generated rainfall r' the desired distribution, provided that the constant R is the mean wet-day rainfall R_w for the current month. The probability of low and high rainfall can be increased by processing X before taking its logarithm: a suitable algorithm is:

$$r' = -R \ln(\mathrm{Z}) \qquad \text{and} \qquad Z = \mathrm{a}X + \mathrm{b}X^2 + \mathrm{c}X^3 \qquad \text{where} \quad \mathrm{b} = 3(1\text{-a});\ \mathrm{c} = -2(1\text{-a})$$

Table 3. Procedure for generating pseudo-daily rainfall *r"* from monthly rainfall R_m

Step	**Procedure**
(i)	Estimate the wet-day probability *P* from the monthly rainfall R_m and hence also obtain mean wet-day rainfall R_w for that month,
(ii)	Use a random number generator in conjunction with the threshold *P* to decide if today is a wet day,
(iii)	If it is a wet day, generate a random rainfall *r'* for today which has a suitable amplitude distribution (assuming no influence from the previous day's rainfall)
(iv)	Re-scale all the daily *r'* values for the month to generate new values *r"* that add correctly to the given monthly total R_m

Using this procedure, pseudo rainfall sequences were generated for three tropical locations with respectively double rains, Monsoon rains and low erratic rains. The agreement between actual and pseudo daily rainfall – in terms of totals, rainy days and distributions as shown in

Table 4 – is quite good, and of course much superior to the agreement between actual and *uniform* simulated rainfall as shown italicised.

Table 4. Comparison of characteristics of actual, pseudo and 'uniform' daily rainfall

Location	Data type	Annual rainfall	Wet days per year	Wet days/year >30mm rain	Wet days/yr <10mm rain	Auto-correlation
Saiya W Kenya	Actual	1482	131	11	84	0.144
	Pseudo	1486	133	11	82	0.025
	Uniform	*1486*	*354*	*0*	*339*	
Bangkok	Actual	1546	121	15	75	0.250
	Pseudo	1551	113	16	65	0.149
	Uniform	*1551*	*318*	*0*	*270*	
Panama	Actual	1528	154	13	108	0.107
	Pseudo	1530	127	13	76	0.083
	Uniform	*1531*	*345*	*0*	*318*	
Petrolina NE Brazil	Actual	507	56	3	41	0.221
	Pseudo	506	65	3	46	0.094
	Uniform	*507*	*312*	*0*	*312*	

On the strength of this comparison we may now test the accuracy of employing synthetic daily rainfall data where we have no actual such data.

5. PERFORMANCE OF MODELS USING PSEUDO DAILY DATA

Table 5 mimics Table 1 except that now we are using more carefully generated pseudo daily rainfall. As before we compare predicted *reliability* for a range of RWH scenarios using respectively actual and pseudo rainfall data. The comparison is fairly good but not excellent. In the range of practical interest (*reliability* > 50%) the error does not exceed 5% and averages about 2%. As the error is always positive (pseudo data over-estimates performance) it might be prudent to subtract 2% from all *reliability* forecasts.

The source of what error there is may lie in the randomisation process itself – the fluctuations it produces may need longer than 10 years modelling to cancel out. More likely the practical decision to treat successive days as statistically independent, ignoring the small influence of one day's rain on the next day's likelihood of rain, has made the pseudo data slightly too favourable. Extensive natural clumping of rainy days is likely to increase the probability of tank overflow and hence degrade performance: such clumping is absent in the pseudo data.

The comparison was deemed sufficiently close to justify offering an open-access RWH system modelling service on the web at www.eng.warwick.ac.uk/dtu/rwh/model which is driven by user's monthly rainfall data and proposed system details and which yields *reliability, satisfaction* and *efficiency* estimates.

Table 5. Computed reliability of RWH supply as a function of rainfall data used in model

(Constant demand, actual daily data compared with pseudo daily rainfall)

Location, annual rainfall and climate type	**Rainfall data used for model**	**Small tank, 700 l**		**Medium tank, 3000 l**		**Large tank, 12000 l**	
		Nominal daily demand as percentage of mean daily roof run-off					
		60%	**120%**	**60%**	**120%**	**60%**	**120%**
Saiya, W Kenya	actual daily	0.79	0.51	0.97	0.69	1.00	0.76
1507 mm/year	pseudo daily	0.84	0.53	0.99	0.70	1.00	0.76
Double rains	*error %*	5%	2%	2%	1%	0%	0%
Bangkok, Thai	actual daily	0.58	0.42	0.73	0.57	1.00	0.76
1500 mm/year	pseudo daily	0.62	0.44	0.76	0.59	1.00	0.76
Monsoon	*error %*	4%	2%	3%	2%	0%	0%
Panama	actual daily	0.71	0.50	0.83	0.66	1.00	0.76
1500 mm/year	pseudo daily	0.72	0.51	0.84	0.66	1.00	0.76
Long single rains	*error %*	1%	1%	1%	0%	0%	0%
Petrolina, Brazil	actual daily	0.45	0.27	0.65	0.47	0.96	0.76
mm/year	pseudo daily	0.53	0.33	0.70	0.52	0.98	0.76
Semi-arid	*error %*	8%	7%	4%	5%	2%	0%

6. CONCLUSIONS

In the humid tropics, where economy rather than extreme water reliability is the normal design objective, RWH tank volumes may be as small as 7 days' mean roof run-off. Under these circumstances direct use of monthly rainfall data in RWH system modelling will lead to considerable bias in performance predictions.

The conflict, between wanting to use *daily* rainfall data to achieve unbiased predictions and having to make do with coarser available data like actual monthly or even mean monthly rainfall data, can be resolved by generating pseudo daily data. A RWH model that employs suitably generated pseudo daily data as its input gives performance predictions quite similar to one using actual daily rainfall. A procedure for generating such pseudo data, valid for tropical sites, has been identified and employed in a web-based modelling service.

The procedure assumes that 10 years *actual monthly* rainfall data is available for the proposed RWH system site. This condition can often not be met. Since *mean monthly* data is more often available, the user of the performance forecasts can use that instead and hope that an average year is a typical year. This hope is slightly optimistic but acceptable in most circumstances.

Meanwhile studies continue into whether *pseudo monthly* rainfall data, generated by randomising *mean monthly* data can reliably be used for RWH performance forecasting. Other modelling refinements, including using conditional probabilities in lieu of a fixed local value of wet-day probability P, may reduce prediction errors further.

7. BIBLIOGRAPHY

Gould, John and Nissen-Petersen, Erik (1999) *Rainwater catchment systems for domestic supply,* IT Publications London.

Heggen, R J, (1993) "Value of daily data for rainwater catchment", in Bambrah, G. K. (ed.), *Proc 6th Int Conf Rainwater Catchment Systems,* Nairobi.

Pearce E A & Smith C G (1998) *Hutchinson World Weather Guide,* Helicon Oxford.

Vyas V (1999). *Modelling water sources for rural communities* (SimTanka). Ajit Foundation.

Domestic Roofwater Harvesting
Research Programme

DTU

RN – RWH05

Domestic Water Supply using Runoff from the Roofs of Institutional Buildings

by

Terry Thomas

August 2002

This document is an output from a project funded by the UK Department for International Development (DFID) for the benefit of developing countries. The views expressed are not necessarily those of the DFID.

CONTENTS

PREFACE

This Research Note is based in part upon a 'mini-conference' on the topic held in June-July 2002 on the discussion forum RWH@JISCMAIL.AC.UK. RWH is used throughout this Note as an abbreviation for 'roofwater harvesting'. The Note incorporates the (DTU) paper used to initiate the discussion and the observations from some 16 contributions covering international experience and specific experiences in Brazil, Ethiopia, India, Sri Lanka and Uganda. These contributions are included in an appendix. Note that the topic is *not* institutional RWH in general, but only the supply of *domestic* water employing *institutional* roofs, for which the abbreviation 'IRWHDS' has been applied.

1. BACKGROUND

In their classic forms, 'institutional RWH' uses run-off from institutional roofs to meet the water needs of that institution and 'domestic RWH' uses runoff from the roof of a house to meet the needs of its inhabitants. However there is a hybrid form of RWH that bridges these two forms by using the 'spare' or 'unused' capacity of some institutional roofs to collect water for domestic use by households outside that institution.

Not all households possess of roofs of sufficient size or quality to practice their own roofwater harvesting. However community buildings such as schools and places of worship, or commercial buildings, may be present that possess large and potentially clean collection surfaces suitable for rainwater harvesting. Such roofs are already sometimes used for institutional water supply – for example school roofs supply water to school pupils. The abbreviation IRWHDS stands for 'Institutional Roof Water-Harvesting System for Domestic Supply'. An IRWHDS system bears some operational similarities to public water supplies using rock catchments (e.g. as in Gibraltar), to the sale of roofwater to neighbours by householders with large roofs and to the sale of ground-runoff water stored in private ponds and tanks.

Roof run-off is almost always of higher quality than ground run-off, the latter usually requiring treatment if it is to achieve potable quality. This note addresses specifically rainwater harvesting from roofs rather than other catchment surfaces.

2. THEORY OF IRWHDS AND DISCUSSION OF ITS VARIOUS FORMS

2.1. Arguments or circumstances favouring IRWHDS

1. Roofs of communal buildings are often hard (e.g. made of tile, iron or asbestos) even in districts where many private households still only possess soft (e.g. grass) roofs. Access to a hard roof is normally a pre-requisite for successful RWH.
2. There are economies of scale to be had from harvesting from large roofs. The sensitivity of tank costs to tank volume is about 0.6. So that a 5-fold increase in tank

size will halve its unit cost (cost per litre capacity), and a 15-fold increase will bring unit costs down below one third. There are likely to be similar economies in guttering and water treatment.

3. Institutional roofs are generally cleaner than domestic ones, being higher and thus less accessible to humans or vermin. Moreover all roofwater is chemically cleaner than water from most other sources and is often biologically cleaner as well.
4. Certain funding agencies would prefer supporting a ‘communal’ water supply scheme than subsidising supplies to individual households.
5. The presence of a large ‘communal’ tank greatly facilitates the supply of water by bowser to that community if and when all of its other water sources have failed during a drought.

2.2. Arguments or circumstances discouraging IRWHDS

1. Unless the institutional building is close to those households drawing water from it, significant extra collection time is incurred above that needed to collect from house roofs.
2. The management (e.g. rationing) of the water in a large tank shared by many households is difficult – it may require the staffing of a ‘water kiosk’.
3. The total area of institutional roof is rarely sufficient to capture adequate water for more than a small fraction of households in the institution’s catchment area.
4. There are few incentives for an institution to invest, in or to energetically maintain, a communal water supply. In almost all cases the operation of an IRWHDS system will not be the main purpose or interest of the institution, its neglect is therefore quite likely. For success a formal operational arrangement is likely to be required, probably commercial.

2.3. Styles of operation

Any style of operation of a IRWHDS has to address the issues of water management, cost recovery and selection of beneficiaries. It is likely that the (institutional) owner of the roof will require some reward. The maintenance of high water quality is likely to be more demanded of a IRWHDS than of a domestic RWH system.

Some likely modes of operation are

OP1. roof is primarily for institutional water supply with any surplus being gifted to associates of the institution, to ‘deserving’ or to influential households;

OP2. all water from the institutional roof is charitably supplied to deserving households;

OP3. the institutional roof is operated like a business asset, benefiting the institution by the sale of water by the litre, (a variant of this mode is to combine IRWHDS with mobile water-vending, so that the water is delivered by the business to the user households);

OP4. water from the institutional roof is harvested by the community with recovery of capital, maintenance and perhaps end-of-life replacement costs covered via access charges (per litre or per day);

OP5. individual households are allowed to each attach their own tank to part of a communal roof (like a 'sow with piglets') - a rent may be payable to the institution for 'use' of a portion of its roof. Note this mode loses the economy of scale of employing a very large tank.

The amount of water 'surplus to internal needs' will depend upon the sort of institution. Assume a location in the humid tropics with 1200 mm annual rainfall (60% of which reaches a user) and the provision from roofwater of 700 litres per 'institutional member' per year. This gives a rough requirement of $1m^2$ of roofing per such member just to meet their water needs. Only if this threshold is exceeded will there be excess water available for consumption by outside households via any of the operational modes above. If the 'institutional members' are resident on site (i.e. overnight as well as by day) their water requirements will be higher – perhaps 4000 litres per year each – and the threshold of availability of any surplus may rise to $6\ m^2$ per member.

Thus in a single-storey boarding school, hospital or prison there is unlikely to be an exportable surplus, since overall roof area per resident is generally under such a $6m^2$ threshold. (In a multi-storied institution of such a type, an exportable surplus is even less likely.) A day school providing only say 1000 litres per pupil per year (including teachers' families' consumption), may be able to export some of its roofwater. IRWHDS is most likely to be viable if the institution is an office or a place of worship, where almost all the roof run-off is available for export.

2.4. Water management options

As with all other forms of RWH, supply from an IRWHDS is more generous or 'easier' in the wet season than in the dry. Indeed it is so expensive or difficult to guarantee a plentiful supply towards the end of a long dry spell that it is rarely economical to rely solely on RWH at such times. Thus users of an IRWHDS are likely to have to both reduce their consumption and seek water from other sources at certain times of year. As the *unit* value of water to a household declines with increase in daily consumption, and as the effective cost of water from other sources rises in the dry season, there is an economic case for rationing water from any tank. Not only should some wet season run-off be retained for dry season use, but also the 'ration' (i.e. consumption) in the dry season should be lower than in the wet. Whatever precise strategy is decided in the long or short term, it needs to be implemented by controls on the quantities drawn by each user.

WM1 One style of water management is to allocate a declared amount to each household – an amount that varies with household size, with recent rainfall or with current tank contents.

WM2 Another is to permit purchase of any quantity of water, setting a per-unit price that varies by season in order to either achieve a social optimum or to maximise annual income.

In either of these control modes a regulator or bailiff is likely to be needed, directed by the 'owner' of the IRWHDS. By contrast, operation mode OP5 above (sow and piglets) leaves the management of water abstraction entirely to the individual households.

2.5. Cost recovery options

Any RWH system is characterised by the dominance of capital over running costs. Therefore construction has to be authorised by the owner of the necessary capital, whether accumulated before construction by potential beneficiaries, loaned by a financial institution or donated by a charity or government. (Physically the owner of the roof also has powers of embargo and there is a complex interaction between the justification of the roof as a building component and its justification as a water source.) The longevity of the hardware is likely to be at least 10 years, but the asset is hard to 'recover' (move or dismantle into saleable parts) by any owner in the case of users' failure to pay. Uncertainties in payback would be introduced by changes in the quality or accessibility of other water sources in the neighbourhood – including any increase in hard roofing in households.

Running costs increase substantially where staffing is required to issue or to deliver water and the latter will almost certainly have to be reflected in some sort of additional 'delivery charge'.

Thus we arrive at four main modes of cost-recovery

CR1 saving by beneficiaries prior to construction

CR2 loan followed by user subscription

CR3 loan followed by sale by the litre

CR4 gift with no expectation of recovery

2.6. Selection of beneficiaries

The discussion above of operating modes includes some mention of selecting beneficiaries. We can now formalise selection by grouping as follows, with selection of beneficiaries according to respectively:

SE1 their personal closeness to, or social leverage over, the institution

SE2 their geographical closeness to the institutional roof

SE3 their welfare status (e.g. water only to widows)

SE4 their participation in a savings and building group

SE5 their ability to pay for the water service

2.7. Health issues

The quality of water from a shared source needs to be higher, and more thoroughly monitored, than that of any single-household source. We may expect the chemical and physical quality of institutional roofwater to be better than that of other sources, although perhaps rather lacking in taste. The bacterial quality may be somewhat inferior to good groundwater sources. Several design options for obtaining cleaner or clearer water are more viable (in terms of cost and ease of operation) for large roof schemes such as IRWHDS than for small domestic schemes. These options include first-flush diversion, sedimentation buffering of inflows, chlorination and filtering at the outlet. Periodic testing should be applied in accordance with local practice for comparable sources such as protected shallow wells.

3. PRACTICAL EXPERIENCE WITH IRWHDS

IRWHDS is poorly documented in the RWH literature. Gould & Nissen Petersen[1], who devote a chapter of their book to describing numerous examples of domestic RWH around the world, make no specific reference to IRWHDS. The practice has not been described in papers at any of the last three IRCSA conferences (Iran 1997, Brazil 1999 and Germany 2001). Communal and commercial RWH systems are quite numerous, but the water harvested seems to be used only by members of the organisation owning the roof, or for aquifer replenishment, and almost never by neighbouring households.

Participants in the mini-conference did identify a few applications. This in no way could be described as a controlled survey, so the number of systems described has no significance. However the reports give some insight into choices between the many options listed in sections 2.3 to 2.6 above.

In section 2.3 it was argued that 'institutional roofs' capable of yielding surplus water (over institutional needs) were likely to be on places of worship or day schools, rather than on residential establishments. In the examples reported, all were indeed either temples/churches or day schools. Participants also observed that establishing any management/maintenance organisation for buildings such as markets, sports centres and government offices is more difficult than doing so for a church or school. Moreover of the five modes of possible operation (OP1 to OP5), no examples of OP3 (roof managed as an income generating asset) or OP5 ('sow and piglets' attachment of individual tanks) were mentioned. Most reported IRWHDS systems were operated on a basis of partial cost-recovery and with priority given to the (small) water needs of the roof-owning institution.

Cost recovery and tank management is clearly no casual matter and generally requires some sort of water bailiff, whether paid or working voluntarily, with powers to control water issue. (Padlocks are mentioned). Indeed the development of a strong water management structure is asserted to be the key to success in IRWHDS.

Even so, the schemes described were generally sized too small to meet all the water needs of the assigned beneficiaries (or put another way, the choice of beneficiaries was inappropriate, their number well exceeding the system's capacity). One underlying constraint is the relatively small ratio between institutional and domestic roof areas. Often an institutional roof has only say 10 times the area of a typical house's roof, and the latter represents a just-adequate RWH collection area. It hardly justifies the complexity of establishing an IRWHDS management to supply under 20 households.

No survey of overall institutional roof area per inhabitant has been undertaken. A crude calculation (for single storey schools in a country where schoolchildren comprise 25% of the

---

[1] Gould J & Nissen-Petersen E, 1999, *Rainwater Catchment Systems*, Intermediate Technology Pubs, Chap 9

population and where there is 0.4 m^2 of school roof per schoolchild) shows only some 0.1 m^2 of school roof per inhabitant. On this basis, such roofs could meet the water needs of only a tiny fraction – e.g. under 2% - of the population. This raises big problems of water allocation that the institution would normally wish to avoid.

No examples of wholly 'commercial' operation were reported and the capital for construction seems always to have come from government or NGOs, sometimes supplemented by 'revolving fund' savings. This may reflect the legal or social difficulties of any 'public' institution exploiting its assets via a commercial intermediary. However the ideology of aid and development is currently shifting towards the creation of 'livelihoods' within a market economy, so the practice of an institution franchising its roof to a local small business may become more attractive. It seems clear that without an unusual local 'champion' any activity as marginal to an institutions main purpose as utilising its roof for water supply will receive poor institutional support. So devolving any RWH activity to a separate body from the institution itself seems necessary for IRWHDS success. In the reported examples, that devolved body was some sort of committee. Committees function well if their remit is of high priority or community-wide. IRWHDS justifies the description of high priority only under conditions of serious water scarcity; it can rarely be community-wide.

Institutional RWH is sometimes seen as a good demonstrator for domestic RWH and some of the reports hinted at that being the main justification for engaging with IRWHDS.

4. SCOPE FOR IRWHDS

4.1. Scenarios

The scope for employing IRWHDS is strongly limited by the lack of adequate institutional roof area to serve the bulk of the population. It is further limited by the absence of well-established models of management and dependence on rather unsustainable stop-gap modes of operation. IRWHDS is likely to be viable when RWH itself is viable (suitable rainfall, problems with other sources) yet most households cannot for some reason practise it. In a 'charity' mode it may be a useful way of alleviating water scarcity of very poor people living in grass-roofed houses. In a more commercial mode it may offer poor institutions a means of earning some extra money. However both field observations and logical analysis suggest IRWHDS will never have a large role.

4.2. Technology for IRWHDS

IRWHDS requires the same technology as other forms of institutional RWH. The tanks and gutters are larger (and cheaper per capita supplied) than in domestic RWH systems. Water treatment by sedimentation or chlorination is both more necessary and easier to organise than in domestic RWH systems. Instrumentation or equipment to support tight water management seems essential.

4.3. Economics of IRWHDS

As with all other forms of RWH, the economic return from an IRWHDS system is improved by using comparatively small tanks and designing for only modest levels of demand satisfaction. A well-managed IRWHDS system should be able to provide water at 40 to 60% the cost from a single-house RWH system. Each IRWHDS system offers some 14 hours per week of paid employment , and if well-managed can offer water at 50% to 60% the cost of water from domestic RWH of similar reliability and condition of say 5-year amortisation of construction cost.. This tariff includes the employee cost.

4.4. Socio-gender impact

IRWHDS is likely to have the same socio-gender impact as other forms of RWH. It will save time spent in water-fetching (frequently a predominantly female activity). It can be operated under subsidy with a poverty focus or fully-commercially with a richer-household bias. The decision to install a IRWHDS system is likely to be taken outside the user household. In most cases this will also mean less control by female beneficiaries, but in a few cases women may have proportionally more power in the institution (e.g. a church) than in their own homes. Dependency of poor households upon an institution for an essential resource like water may give unhealthy scope for their greater subordination to that institution.

5. CONCLUSIONS AND RECOMMENDATIONS

For lack of much field evidence, these conclusions are over-dominated by theoretical analysis.

1. There is apparently little interest in IRWHDS except as an informal adjunct to conventional institution RWH (via sale of water surplus to institutional requirements). The fraction of a population it could serve is low and economies of scale in construction are offset by complexity of operation.

2. The per-beneficiary capital cost of IRWHDS will generally be less than 1/2 that of domestic RWH giving the same service. However a 'sow and piglets' configuration of IRWHDS (each user has her own cistern) which removes inter-user conflicts, does so with the loss of this strong economy of scale.

3. The management of a seasonally varying water store requires a formal structure, such as a water steward working to rules accepted by the beneficiaries or the issue of water against payment.

4. Achieving high water quality is more important with IRWHDS than with domestic RWH; however it is also easier due to the cleaner roofs used, the lower unit cost of quality assurance measures, the more professional operation and the larger size of cisterns.

5. Only institutions with more than about 6 m^2 of roofing per *institutional resident* will have enough 'surplus' water to justify organising its distribution to outside households. In non-residential institutions the threshold above which exportable water becomes available may fall to 1 m^2 of roofing per institutional member.

6. The sort of operational/commercial body best suited to managing an IRWHDS is often incompatible with the institution whose roofing is to be used. Therefore IRWHDS design must specifically address the institution's interests if it is to succeed.

7. IRWHDS awaits convincing experimentation and demonstration, not so much with respect to technology but with respect to water and financial management.

APPENDIX

Transcript of email mini-conference

(compressed and excluding material not directly concerning IRWHDS)

[First contribution after circulation of introductory paper, dated 28.6.2002]

From: Bisrat Woldemariam, Water Action, Ethiopia <wact@TELECOM.NET.ET>

Concerning the points you have raised on how the systems are run. Please find the following information.

The beneficiaries of the schemes are the students, teachers, priests and the surrounding communities. The school structures serve 1,100 people and the church structures serves 500 more. The project has built up a water committee which charges 0.15 birr for 2 jerry cans of water ($1=8 Ethiopian Birr and 1jerrycan=20 litre). The money is collected by two committees and deposited with the main committee. The committee consists of seven members and sub-committees are located in the school and monastery.

The water in the school compound is rationed as follows

One 40m3 reservoir for teachers,

One 40m3 reservoir for students and

Two-40m3 reservoirs and one 75m3 reservoir for the surrounding communities. The maximum amount of water a household could get is only 4 jerry cans per day.

From the church schemes they give priority for the church service and then to the priests monks, nuns and the surrounding communities.

Please note that the total effective run-off from the school and the churches roof catchments was found 992m3/annum. Though the criteria were set deliberately considering only the water usage i.e. (only for drinking and cooking purpose), there is still a deficit of 2900m3 of water per annum to satisfy the whole requirement of the beneficiaries. However, it is important to bear in mind that the schemes were not intended to supply the whole requirement but they were intended to supplement the usual water shortage occurring during the dry season from ponds, and hence, improve the health condition of the beneficiaries to a certain level

The impact of the programme is that it has released people, particularly women, from walking long distances to collect water and saved working time. It has also improved people's health. Since base line health data is not available, clear indication of changes are not possible. Before 1995 the area faced severe water shortages. The main source of water was the Zeba River and hand dug wells, which dried out in early summer. To fetch water from the river was a 6 hours round trip. To reach it was a difficult task with the path on a cliff edge.

Moreover, it's believed that the scheme have a great impact and role on community awareness towards the use of other alternative water sources other than the traditional ponds and may encourage the beneficiaries to similar replication of roof water harvesting individually or on group basis. In fact till now there is no a replication of institutions RWH schemes constructed by the community itself.

From Brett Martinson, DTU Warwick University **dbm@eng.warwick.ac.uk**

Bisrat raises some interesting points about the use of institutional roofs for domestic supply.

1. The problems of serving a large number of people (in this case 1,100) from a community roof. Even if the roof is over 500m2, the area per person is less than 1m2, at such a small roof area per capita, demand will soon outstrip supply - particularly if it is envisaged the RWH system will be used for bridging dry seasons. Strict rationing will be required. What conflicts can be expected and how would they be resolved?

2. Replication. Institutional rainwater harvesting is often put in to demystify the technology to surrounding households and encourage replication. What experiences do people have of the effectiveness of this. Does the technology take off or does it simply stay in the schoolyard?

From: Rajindra de S Ariyabandu, Sri Lanka **wrsrds@sltnet.lk**

Let me also contribute my two cents worth to the institutional roofs debate. I think we have to take these in the context of the location. It appears from Bisrat's account of institutional roofs in Ethiopia is a success. The main reason is the other water sources are 6 hours away. Even the limited quantity of water per household appears to be properly managed. We in Sri Lanka do not have large scale institutional roof water harvesting at present. However, we did try in schools and at an Agrarian Service Centre in a rural area. The first one had limited success and the second one failed miserably. Both for the same reason, management! The question with institutional roofs is who owns the institution, or for that matter the roof. When there were champions to manage the RW systems in schools it became a success. Champions are those who take special interest in managing the RWH systems. In one school it was the Agricultural teacher who apparently resided in the school quarters. This incidentally, was the same reason why such institutional systems failed in Thailand schools. Who owns and who manages? The system at the Agrarian Service Centre functioned to a limited extent during the time of a very powerful secretary to the Ministry of Agriculture. After he retired the system was neglected, though there was definite benefits from the system. So the point is there is something more that just technical criteria for the success of external interventions in rural areas.

By the way Bisrat, who manages the school reservoirs you have in Ethiopia? How effective is the management?

From: Bisrat, Water Action, Ethiopia

Glad to hear from you again. The school reservoirs are managed by a committee. The committee consists of seven members and sub-committees are located in the school and monastery. Though some training was given for committee members on technical and financial management, the committee expressed a clear need for additional training to water committee members.

From: Kobusingye Annette, ACORD, Uganda

This is my small contribution on the above matter

Institutional Rain Water Harvesting in Uganda is practiced and mainly supported by NGO's, like UNICEF in primary schools and ACORD in some Secondary schools and hospitals. UNICEF's goal is to improve sanitation and hygiene in schools so the water is not accessible to the rest of the community members. The tanks are fairly large but the water is only for the pupils and the teachers.

ACORD has built ferro-cement tanks in schools too but the water is also for the targeted people not the whole community.

Institutional Rainwater harvested from hospitals is used by the patients and the nurses and to a small extent by the neighbouring community which buys this water. Institutional rainwater is sold in order to generate some income to maintain the structure.

From: **Paito Obote, WaterAid, Uganda** **<paitoobote@wateraid.or.ug>**

To add a word to the debate, WaterAid in Uganda supported a small NGO in Kisoro district, southwestern Uganda to provide 'communal' roof catchment systems in an area where rainwater is the only readily available source of potable water supply. Initially every household harvests rainwater on a small scale during the rainy season and in the dry season they all converge to a pond with the animals. Most of the tanks were constructed in private homes and a few were constructed on the few institutional houses like schools, churches, mosques, and administrative buildings. The unique feature here is that all houses are roofed with iron sheet simply because there is no thatching material around, so every home has a source of water. The NGO supported by WaterAid was already running a housing scheme (incorporating rainwater water collection system) for its members; the water from the tanks was sold to members at some fixed price. This arrangement would continue until all the members are covered. The new project focused on rainwater harvesting (i.e. not housing) for the whole community but the management arrangements of rationing and selling water remained as before and it has been very successful. Part of the revenue raised is being re-invested in new rainwater tanks to expand the coverage. The project has been very successful and a number of individuals and even district supported projects are copying the system design and operation.

Three main conditions however led to the success of this project:

1.. The existing demand created by the need and the developed tradition of rainwater harvesting.

2.. The existence of a local NGO that provided the framework for local capacity building.

3.. Maximum utilisation of local resources (materials and human) that ensures sustainability and replication.

I believe that with proper management system based on the user community, communal rainwater systems can be successful and highly beneficial.

From: **Ddamulira Dunstan, ACORD Uganda** **dpdunstan@yahoo.com**

I have been following the on going discussion on institutional rain water harvesting with keen interest. ACORD Mbarara supports supports both domestic and institutional rainwater harvesting initiatives in its Mbarara rural programme area. The experience i have gained in this project prompts me to categorise institutions in three kinds:

1) Educational institutions (like primary schools)

2)Religious or faith based institutions(eg Mosques and churches)

3) General public institutions (like Markets and recreation centres)

Apart from General public institutions, promoting rain water harvesting on other categories and ensuring sustainability is quite easy since quite often these institutions have got well established management structures (eg school management board/committees) that can spearhead rainwater harvesting implementation and later sustainability of the facilities put in place. For general institutions sustainability requires setting up and training a committee to take care of the facility(there was a case in Uganda- Mbizzi Nya market where replacing just a broken tap on a roof tank took a lot of time)

However for religious related institutions a balance has to be taken by any R.W.H promoting agency to support all religious institutions within the vicinity so that one religion doesn't feel left out or marginalised.

Institutions present the advantage of constructing large volume tanks because they have large catchment roofs in one place to fill the tanks(ACORD has supported construction of underground tanks of capacity more than 100,000 litres on primary schools and teacher training colleges in Mbarara district Uganda) But often even these overwhelmed by the demand from the beneficiary communities. This prompts the institution's management to either ration or limit the water to a few close people. (Raising a question whether institutional tanks can be classified as communal or not).

Does any one have anything to comment on roof rainwater harvesting from shared individual roofs? Should one take them to be institutional or not? What are the challenges? [I am just sparking off a discussion!]

From: **"R. de S. Ariyabandu" <wrsrds@sltnet.lk>**

I think we have a definitional issue here. What we have been considering as institutional RWH systems are those which can harvest a large volume of water due the high availability of roof area. They can be community systems in case if they supply to a nearby village community of it can also be individual if the water is used solely by the occupants of the building. Often RWH using government buildings are not community systems though it serves a large number of persons. On the other hand individual household systems in a village community can be termed as community systems project or programme. In this case it is number of individual units serving a larger population. So you see the difference is only how you defined it. On another issue what is the progress of the water policy document? have you posted it already?

From: **"Kyung H. Yoo" <khyoo@eng.auburn.edu>**

It is not a rainwater catchment but a communal water use system from a spring. Each user pays a small token of water uses and the collected funds are used for paying the operator and maintain the system. This is a system Haiti. Management and management is the most important after a system is established. It is important for the community to manage the limited water for critical period of a year and maintain the facility for long term uses. My experience in Brasil deals with individual house RCR (roof catchment of rainwater) systems and communal water supply from an embankment. This project has not been completed but when completed the community will need to develop and enforce water management guidelines.

From: **Terry Thomas, Warwick University, UK** dtu@eng.warwick.ac.uk

Thanks for raising the problem of definition. Straightforward domestic RWH and 'straightforward' institutional RWH (where an institution collects water just for its own members' use) are both widely used. The current e-conference was intended to examine the rather rare circumstance whereby the primary objective is supply to households but the means of doing so is to use an institutional roof. The reasons for this practice might be that the roofs on the houses are unsuitable for RWH or that economies can be made by having 1 large system rather than many small ones. Unfortunately the border between this situation of water being exported from the institution and the situation where the institution itself includes 'houses' - for example teachers' houses - is very arbitrary. Moreover some systems are mixed, with some water going to institution members (such as prisoners or priests and worshippers) and the remainder to neighbouring households.

For the purposes of this discussion I suggest we exclude systems where water only goes to institution 'members' but include ones where water also goes to members' families living nearby.

From: **Chitra Vishwanath, Rainwater Club, Bamgalore, India**
chitravishwanath@vsnl.com

Happened to visit a place called MANAPAD on the east coast of South India near Kanyakumari (the southern most point of mainland India). Salinity ingress to up to 5 km has rendered most shallow wells and borewells unfit to drink from for many villages on the coast. An organisation called ANAWIM has set up 5 ferrocement tanks of capacity 10000 litres each in 5 villages. The one village that I visited had a temple roof as its catchment (about 2500 square feet), actually half a temple roof. The tap to the ferrocement rainwater tank had a lock and one of the ladies of the community kept the key and rationed out water (1 pot per family per day roughly 10 litres). This apparently was good enough for drinking water for the family. The overall system seemed to furnish enough drinking water for about 40 households for 6 months. The ferrocement tank was constructed using a steel mould and was exceedingly well done. The households were very happy with the quality of the water. Unfortunately the other half of the temple roof remains unutilised and funds have not been raised to increase coverage.

From: **Ddamulira dunstan, ACORD Uganda**

Thanks for raising those interesting observations. However the most interesting one is that in most cases people using water from institutions communally tend to use up all the water without restrictions making the systems run out of water in a very short time. My conscience is telling me that perhaps they do use up the water in the systems unknowingly (unconsciously). You see most system designs never incorporate any kind of component permitting/allowing easy monitoring of water remaining in the tanks. Perhaps if there was any easy way people could easily tell how much water is remaining in the systems then they would try to use it sparingly so as to pass them throughout the dry period until the next rains.

Isn't it high time water professional started seriously thinking about some form of WATER LEVEL MONITORING METER? It can be in form of a simple wooden stick calibrated in a simple area specific measurement (e.g. in Uganda Jerricans), floating on the water and extending out through the cover or breather in the tank roof so that can easily be observed by the users as it goes up and down following the level of the water. Perhaps this combined with other innovations can help people monitor their water usage/consumption and hence ration it depending on the availability of rains to re-fill. The problem of gutter span has also been faced our end. Fortunately for our case the tank was underground and we had to divide the span into two so that the first section near the tank is on top of the roof and the second is connected by a vertical down pipe to a ground level horizontal pipe which directs the water into the tank. Otherwise I will come back to you about the deflector plates.

From: Brian Skinner, WEDC, Loughborough University <B.H.Skinner@lboro.ac.uk>

I agree that a way of monitoring the water level can be useful, particularly for a household system. However for a communal system I don't think that your ideas will usually work. I feel that unfortunately people tend to be rather selfish and do not have a good level of self-control. Where there is an appreciable walk to the alternative source of water people will tend to take as much as they can from the nearer rainwater tank if they can do so with without other members of the community knowing! If other community members think that someone else is 'cheating' they may well join in, thinking why should only they have the easy life!

I feel that people will only restrict the amount that they collect in well-disciplined communities, that use appropriate enforcement of punishments for drawing more than allowed. Has anyone on the list experience of communities successfully using a communal tank?

[Last contribution dated 10th June 2002]

From: "Kyung H. Yoo" <khyoo@eng.auburn.edu>

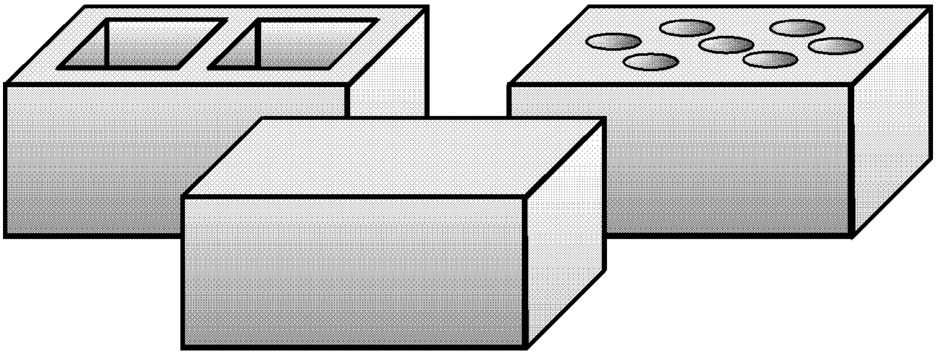
I agree. One possibility would be charging the water at low but high enough rate and use the collected funds to hire a person to police and operate the system. I don't know how it would work but it could be an alternative to getting whole community thirsty during the critical dry season.

delling water sources for rural communities (SimTanka). Ajit Foundation.

Development Technology Unit
School of Engineering
University of Warwick
Coventry CV4 7AL UK
Tel (Office) +44/0 1203 522339
Director 523122, Fax 418922
Email dtu@eng.warwick.ac.uk

Web site at http://www.eng.warwick.ac.uk/DTU/

Stabilised Soil Research Progress Report SSRPR2



How does cement stabilisation work?

Author: Mr D E Montgomery
October 1998

E-mail: d.e.montgomery@warwick.ac.uk

These reports cover 'work in progress' by research students in the Development Technology Unit (DTU) of the School of Engineering at Warwick University. Their primary purpose is internal - a format for recording ideas and data in a way that allows them to be better discussed before their incorporation into theses, DTU Working Papers or external publications. However they also have a secondary purpose, that of facilitating the sharing of our research with other innovators in the field of building with stabilised soil. Each report, after some initial internal discussion and refining will be posted as a title and synopsis on the DTU web pages (home page= http://www.eng.warwick.ac.uk/dtu). Full copies can be obtained from the respective named authors.

A dedication to someone special

Sometimes at the beginning of a publication one finds a dedication to a certain person or member of the family who has been an influence in the author's life either in general or specifically in generating the work in question. There is one person in my life that immediately springs to mind who is worthy of such a dedication. Furthermore, my experience with this person is not unique as millions of others have found him to be a great inspiration, comfort, guide and friend. "What's his name?" you may be asking yourself and, "Why haven't I heard of this incredibly influential person". The sad thing is that you probably have, but you have never accepted him as such or welcomed him into your heart and life. Well, now you have an opportunity to do just that. Please read on.

The man's name is Jesus and although he was born nearly 2000 years ago his testimony still remains and his power to save is just as great. "Save from what?" you may ask, sin and the consequences thereof, or more specifically, your sins and the consequences you face when you die. As humans we demand justice to be done, and justice will be done, but on a perfect scale and to a perfect standard. That leaves us all falling short and without hope when we come face to face with a holy God. But, God in his great love towards us send his only begotten Son into the world that the world through him might be saved. Jesus Christ died for you so that you would not have to be punished for what you have done wrong. You can be spared eternal punishment in hell and enjoy love and peace in the presence of God forever. Today the choice is yours. Reject God's free gift of love at your peril, accept it and who knows you too may have the joy of writing a dedication such as this someday. Please ponder the verses below and make your choice carefully, it will be the most important decision you ever make.

"*For by grace are ye saved through faith; and that not of yourselves: it is the gift of God: not of works, lest any man should boast.*" Ephesians 2:8,9.

"*For God so loved the world, that he gave his only begotten Son, that whosoever believeth in him should not perish, but have everlasting life.*" John 3:16.

"*For whosoever shall call upon the name of the Lord shall be saved.*" Romans 10:13

"*He that believeth on him is not condemned: be he that believeth not is condemned already, because he hath not believed in the name of the only begotten Son of God.*" John 3:18.

"*Jesus saith unto him, I am the way, the truth, and the life: no man commeth unto the Father, but by me.*" John 14:6.

Abstract

After a brief study of some relevant texts documenting the production, characteristics and use of Portland cement a better understanding of its cementitous qualities has been gained. The bonding of cement is caused by the hydration of the cement particles which grow into crystals that can interlock with one another giving a high compressive strength.

In order to achieve a successful bond the cement particles need to coat most of the material particles so that upon hydration a crystalline structure is created throughout the mixture of particles. Particle intimacy is important to ensure a good number of cementitic bonds between adjoining particles and this can be helped by mixing the cement into a mixture of particles with a good size distribution. The water in the mixture needs to be monitored to guarantee sufficient hydration of the cement and also to ensure adequate workability of the mix. Too much water will leave voids in the mixture after the water has evaporated off and will reduce the final set strength of the material.

The limitations to cement besides the careful control of materials and moisture are that cement requires time to fully cure and that it is susceptible to chemical attack. Never-the-less it is a highly suitable method of stabilisation and can easily be applied to stabilise a moderate variety of different soils for use in making building materials.

Nomenclature

Aggregate: Pieces of crushed stone, gravel, etc. used in making concrete.

Brick: An object usually of fired clay used in construction, usually of rectangular shape, whose largest dimension does not exceed 300mm.

Block: A larger type of brick not necessarily made of fired clay, but stabilised in some way, sometimes with central cores removed to reduce the weight.

Cement: Ordinary Portland Cement (OPC), a finely ground clinker which sets hard after mixing with water.

Clay: The finest of the particles found in soil, usually of less than 0.002mm in size and possesses significant cohesive properties.

Clinker: A slag formed when clay and lime a burnt in a furnace together.

Concrete: The finished form of a mixture of cement, sand, aggregate and water.

Dynamic Compaction: A process that densifies soil by applying a series of impact blows to it.

Gravel: A mixture of rock particles ranging from 2mm to 60 mm in diameter.

Green: Describing the state of material containing cement and water before it reaches the critical time, after which further plastic deformation hinders the final set strength.

Gypsum: A hydrated form of calcium sulphate.

Mortar: A mixture of sand, cement and water.

Sand: A mixture of rock particles ranging from 0.06mm to 2 mm in diameter.

Sandcrete (Cured Mortar): The finished form of a mixture of cement, sand and water.

Sharp Sand: Describes the angular nature of sand particles that are very good for making concrete or mortar.

Silt: Moderately fine particles of rock from 0.002mm to 0.06mm in size.

Slaked Lime (Lime): Quicklime (calcium oxide obtained by burning limestone), that has been mixed with water creating calcium hydroxide which has further setting qualities.

Soil: Material found on the surface of the earth not bigger than 20mm in size, not including rocks and boulders and predominantly non-organic. If soil is to be used for building material it must not contain any organic material and it can be a natural selection of particles or a mixture of different soils to attain a more suitable particle distribution.

Soil-cement: Similar to mortar, but prepared from soil with a wider particle distribution.

Stabilised soil: Soil which has been stabilised (treated to improve structural characteristics) by using one or more of the following stabilisation techniques: mechanical, chemical and physical.

Table of Contents

1. Introduction

Cement is to be the primary means of chemically stabilising the soil samples during this research project. Consequently a good understanding of how cement works and how it forms cemetitious bonds with other particles would be most desirable. This report will briefly outline what cement is made of, and how it is produced, but it will spend more time detailing the bonding and curing processes in concrete. During the report it will also establish the various requirements that cement has in being able to perform properly as a stabilising medium. Finally, these theories will be applied to the stabilising of soil.

As a stabilising material cement is well researched, well understood and its properties clearly defined. Portland cement is readily available in most urban areas, and usually available in semi-urban areas, as it is one of the major components for any building construction. Earlier studies have shown that cement is a suitable stabiliser for use with soil in the production of soil-cement blocks, (International Labour Office, 1987), (p. 38). As this is established and recognised technology it provides a suitable basis for further research into the production of better soil-cement materials. Further studies hope to minimise the quantity of cement required to form soil-cement structures.

For the purposes of this report and further study it is assumed that ordinary Portland cement (OPC) is readily available in bags on location. A significant cost may have been incurred in getting the cement to where it is needed, but this report is not intended to analyse the cost effectiveness of cement over other brick stabilising methods. Instead it is to concentrate on modifying and improving the existing cement stabilising of soil, with perhaps a breakthrough in the entire block production routine.

2. Some facts about cement.

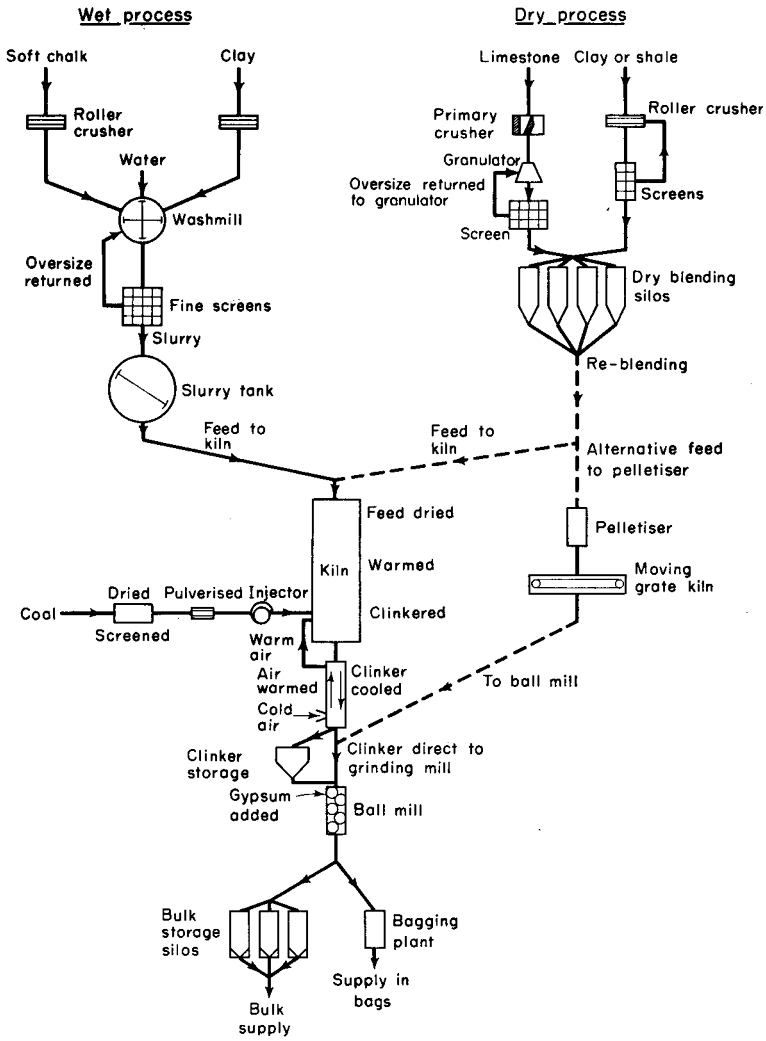
This section will concentrate on establishing the properties and composition of cement. This knowledge will provide a simple foundation for understanding the way that cement works. It will not describe in too much detail the characteristics or production of cement, as this has already been previously established to a sufficient level, (Akroyd, 1962), (p. 46-54), (United Nations, 1972).

2.1 Chemical composition and production

Cement can simply be described as being a mixture of lime and clay which is heated to about 1,500°C, and the resulting clinker has gypsum added and the sum is then ground to very fine powder. An extract from (Akroyd, 1962), (p. 50) contains sufficient detail of the chemical composition of cement itself, featured below.

	Percent (%)	Average(%)
Lime (CaO)	59 -67	64
Silica (SiO_2)	17 -25	21
Alumina ($A1_20_3$)	3 - 9	7
Iron oxide (Fe_2O_3)	0.5-6	3
Magnesia (MgO)	0.1-4	2
Sulphur trioxide ($S0_3$)	1 - 3	2
Sodium potash	0.5-1.3	1

Below is a diagram showing both the Wet and Dry cement manufacturing processes as extracted from (Akroyd, 1962), (p. 48). There has been a move from the former to the latter in recent years, as the dry process requires less energy per unit of cement output.



2.2 Relative cost to other materials

The price per kilogram of cement will vary greatly depending on the distribution network and the proximity to the cement processing plant. Cement can usually be considered to be one of the more expensive materials necessary for building construction. In the field of low tech, low cost soil brick housing, it is crucial that the total cost of the cement as a proportion of the entire structure is kept as low as possible. One would ultimately like to minimise the cement content and maximise the strength and life of the structure. Through a variety of procedures the amount of cement necessary can be reduced and these may be investigated in more depth separately later on in the research. For the moment, the author is taking previous research to suggest that a nominal 5% cement is sufficient for good stabilising of soil blocks.

2.3 Distribution problem

In the vast majority of cases OPC will not be made on site, consequently it will have to be delivered. Once cement has been manufactured, it is generally available in two forms. It can be purchased in a bulk form from a silo, or it can come in bags of 50 kg each. (A new bag size of 25kg is becoming popular in some countries.) Cement that is purchased from a silo is mixed and delivered using cement trucks. These will usually ensure that the cement arrives in good condition, ready for immediate use. However, if cement is purchased in bags, there is no guarantee what state the cement will arrive in. Cement is usually distributed in a multi-layer paper bag that only gives it a small degree of protection. If bagged cement has come a long distance and has been exposed to the elements for any period of time it is highly likely that the cement will have absorbed some moisture and will have started to set. This partial setting of the bag of cement does not render the entire bag useless but it does hinder the use of what is still OK. The good cement has to be sifted out and the remaining lumps can be broken up to make a lower quality cement.

3. Making concrete

In this section the main focus will be on the existing procedures for making concrete. There are established techniques for achieving different grades of concrete, each of which performs a specific task. The analysis of these different grades and how exactly they are all generated is not of great relevance here, but understanding the underlying principles of concrete manufacture will be helpful in later applying similar methods to stabilising soil. Some specialised grades of concrete requiring cements other than OPC, but for the ensuing discussion assumes that OPC has been selected.

3.1 Material selection and requirements

Cement can be mixed with virtually any size and shape of sand or aggregate, depending on the purpose of the concrete that is to be made. Particles are usually graded according to physical size ranging from clay particles (< 0.002 mm) up to boulders (> 200 mm). Particles smaller than 0.02 mm are considered to consist of silts and clays, too much of which will hinder the cementitous process. Particles larger than 60 mm are only usually used in large continuous structures such as dams etc. Cement is mixed in with these other particles and when water is added that starts a chemical reaction within the cement particles that grow to form an interlocking matrix. To aid the particle intimacy, a mixture of aggregate grades are mixed together giving a spectrum of different particle sizes that reduces the size of air voids in the material. This further enhances the final concrete block strength.

The concrete composition depends on the job that is being done. Each concrete mix should be designed for the purpose for which it is intended, (for example a concrete mix for a floor cast will be very different to a mortar mix for brick laying.). This requires a selection of grades of sands and aggregates to be mixed with specified quantities of cement and water. Additional ingredients can give the concrete special properties if necessary and these need to be determined and added in the correct quantity. These may affect one or more of the following; workability, strength, density, thermal characteristics, elastic modulus, durability and speed of setting.

The two characteristics of sands and aggregates that affect their performance when mixed with cement are the particle’s shape and surface texture. The shape affects the workability of the cement during mixing and placement and the surface texture affects the bond between the particles and the cement. Very large angular particles decrease the workability of the mix, whilst smaller more rounded particles do the opposite. Angular shaped particles are generally formed by crushing larger particles down to size. More rounded particles can exist naturally as their shape has been formed due to slow abrasive action between particles in the environment. Angular particles usually have a lower workability but achieve a higher strength since angularity is usually accompanied by surface roughness. Crushing or selection of angular aggregates is only necessary when a very high compressive strength (over 50MPa) is necessary, (Teychenne et al., 1988), (p. 7). For the purposes of this project a compressive strength of that order will not be necessary.

3.2 Mixing quantities and preparation

The approximate quantities of cement, sand and gravel are often found quoted in a ratio of their respective volumes, e.g.: 1:2:4; one part cement, two parts sand and four parts gravel. There are standards for mixing cement so that a required compressive strength can be reached in a given time provided that the type of aggregate and the free-water to cement ratio is known, (Teychenne et al., 1988), (p. 10).

The free-water content is calculated from the slump or Vebe time test. In simple terms the higher the free-water content the greater the amount of slump will occur over a given period. Ideally the amount of water used in the mix should also be monitored to be sufficient to hydrate all the cement and not more than is necessary to fill all the voids present in the material as further moisture drives the particles further apart. Unfortunately this yields a highly unworkable mixture and more water has to be added to form the mixture into the desired shape. Excess free-water increases the workability of the mix but will be detrimental to the final strength of the concrete. The minimum water/cement volume ratio is between 0.22 and 0.25 (Akroyd, 1962), (p. 13) for adequate cement hydration, but this is generally increased to the order of between 0.5 and 0.8 for normal mixes, (Lea, 1970), (p. 392).

The aggregates that are to be used in the concrete mix usually need to be washed before mixing with the sand and cement. The washing process removes fine particles on the surface of the aggregate allowing the cement to achieve a better bond. In the case of purchased aggregate this is usually done for you, but if excavation is part of the process then washing should be included in the preparation of the aggregate particles before mixing with cement.

3.3 The effect of compaction

It has been shown that if the particles in a cement mixture are in some way brought closer together, the greater particle intimacy results in a higher final strength. Achieving this closer particle intimacy can be done in a number of ways. We have already noted that excess water in the cement mix will drive particles further apart and will cause a loss in strength. So keeping the free-water content to a minimum is a good way of ensuring closeness between particles.

Another method is to use a vibrator, that effectively shakes the cement mixture and helps to drive any air pockets to the surface. This is usually done in concrete casts as the vibrator can be inserted into the mix and the vibrating action will permeate throughout the mix. The size and number of vibrators will depend on the size of the cast. Obviously this technique cannot be used with very small casts (where it is normal to externally vibrate the whole mould instead) and there are some other drawbacks to the method. A higher free-water content is necessary for gravity-cast concrete in order to permit the cement mix to flow into all parts of the mould. Properly mixed concrete will have the different grades of aggregate well dispersed throughout the mixture. Using a vibrator in a cast with a high free-water content can cause the larger particles of the mix to sink to the bottom, resulting in a non-uniform distribution of particles.

As yet the author has not found little information (other than previous work at Warwick) on the compacting of a green mix using a moving mass, such as a hammer or weight. This process has been hinted at in (Akroyd, 1962), (p. 196), but no details were given, as the compaction process has been replaced by internal and external vibrators to reduce the labour costs of manual compaction. It is precisely this manual compaction that is of interest to this project as the stabilised soil samples are to be compacted and hence the cement also is compacted. What we can learn from the references Akroyd and Gooding, is that compacted concrete has stronger characteristics than un-compacted concrete.

4. Curing process

By way of a simple illustration of the adhesive qualities of cement we can look at a much simpler example of Papier Mâché. Papier Mâché consists of a flour suspension in water into which paper strips can be immersed and then laid over a former to create a hard shell when it dries. Neither flour nor water have adhesive properties on their own, but when mixed and allowed to dry with a close particle particle intimacy a remarkably strong structure can be created. The flour particles become embedded into the pieces of paper, because the paper absorbs some of the water as well, and once the water is displaced by evaporation, strong bonds between the pieces of paper remain.

The analogy between Papier Mâché and cement breaks down when you add water to the structure again. Cement will retain much of its strength (e.g. 50%) whilst Papier Mâché will break down again and become weak. With Papier Mâché the bonds stay strong so long as moisture is absent, the cohesion is caused by inter-particle intimacy and that breaks down when water is added, as the particles are driven apart by the presence of water coating the surface of the particles. Cement on the other hand undergoes a chemical reaction that remains strong even after moisture is re-applied. Exactly what happens as cement bonds with adjoining particles is what this chapter will endeavour to describe.

4.1 Inter-particle bonds, Why are they formed?

Originally they were two popular theories about how OPC worked. The older of the two was a crystalline theory of Le Chatelier which dates back to 1882. This theory stated that the hardening is due to the locking together of an inter-growth of crystals hence giving the crystalline theory. The alternative theory came later in 1893 proposed by Michaelis which was the gel theory. He suggested a non-reversible gel is formed in saturated solution which surrounds the cemented particles. As the gel coagulates the cement sets. These two theories were then integrated into a combined gel/crystalline theory that describe the different stages of curing, (Lea, 1970), (p. 253-260).

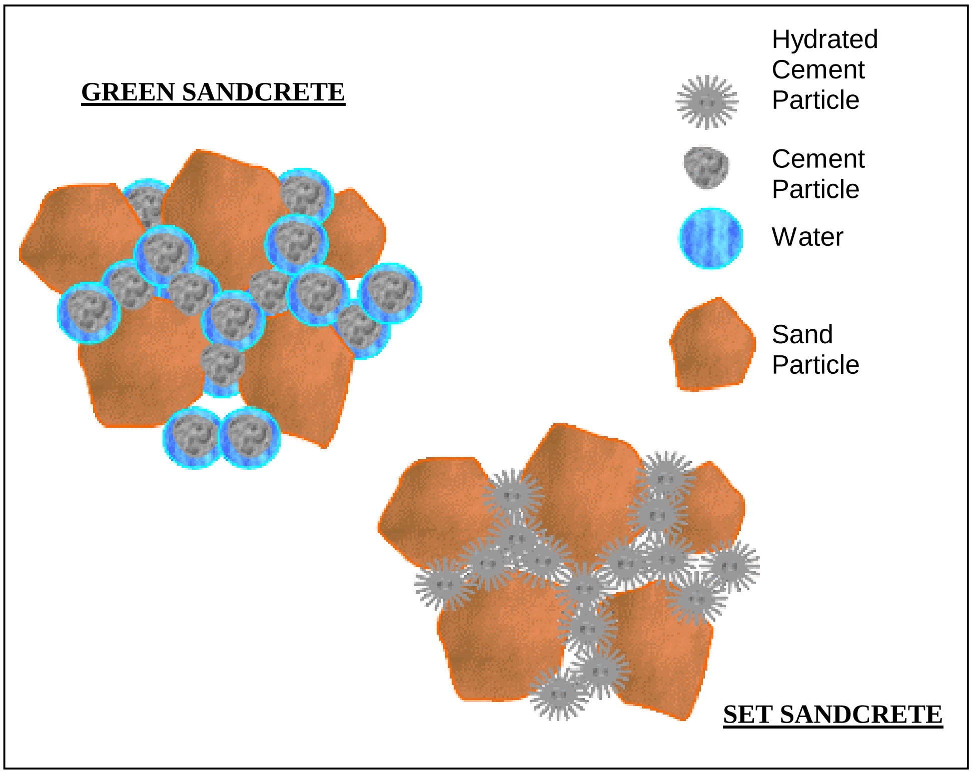
Once cement, sand, aggregate and water are mixed thoroughly the mixture gains a certain cohesion with itself. This cohesion greatly depends on the amount of free-water present as an excess of water will lead to a more runny consistency. Assuming the correct amount of water is applied to ensure complete hydration of the cement, each cement particle will be coated in water and this turns into a gel-like film. These gel-coated particles of cement are themselves coated all over the sand and aggregate particles throughout the mixing process. At this stage the cement is still workable and has not begun to set. The reaction between the water and the cement begins a crystallisation process and small single crystals begin to form.

After the “critical time” has passed, these single crystals grow into one another and a huge crystalline network begins to form. The critical time is loosely defined as the time after which further working of the cement is detrimental to the final set strength. Adjoining crystals do not chemically join, but are attracted to one another by Van de Waal forces. The small single crystals begin to inter-link to form a network of interlocking crystals throughout the mixture. If the mixture has been properly graded to include a good range of particles sizes, and these have been thoroughly mixed together with the cement, the crystalline structure will be surrounding each of the particles interlocking them one to another.

There may still be moisture present in the mixture after the crystalline network has been formed and this will slowly be evaporated to the atmosphere as the water particles are drawn to the surface by capillary action. During this drying out phase the concrete will experience a small amount of shrinkage depending on the excess of free-water present. Part of the attraction of employing blocks rather than mass concrete walling is that shrinkage takes place where, due to lack of hard constraints, cracking is unlikely to ensue. This drying out process can take some time to finish completely, but for general purposes it can be assumed that the concrete has virtually reached its’ final strength after 28 days. The final result is a chemically bonded solid mass with a very high compressive strength.

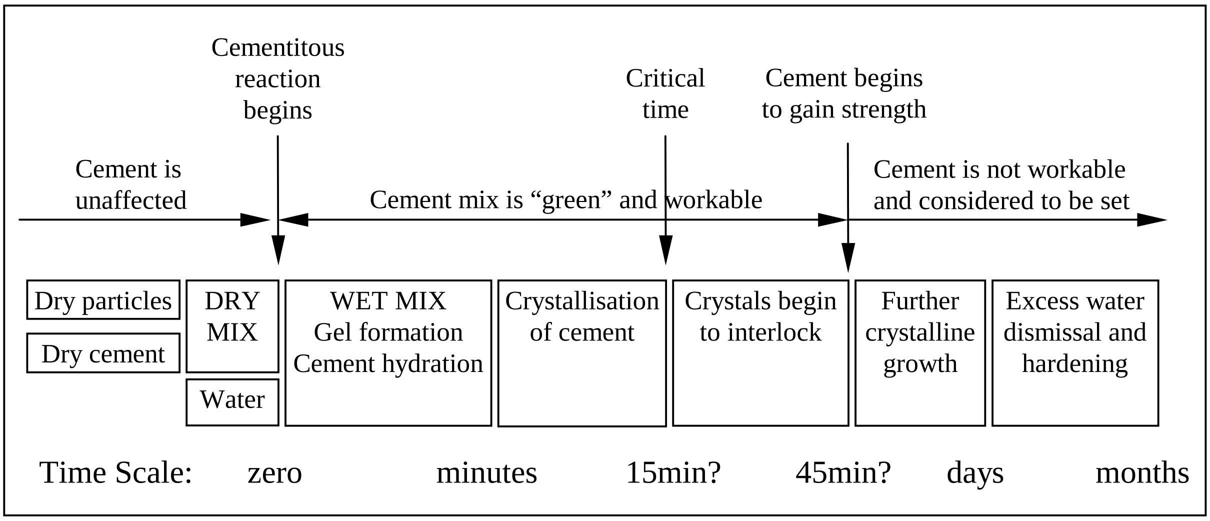
The difference between the strength available in tension and compression is suggested to be that in tension the particles are held together with relatively weak Van der Waal forces. However, in order to separate the particles in compression the forces are acting against the much stronger hydrogen bonds in the crystals that are heavily interlocked with one another. It has been suggested that the crystals do not actually bond with one another, but instead nest together giving the concrete a more mechanical bond than a chemical one. Compression intensifies this bond whereas tension opens up cracks that in turn generate stress concentrations at their ends. The final tensile strength of concrete is typically only 10% of the compressive strength and consequently if loaded in this fashion it must be reinforced with steel. For the purpose of building walls the load is almost always compressive and so this reinforcing with steel is not going to be considered further as it will be outside of the scope of this project.

The diagram below aims to help visualise the bonding process between the cement particles and the sand particles which are in turn bonded to the larger lumps of aggregate. This diagram is not to scale nor is any of the chemical changes that occur noted in diagrammatic form. It merely illustrates the particle arrangement and the presence of moisture coating the cement particles that in turn disappear leaving the strong cementitic bonds behind.



4.2 Wet strength and curing times or cycles

As mentioned above the cement mixture can still be worked up until the critical time is reached without causing a loss in the final set strength. After this point the crystallisation process begins to give the mixture a more rigid nature. The mixture has not fully set at this point, but it does have some internal cohesion as bonds are being formed. The strength of the mixture early on in the curing process is called the "wet strength" or the "green strength". Certain levels of green strength will permit the mixture to be handled in a solid form, but it will still be very fragile. Below is a time-line diagram to illustrate the setting of cement.



In the example of making blocks from concrete the mixture is placed into a mould and after a set period of time the formed block can be removed and put to one side so that the mould can be reused. The length of time that the block must be left in the mould will depend on the wet strength that is required for ejection from the mould and subsequent handling. The time to reach this point will vary depending on the speed of the cement curing and beginning the crystallisation process. This will depend on the amount of cement present, the final block density and the free-water content.

In order to maximise the green strength of the mixture one needs to ensure that the free-water content is as small as possible and to leave the mixture untouched for as long as possible to permit the cementitous action to bond the particles together. The exact length of time that is necessary to achieve this may be discovered by trial and error, but as mixture quantities and handling techniques may vary "as long as possible" may be a good initial estimate. Setting can take place in as little as 45 minutes, but useful hardening will take much longer, (Stulz & Mukerji, 1993), (p. 63). The green strength of mixed samples can be tested to give a more accurate answer, using both destructive, i.e. compressive tests, or non-destructive, i.e. scratch tests, to determine the approximate green strength of the formed mixture.

It may be possible to put the mixture through a series of curing cycles to achieve a greater overall strength over a longer period of time. The initial curing time may only be sufficient to manipulate the formed mixture and place it in its final position, in a wall for example. Further cycles of wetting and drying could then encourage any unhydrated cement particles to become hydrated and cure within the finished product. This is of particular interest where the cement content, and therefore the water content, is very low. This low water content may be able to hydrate all the cement, if given time to do so, but evaporation takes over and takes away the moisture before the cement has had a chance to hydrate properly.

The amount of hydration that is necessary to achieve the desired strength is another point in question. Tests done by Grun (Lea, 1970), (p. 268) have been done that illustrate that even after a cement sample has fully cured, it can be broken up again and rehydrated and encouraged to cure again. This evidence strongly suggests that all the cement is not hydrated in the first setting period. Therefore, in order to achieve a desired strength, in the long term, complete hydration could theoretically occur over a period of time ranging from weeks to even years depending on the circumstances.

As concrete is porous when set it would still be able to receive moisture into the surface and permit further hydration of the unhydrated cement particles. What increase in strength this would give is not clear as the porosity itself is a weakening factor due to the voids present between particles. These voids provide no structural strength and the re-hydration will only help the overall strength if has the potential of filling some of these voids with cementitic crystals. Although this is perfectly plausible, how effective it is in practice is unclear.

4.3 Moisture dismissal and shrinkage

The moisture content of a cement mixture is of great importance, primarily because too little water will cause insufficient cement hydration, and too much water will reduce the final set strength. Keeping the right moisture content during the mixing and forming stage would therefore be quite important to monitor and control, if possible. This is especially true in hot climates where the moisture content will drop rapidly if left in the environment unmonitored and uncontrolled.

In order to ensure that the cement goes through a complete hydration process, and maximising final strength, the water content needs to be minimised, whilst also preventing the existing moisture from escaping. In practice this has been done in two ways. Once the initial cement has set the formed mixture can be submersed in water during the hardening process. This guarantees that there is sufficient water present for the cement to hydrate, but since the immersion occurs after initial setting, the extra water present will not affect particle intimacy and jeopardise the final strength of the formed mixture. The other, simpler, method is to keep the formed mixture in an environment with a 100% humidity. This prevents water within the block from escaping to the surface too quickly as the surface evaporation will be almost non-existent in an environment with a 100% humidity. In practice this too can be difficult and so a compromise of sprinkling water over the formed mixture repeatedly during the hardening process helps to minimise internal moisture from evaporating too quickly.

We have already discussed the movement of water through the mixture during the curing process by mechanisms of evaporation and capillary action. What now needs looking at is the effect that this moisture movement has on the finished article. By inspection the limiting factors for shrinkage are the amount of excess water present and consequently the voids that it leaves behind, and the overall density of the mixture prior to curing.

To minimise shrinkage one must minimise the potential space between particles in the mixture. Clearly the sand and aggregate particles themselves do not shrink, and similarly the cement and formed cementitic crystals are not prone to shrinkage. This leaves the physical gaps between adjoining particles and the gaps left by excess water when it has evaporated off, being the primary cause for potential shrinkage.

The problems associated with shrinkage are mainly to do with uneven shrinkage and different relative amounts of shrinkage. If every mixture shrunk in exactly the same way and by the same amount each time, then it could be accounted for and there would be no problem. In practice the shrinkage is often uneven, due to insufficient mixing or uneven drying. The desired form into which the cement mix was placed will not be the same as what is finally achieved after the hardening process is finished, and this may not completely finish for many months.

The amount of potential shrinkage is not insignificant either. Gessner discovered that using a pure OPC-water mixture a volumetric change of over 6% could be noted in the 28 day curing time, (Lea, 1970), (p. 269). The cement samples that were used had quite a high water content using three parts cement to one part water. Previous suggestions were that the ratio should be closer to four to one or four and a half to one instead of the three to one that Gessner used. This could partially account for the high shrinkage, and a better cement to water ratio may yield much better shrinkage results, never-the-less, it does illustrate the significance of potential shrinkage that may occur during curing. As we will see later, this potential shrinkage is a considerable nuisance when trying to build structures with many slightly different formed cement mixtures.

4.4 Strength testing

The strength of a concrete structure is limited by one of two factors. Simply speaking, either the bond between the cement and the aggregate fails (cement matrix failure), or the aggregate itself fails and shears along existing fault lines within the material. Usually the former occurs because the aggregate has a higher crushing strength than cement, (Akroyd, 1962), (p. 85). A stronger bond between the cement and aggregate can be achieved if the aggregate is angular and clean, which has already been recommended earlier in this report.

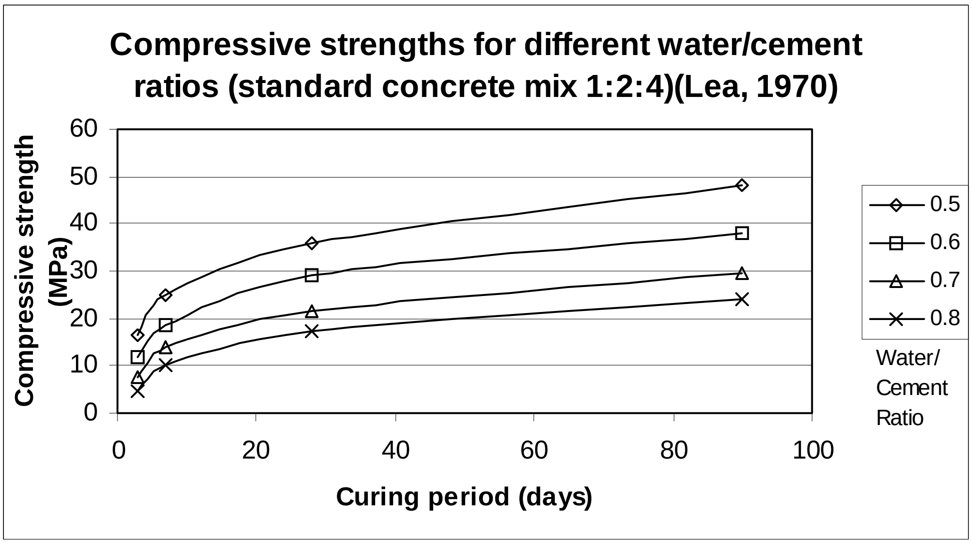
The final set strength of a concrete mix is directly proportional to the water/cement ratio, provided that the workable concrete is compacted so that it contain less than 1% by volume of air voids. This relationship can be expressed as $S = A/B^x$, where S is the compressive strength, x is the water/cement ratio, and A and B are constants determined by the materials used and the conditions of the test. The table below, based on (Lea, 1970), (p. 392), shows how the water/cement ratio affects the strength of the concrete after different periods of curing.

The influence of water content on the strength of a 1:2:4 concrete based on Lea, Table 59.

TABLE 59

Water/cement ratio	Compressive strength (MPa) 3 days	7 days	28 days	90 days
0.5	16.55	24.82	35.85	48.27
0.6	11.72	18.62	28.96	37.92
0.7	7.58	13.79	21.37	29.65
0.8	4.83	10.34	17.24	24.13

Or viewed graphically:



4.5 Longevity, environmental attack

Well-made concrete using quality ingredients is usually considered to be a building material of a very high standard. Such concrete has a very high resistance to environmental attack of any kind, apart from major natural disasters of course. Consequently as a building material it can in theory last for a very long period of time. There are of course certain chemicals that will cause slow deterioration of finished concrete, but most of these can be ignored as their occurrence would be so improbable in wall construction that they are not worth considering. (Akroyd, 1962), (p. 247-250), gives a list of such chemicals; Carbon dioxide, Chloride, Chlorine, Chromium salts, Detergents, Fatty oils, Formaldehyde, Fruit juices and sugars, Gypsum, Hydrogen sulphide, Inorganic acids, Lactic acid, Lead, Oils, Organic acids, Salt for de-icing and Water.

Two of the above chemicals stand out as being strange in a list of things harmful to concrete; gypsum and water. Gypsum is an additive used in making cement, but it is also a sulphate and all sulphates attack concrete, so it must be included in the list. Water itself is not harmful to cement, but water often carries with it harmful salts and sulphates and these are what cause the problem. In extremes of temperature change, where frost and freezing occur water can pose a problem if the porosity of the concrete is high. If water is permitted to penetrate the surface of the concrete and this is subsequently frozen it will expand can cause damage to the concrete. This damage may occur superficially as spalling or it may cause deep internal cracking that is much more severe. These cases are worth considering generally, but for the purposes of this project such extremes are not going to be considered.

5. Application of cement to stabilise soil

By now we have a better understanding of the way that cement bonds with itself and other particles in making concrete. We also know some of the important guidelines that need to be followed when making successful mixes of concrete. Furthermore, many of these guidelines can be followed when applying the same principles to mixing cement with soil as this chapter will set about to illustrate.

5.1 Basic requirements of soil

According to the ideal specifications given by the United Nations, in “Soil-cement: Its use in Building, (1964)”, as quoted by (Gooding, 1993), (p. 263), the best soil composition for soil-cement is as follows; 75% sand, 25% silt and clay, of which more than 10% is clay. This composition will yield a sandcrete product if mixed with cement and will exhibit good structural characteristics. Unfortunately, soil with these exact characteristics will not be found easily near every potential building site and so one of two things must be done. Either the soil is tested and the required parts added to make the ideal soil, or a compromise is made and a slightly higher percentage of cement is used to ensure a satisfactory outcome whatever the type of soil is used.

Unfortunately, there is an underlying problem with randomly mixing cement with any type of soil, and it is to do with the clay fraction of the soil. Clay consists of the finest particles in the soil and can, in same way that cement does, coat the other particles when mixed with water and cause a significant cohesion after the mixture is dried. Indeed this is how the majority of earth bricks are made today. Clayey soil is mixed with water, formed in moulds, ejected and left to dry in the sun. The clay in the soil has to be protected from getting wet again, as moisture will drive the clay particles apart and cause considerable material breakdown. To do this, these formed bricks can be fired, or be placed into a structure and protected from the elements with some form of paint or render, an effective damp-course and an effective roof.

Clay and cement will work against one another if the quantities are not carefully monitored. Too much clay will result in the cement not coating all the particles sufficiently and subsequent wetting will cause expansion of the formed mixture breaking apart the cement crystals and causing breakdown. [Remember,] Cement is not strong in tension and the expansion of the clay particles cause internal expansion working against the weaker of the cementitous bonds. Also because clay is so very small (0.002 - 0mm) it is difficult for the cement to successfully coat the clay particles. Therefore, let us assume that a high composition of clay in a soil that is to be stabilised with a very small quantity of cement, makes it unacceptable.

According to (Norton, 1997), (p. 16), a suitable particle size distribution for building with earth is:

Sand/fine gravel	40 - 75%
Silt	10 - 30%
Clay	15 - 30%

The values may of course need to be more closely defined for soil-cement, and it may be the case that the clay fraction is the critical quantity. Clays can be removed from soils by washlines. However washlines may be impractical in the field because of the large amounts of water necessary and another source of soil may have to be found. Sieving the soil can also separate out the larger grains but this is also time consuming and labour intensive. Soil sieving may only be practical for removing large particles such as coarse gravel, (over 20 mm in size).

Particles within the soil will generally be rounded due to the natural environment that the soil is being excavated from. Secondary crushing techniques are assumed not to be used in developing countries because of the high cost of the complex and heavy duty machinery required to crush large aggregate into smaller angular particles. The extraction and breaking up of soil clumps will be enough of a labour intensive exercise, without having to further crush rocks up into smaller angular pieces. For the purposes of soil-cement building materials any particles over 20 mm are considered too large and should be discarded. Thus we will normally be working with soils having rounded particles in the size range clay to fine gravel.

5.2 Particle, particle interaction/intimacy

The main source of soil will be to dig it out of the ground. It will therefore be removed in dense clumps which will have to be broken up and have the cement thoroughly mixed into it. This process of breaking up the clumps will lower the overall density of the soil and reduce the particle intimacy. This will need to be reversed after the cement is mixed into the soil to ensure maximum strength and minimum porosity.

We have described already the necessity of keeping the particles closer together in the previous chapter, and also the consistency of particle intimacy throughout the mix. Keeping particles close together reduces the air voids present in a soil mixture and will generate two distinct benefits. Firstly the closer particle interaction will help to ensure good bonds between the cement and the particles, and secondly the porosity of the mixture will decrease leading to reduced levels of water penetration.

It is intended that the soil samples will be compacted by impact and this requires a degree of workability within the soil and compatible with a high speed of production. Good workability is desirable as the particles will need to "flow" past each other to achieve a uniform density through the compacted sample. Workability is determined by particle shape and the moisture content, the former depends on the soil and the latter we want to keep as low as possible.

The time between mixing in the cement and water and the final finishing impact could be the most crucial factor in compacting the mixture. One hardly wants to be breaking the cementitious bonds through the impacting process and therefore the compacting of the mixture should take place before the mixture passes the critical time. In order to achieve this, the time taken for the mixture to reach the critical time in different circumstances will need to be determined. More practically, this will probably lead to a small batch production of the cement mix so that it can be quickly compacted into finished stabilised blocks.

5.3 The curing process re-applied

The curing of the cement within the soil needs to take place in the same way that it would in a well mixed batch of concrete. Before adding moisture and allowing the curing process to begin, there should be a good particle size distribution and all the particles in the mixture should be closely packed with one another and the cement. The theory behind the bonding of the hydrated cement crystals is exactly the same with soil as it is with concrete additives. Upon the addition of water the crystals form and grow to interlock with one another leading to a high compressive strength. Full strength will not be reached for many weeks and to help the cement hydrate fully the finished mixture should be kept in a 100% humidity environment for the curing period.

5.4 Moisture attack

Most soils contain a fraction of clay as a part of their overall composition. Clay is the finest of the soil particles and can actually bond other particles together if sufficient clay and moisture is present. Clay has a very large volumetric expansion when water is added. If the moisture in unstabilised soil increases, swelling occurs. Conversely, drying causes shrinkage and therefore danger of cracking. This process leads to the breakdown of the soil and internal strength is lost making the material useless for building construction.

The balance of clay with respect to the other fractions is quite important. On one hand clay helps bond particles together, yet if another stabilising medium is not applied the clay can be instrumental in driving the particles apart should the material get wet. The common practice of firing clay bricks converts a loose particulate material into a solid ceramic. These fired bricks are no longer affected by moisture and although a modest level of porosity is still present, sustained contact with water is not detrimental to the integrity of the brick. Firing of the brick uses a great deal of energy, which either means using large quantities of firewood for small scale manufacture, or consuming fossil fuels in large-scale dedicated tunnel furnaces. The manufacture of cement also uses a large amount of energy, but that can be done away from the building site and the finished product can be delivered to where the structure needs to be erected.

6. Conclusions and recommendations for further work

Cement as a stabilising medium can be very effective if used properly. Appropriate particle size distribution, thorough mixing and maintenance of optimum moisture levels will yield a successful mix with maximum final set strength. A compromise in any of the above will result in a reduction in strength of the finished product. However final set strength is not the only requirement of a cement, adequate workability and adequate (if low) strength prior to curing are two others. These other requirements often conflict with the maximisation of final strength – for example by calling for a higher clay content.

A suitable soil can be considered to be one that has no organic material, has a clay content between 10% and 20% and has a fair range of well distributed particle sizes up to a maximum of 20mm in diameter. The moisture of the soil-cement mixture needs to be carefully controlled. There needs to be sufficient moisture for the cement to fully hydrate but no excess of water which would reduce the final density, increase porosity and reduce final strength.

The dry soil is to be mixed with the cement and the required water added. The mixture then needs to be formed and left in a 100% humidity environment within 30 to 45 minutes of mixing the cement and soil with the water. This is to ensure that the cement has sufficient water to hydrate and also that the mixture is not manipulated again after the critical time.

Curing of the mixture takes several weeks, but the green strength of the material must be sufficient to remove the formed material, handle it and perhaps even directly place it into a structure. Multi-stage curing may be possible, but the re-application of moisture may cause surface cracking and the extent of this needs to be further investigated.

Topics for further investigation

- A more detailed account of the interaction between cement and clay and why too much clay in the mixture is detrimental to the effectiveness of the cement.
- A brief study of the effect of multi-stage curing or wetting cycles on cement stabilised soil. Is an environment of 100% humidity totally necessary? or can a series of wetting cycles be just as adequate?
- How critical is the moisture content for dynamic compaction? Can a drier mix of soil can be compacted by this method better than quasi-staticly compressed soil-cement?
- If a much drier soil is used for compacting, can wetting after compacting encourage further cement particles to hydrate and hence increase the overall strength?

These questions and more will hopefully be answered later on in the project after further investigation into the available literature and perhaps after some experimental analysis of some of the interesting characteristics of soil-cement.

7. Summary

The subject of how cement stabilises soil has not been exhaustively investigated and documented during this report. However, what has been achieved is a broad understanding of the simple processes and requirements of the technique of using cement to stabilise soil. By investigating the literature available on how concrete is made, we are able to make general statements on how to stabilise soil effectively using cement as a stabiliser.

The investigation has revealed that many different factors are responsible for ensuring a good bond between the cement and the particles mixed within it. These requirements not only affect the components of the mixture used, how it is prepared, delivered into its final state, but also subsequent curing times and environmental conditions of the finished product.

Bibliography

1. Akroyd, T. N. W. (1962) *Concrete properties of manufacture,* Pergamon Press (UWL - TA 439 A5).

2. Gooding, D. E. M. (1993). *Improved processes for the production of soil-cement building blocks*. Unpublished doctoral dissertation, Warwick, University of Warwick.

3. International Labour Office (1987) *Small-scale manufacture of stabilised soil blocks,* International Labour Office (UWL - [qto] TP 832 S6).

4. Lea, F. M. (1970) *The chemistry of cement and concrete,* Edward Arnold (UWL - TA 434 L3).

5. Norton, John (1997) *Building with earth - a handbook,* Intermediate Technology Publications London. (UWL - TH 1421 N6).

6. Stulz, R. and Mukerji, K. (1993) *Appropriate building materials,* SKAT (UWL - TH 145 S8).

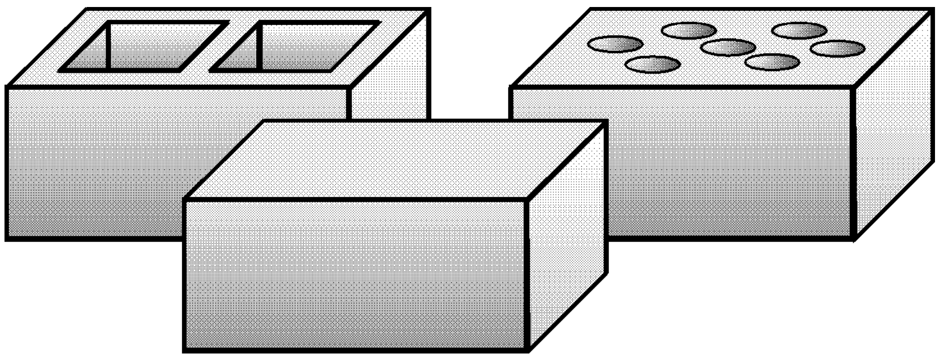
7. Teychenne, D. C., Franklin, R. E., and Erntroy, H. C. (1988) *Design of normal concrete mixes,* Department of the environment (UWL - [Pam] TA 440 T3).

8. United Nations (1972) *The manufacture and utilisation of Portland cement,* United Nations (UWL - [Pam] TP 883 I6).

Development Technology Unit
School of Engineering
University of Warwick
Coventry CV4 7AL UK
Tel (Office) +44/0 1203 522339
Director 523122, Fax 418922
Email dtu@eng.warwick.ac.uk

Web site at http://www.eng.warwick.ac.uk/DTU/

Stabilised Soil Research Progress Report
SSRPR03



Physical characteristics of soils that encourage SSB breakdown during moisture attack

Author: Mr D E Montgomery
November 1998

E-mail: d.e.montgomery@warwick.ac.uk

These reports cover 'work in progress' by research students in the Development Technology Unit (DTU) of the School of Engineering at Warwick University. Their primary purpose is internal - a format for recording ideas and data in a way that allows them to be better discussed before their incorporation into theses, DTU Working Papers or external publications. However they also have a secondary purpose, that of facilitating the sharing of our research with other innovators in the field of building with stabilised soil. Each report, after some initial internal discussion and refining will be posted as a title and synopsis on the DTU web pages (home page= http://www.eng.warwick.ac.uk/dtu). Full copies can be obtained from the respective named authors.

A dedication to someone special

Sometimes at the beginning of a publication one finds a dedication to a certain person or member of the family who has been an influence in the author's life either in general or specifically in generating the work in question. There is one person in my life that immediately springs to mind who is worthy of such a dedication. Furthermore, my experience with this person is not unique as millions of others have found him to be a great inspiration, comfort, guide and friend. "What's his name?" you may be asking yourself and, "Why haven't I heard of this incredibly influential person". The sad thing is that you probably have, but you have never accepted him as such or welcomed him into your heart and life. Well, now you have an opportunity to do just that. Please read on.

The man's name is Jesus and although he was born nearly 2000 years ago his testimony still remains and his power to save is just as great. "Save from what?" you may ask, sin and the consequences thereof, or more specifically, your sins and the consequences you face when you die. As humans we demand justice to be done, and justice will be done, but on a perfect scale and to a perfect standard. That leaves us all falling short and without hope when we come face to face with a holy God. But, God in his great love towards us send his only begotten Son into the world that the world through him might be saved. Jesus Christ died for you so that you would not have to be punished for what you have done wrong. You can be spared eternal punishment in hell and enjoy love and peace in the presence of God forever. Today the choice is yours. Reject God's free gift of love at your peril, accept it and who knows you too may have the joy of writing a dedication such as this someday. Please ponder the verses below and make your choice carefully, it will be the most important decision you ever make.

"*For by grace are ye saved through faith; and that not of yourselves: it is the gift of God: not of works, lest any man should boast.*" Ephesians 2:8,9.

"*For God so loved the world, that he gave his only begotten Son, that whosoever believeth in him should not perish, but have everlasting life.*" John 3:16.

"*For whosoever shall call upon the name of the Lord shall be saved.*" Romans 10:13

"*He that believeth on him is not condemned: be he that believeth not is condemned already, because he hath not believed in the name of the only begotten Son of God.*" John 3:18.

"*Jesus saith unto him, I am the way, the truth, and the life: no man commeth unto the Father, but by me.*" John 14:6.

Abstract

Soil is the major component of a stabilised soil block and consequently its properties are of great interest to the Stabilised Soil Block (SSB) manufacturer. Some soils are considered to be unsuitable for manufacturing SSB’s and need to be modified or discarded, whilst satisfactory soils have certain physical characteristics that can be generally suggested. The soil properties that have been found to yield satisfactory SSB’s are only a small selection of the wide range of different soil characteristics. The properties of the soil used will partly determine the way it performs under moisture attack. Other factors such as the forming technique and any stabilisation process applied will also affect the performance of the SSB during moisture attack.

The general characteristics of soil are listed in this report and special emphasis is placed on those that are known to cause detrimental effects to the SSB’s during moisture attack. If the poorer characteristics of the soil can be isolated and rectified by some means, then the result will be an enhanced product with better qualities. Those factors that cause expansion on wetting are the ones that prove to be the most negative of the characteristics. Those can be isolated into three parts, the presence of a clay fraction, the presence of porosity and the presence of moisture movement. Only with all three parts present will expansion occur and the removal or minimising of any of them will result in the removal or minimising of potential expansion of the SSB. How this can be done is the matter for another study.

Nomenclature

Aggregate: Pieces of crushed stone, gravel, etc. used in making concrete.

Brick: An object (usually of fired clay) used in construction, usually of retangular shape, whose largest dimension does not exceed 300mm.

Block: A larger type of brick not necessarily made of fired clay, but stabilised in some way, sometimes with central cores removed to reduce the weight.

Cement: Ordinary Portland Cement (OPC).

Clay: The finest of the particles found in soil, usually of less than 0.002mm in size and possesses significant cohesive properties.

Concrete: The finished form of a mixture of cement, sand, aggregate and water.

Dynamic Compaction: A process that densifies soil by applying a series of impact blows to it.

Fines: General category of silts and clays.

Gravel: A mixture of rock particles ranging from 2mm to 60 mm in diameter.

Green: Describing the state of material containing cement and water before it reaches the critical time, after which further plastic deformation hinders the final set strength.

Permeability: Describing a material that permits a liquid or gaseous substance to travel through the material.

Porosity: A measure of the void volume as a percentage of the total material volume.

Sand: A mixture of rock particles ranging from 0.06mm to 2 mm in diameter.

Silt: Moderately fine particles of rock from 0.002mm to 0.06mm in size.

Soil: Material found on the surface of the earth not bigger than 20mm in size, not including rocks and boulders and predominantly non-organic. If soil is to be used for building material it must not contain any organic material and it can be a natural selection of particles or a mixture of different soils to attain a more suitable particle distribution.

Stabilised soil: Soil which has been stabilised (treated to improve structural characteristics) by using one or more of the following stabilisation techniques: mechanical, chemical and physical.

Table of Contents

1. Introduction

Of the 29% of the earth's surface that is not covered with water, the vast majority has a soil layer on top of the underlying rock. It is this soil that supports life, man and his structures and will be the main focus of this report. Soil is a general term for particles formed by the gradual wearing away of parent rock material that is then deposited into layers onto the surface of the earth. The parent material of the rock from which the soil has been formed will largely define the composition of the soil present.

Between the top layer of soil and the rock structure there is usually a series of bands that each contain a soil with slightly different characteristics. The very top layer of soil usually contains organic material from the vegetation that has fallen to the ground and is slowly breaking down. Under this layer, one can find a mixture of organic material and small soil particles or "fines". The particle size grows as one digs deeper until the rock structure is reached. Size distribution in the soil is approximately dependent on depth. Larger particles dominate lower levels whilst finer particles the upper levels.

The basic composition of all of these layers may be the same as the underlying rock. Alternatively, material from elsewhere could have been deposited there by natural means, causing a different composition of the top layers to the bottom ones. Glaciation, floods and volcanic activity are some mechanisms through which soil from another area may be deposited locally. The composition of the small particles (sands) found in soil can generally be assessed as minerals that are silicas, silicates or limestones. As well as the solid rock particles and fragments, soil will have a proportion of water and air that fill the gaps between adjoining particles in the soil. This gives natural soil a non-homogenous and porous characteristic.

Soil that is used for building can undergo detrimental physical changes when it becomes wet. Soil will swell and contract when the water content changes and this usually leads to cracks forming in the structure. These physical changes are dependent on the characteristics of the soil both before and after processing to make the building material. The characteristics of the soil that cause these physical changes are the ones that are going to be investigated in this report. The majority of these physical changes are due to the presence off very small particles called clays. Clays perform a valuable function in the production of building blocks, but they can have a detrimental effect on the stability of the material if they get wet.

Clay is necessary to achieve sufficient green strength in a freshly formed block to enable de-moulding and handling without excessive breakage. The low moisture content and the clay particles act as a bonding agent throughout the mixture of other particles before any chemical stabilisation process has had a chance to occur. In the example of using cement as a stabiliser, a considerable period of time must pass before there is any significant gain in strength offered by the cement. A partnership of the clay and cement must be entertained, but their proportions need to be carefully monitored so that the clay gives sufficient initial strength and yet does not blind the cement particles or provoke excessive material expansion upon wetting.

2. Characteristics of soils

During this chapter the author will outline some properties of soil. These properties will include particle composition, shape, size and surface texture, some of which have standards for defining them. Ranges of values for these properties will be suggested but the basic techniques for discovering the properties of a soil sample will be described in a later chapter. This chapter will provide a summary of the characteristics that are possible to determine from a sample of soil.

2.1 Physical

Some of the physical characteristics that could be used to define soil particles are: colour, size, shape, surface texture, density and specific surface area. The variety of physical characteristics of soil particles that can be found is considered to be virtually infinite. The analysis of some of these characteristics can been done using a simple set of field tests and personal interpretation, or, more complex and accurate tests can be carried out in the laboratory. Systems for identifying some major characteristics have been developed to define different ranges of soil characteristics. The most common of these is the size distribution of the soil particles. Below is a list of physical characteristics than can define a sample of soil. See (Houben & Guillaud, 1989), (p. 30,31) for more details.

Colour: Can range from white through to black with shades of tan, brown, red, grey and even blue and green. This is however an arbitrary and trivial description that is not standardised and based entirely on personal interpretation. Good for quick visual identification and can even suggest chemical composition of the soil, but accurate measurement is not defined.

Shape: Broadly defined as three different categories; angular, sub-angular or rounded. This can be assessed using visual interpretation and/or the feel of the soil. Only used as a general descriptive term, as accurate measurement is not a viable option. The ratio of particle surface area to the surface area of a sphere of the same mass can be an indication of shape and is defined as:

$$\lambda = \frac{Surface\_area\_of\_particle}{Surface\_area\_of\_sphere}$$

Where: Rounded – $1 < \lambda < 1.2$

Semi-Angular – $1.2 < \lambda < 1.5$

Angular – $1.5 < \lambda$

Note: a four faced pyramid has λ value of 1.49.

Apparent bulk density: This is a measurement of the overall density of the soil sample including air and/or moisture present within the sample. The measurement of apparent bulk density is a trivial exercise, as one only needs the volume of a sample and its mass. Apparent density of a block is simply $\rho_{apparent} = \frac{M_{sample}}{V_{sample}}$ (measured in kg/m³).

Specific bulk density: Can be accurately measured following British Standard BS7755: 1998. This method splits up the soil into two sections and measures the density of the two sections in different ways. For particles smaller than 2 mm in size a small sample is placed in a pyknometer and the displaced water at a known temperature will give the volume occupied by the soil. The sample is also accurately weighed to give the mass of the sample. The specific bulk density is calculated from the mass of the sample and the displaced volume of water in like manner to the apparent bulk density. For particles over 2 mm in size the sample is weighed and is then suspended in water that is resting on a set of scales. The mass of the displaced water gives the volume for a known ambient temperature and the specific bulk density can be calculated from these two values, (also measured in kg/m³).

Size or texture: One of the most common methods of identifying the size of particles that can be found in a sample of soil is to use the British Standard BS7755: 1998 classification for particle sizes. This separates the soil into different fractions depending on physical dimensions by means of a number of different meshes and sieves. The sample of soil is passed through the largest mesh first and each subsequent mesh until all the soil has been separated off at a level appropriate to its size. For laboratory testing the soil needs to be dry and any particles deflocculated to ensure accurate results. The size ranges as defined by the British Standard along with their common names are listed below.

Trivial name	Size range in mm
Boulders	> 200
Cobbles	60 – 200
Gravel (Coarse)	20 – 60
Gravel (Medium)	6 – 20
Gravel (Fine)	2 – 6
Sand (Coarse)	0.6 – 2
Sand (Medium)	0.2 – 0.6
Sand (Fine)	0.06 – 0.2
Silt (Coarse)	0.02 – 0.06
Silt (Medium)	0.006 – 0.02
Silt (Fine)	0.002 – 0.006
Clay	< 0.002

Moisture content: Soil is very seldom totally dry, and how much moisture is present is important for determining the properties of the soil in general. Measuring this moisture content is done through a process of weighing and drying in an oven. Following British Standard BS1377: 1990 the sample must be weighed at regular intervals until the difference between consecutive weights are less than 0.1% of the whole sample mass. This usually means drying the sample for about 24 hours to ensure that it is virtually dry. The difference in mass from the initial weighing to the last weighing will be the mass of water. The moisture content is calculated as a percentage of the total mass of the sample before drying.

Porosity or voids ratio: A substance is considered porous if it has a matrix of voids throughout it. A very simple and common example of a porous object is a sponge. Soil is another such substance, but the porosity of soil can vary enormously depending on the particle size and distribution within the soil sample. To calculate the porosity of a sample of soil one needs to know both the apparent bulk density and the specific bulk density of the sample. The porosity or voids ratio is unity minus the ratio of the volume of soil alone to the volume of the sample, both of which will have been found when the apparent bulk density and the specific bulk density of the sample were measured. Porosity: $P = 1 - \frac{V_{soil}}{V_{sample}}$ (generally expressed as a percentage).

Permeability: A porous material becomes permeable when these air pockets are arranged in such a way so that a gas or fluid can pass through the substance. The permeability will largely depend on the porosity present in the sample. A sandy soil will be considered highly porous and will have a low resistance to the passage of water through it. A clayey soil is the opposite and will resist the penetration and passage of water. Permeability is a measure of how fast a fluid moves through a substance. The British Standard BS 8004:1986, (Craig, 1997), (p. 40) gives a list of coefficients of permeability and also tests for permeability that can be carried out in the laboratory. A falling head test and a constant head test are two ways that permeability can be measured. The flow of water through a sample is measured and described as a flow rate per unit of time. (Permeability of stabilised soil is too low to be measured by these means).

Effective surface area: Particles that are so small that they will pass through even the finest sieve usually have a different means of identification. This is called the effective surface area of the particles in question and is usually measured in m^2/g of material. This helps to distinguish between small and large clay particles for which any other classifications are useless. Three appropriate examples of this analysis are the three main types of clays Kaolinites, Illites and Montmorillonites that have approximate effective surface areas of $30m^2/g$, $80m^2/g$ and up to $800m^2/g$ respectively, (Houben & Guillaud, 1989), (p. 27).

Adhesion: Described as the ability of soil to stick to other objects at a given humidity. It will increase as the humidity increases up to a point after which it will then decrease as the humidity continues to rise. Of interest usually with soil sticking to metallic tools.

Specific heat capacity: Defined as the amount of energy required raising one kilogram of the soil by one degree Kelvin. Units are J/kgK, (joules/kilogram Kelvin).

Dry strength: Highly dependent on the quantity and type of clays present in the soil sample. Measured in MPa to crush (effectively describing the shear strength of the soil sample). The dry strength of clays varies from around 0.07 MPa to 7 MPa, (Craig, 1997), (p. 31).

Linear contraction: Sometimes described as the shrinkage of a particular sample. Linear contraction is highly dependent on the clay type and content and the water content. Standard shrinkage tests start with a soil at it's liquid limit. It is usually tested in a long narrow trough that is filled with moist soil and allowed to dry out. The contraction is limited to one direction and the linear quantity can be measured as either a percentage or as a ratio of overall length. The general rule is, the greater the shrinkage the greater the clay content.

2.2 Chemical

The chemical composition of the soil particles will be of interest when chemical stabilisation is taking place, or if the soil will be in an environment where the elements will be susceptible to chemical attack, (e.g. limestone is attacked by acid rain). Soil is generally a stable compound because it has been formed over a long period of time and any chemical changes will have already occurred to it in the environment. For the majority of cases the scientist can assume that soil will be chemically unaffected by the environment.

Composition: The soil particles will have roughly the same chemical composition as the parent rock from which the soil was formed. This chemical composition can range from salts or chalk composition through to iron and aluminium oxide composition. (Houben & Guillaud, 1989), (p. 36,37) gives more details on the many different types of soil that are present and their respective chemical composition.

Mineral content: Minerals present in the soil are unstable components that are being processed by the environment usually as a result of decaying organic matter. Any organic matter should be avoided and unstable components resulting from them should also be regarded as potentially detrimental to structure longevity.

Metallic oxides: Soil can contain a significant quantity of metallic oxides such that they are used to produce the metal through smelting the soil. Bauxite for aluminium and ferrous oxide for iron are two common oxides present in soils.

pH levels: Soils can be either acidic or basic in pH level, but they do not usually stray very far away from the neutral point. Their pH will depend on the H^+ and OH^- ions that are present and these ions will depend on the chemical composition of the particles themselves and their interaction with one another.

Sulphates: These are soluble compounds of elements and will be affected by changes in moisture. Leeching of soil can occur if water passes through it removing any soluble salts or substances with it. Sulphates can cause problems with cement and soils with sulphates present should be avoided if cement is being used as a stabilising medium.

3. Measuring soil characteristics

In the previous chapter many different physical and chemical characteristics of soil were defined. However, most of the processes for measuring each of those different characteristics were omitted, that will be the focus of this chapter. It is not necessary for all of the above characteristics to be determined for every soil sample taken. The relevant ones will be discussed and listed. Following this some techniques for measuring these properties will be described. Some techniques are restricted to the laboratory, whilst other are considered to be sufficiently accurate for field tests using limited equipment.

3.1 Some relevant properties for making SSB's

With so many different characteristics that one could discover about a sample of soil, it would be foolhardy to try and discover them all in every situation that soil is to be used for making SSB's. Only a small number of different characteristics are of real relevance to the scientist testing the soil. The chemical composition of the soil is of little importance once the absence of unstable compounds and organic matter has been established. The physical properties are of greater interest for making SSB's as these will help to determine its ease of mixing, forming, de-moulding, porosity, permeability, shrinkage, dry strength and apparent bulk density.

The particle size distribution or texture of the soil is a necessary characteristic to determine, as it will help the scientist to measure the sand, fines and clay content. These are necessary to ensure that the material being used falls within the parameters suggested for making SSB's. The moisture content of the soil is another critical characteristic as it affects a number of factors in SSB production. What moisture content the soil has in relation to its optimum moisture content is of great interest to the SSB manufacturer as this will help determine potential shrinkage. For soil mechanics the Optimum Moisture Content (OMC) is defined as the maximum amount of water that can be added to the sample that completely fills all the air voids present throughout the material and no more. The moisture content also has a marked effect on material workability, cement curing, drying times, de-mould slump and porosity.

Consequently the OMC for a soil may not necessarily be the optimum moisture content for stabilised soil material.

3.2 Field tests

Field-testing methods are many and varied, and will depend vastly on the judgement and previous experience of the person carrying them out. There also seems to be conflicting information about what certain tests reveal about the characteristics of the soil. Gooding noted these differences in his thesis and the summary below is largely taken from his suggestions for interpreting the results received from each test. Assuming that the exact characteristics of the soil are not necessary, these field tests will give the user a reasonable idea of the type of soil that is present.

Smell test: Detects the presence of organic material if a musty odour is sensed. Soil with organic material is unsuitable for manufacturing SSB’s and should be rejected. The organic layer usually exists on the top of the soil and can easily be removed to reveal more suitable soils underneath.

Visual-touch test: This test will determine the range of particle sizes present. A soil containing mostly large particles (over 2 mm in size) is a sandy and gravely soil, it will easily break up and run through the fingers. Such a soil has a low fines content and is unsuitable for making SSB’s. Conversely a soil containing no sand particles and only smaller particles that is hard to the touch, difficult to break up and reveals a fine powder that is difficult to wash off is classified as a soil with an excess of fines and clays. This too is unsuitable for making good SSB’s.

Thread test: If a mixture of sands and fines is present then the soil can be formed into a thread upon the addition of some water to increase its plasticity. If the thread can be rolled to a diameter of less than 3 mm then the fines content is too high and more sand will need to be added. If a thread of as little as 5 mm diameter cannot be formed then insufficient fines is present and more will need to be added. A thread that breaks up at a diameter of around 4 mm has a sufficient fines content for making SSB's.

Shine test: After the above tests have been carried out the shine test indicates the level of fines present in the sample. A mixture with a high fines content will achieve a shiny surface if scratched with the fingernail. This shiny surface is caused by a moderately high presence of silt and clay and is be acceptable for making SSB's. A dull surface finish will indicate a sandy composition with a low fine content and this is also suitable for SSB production.

Glass-jar test: This test will give the investigator a rough idea of what percentage of each fraction is present in the soil sample. The test requires a glass straight-sided jar to be a quarter or a third filled with soil and the remainder filled with water. The jar is then sealed and rotated end over end for several minutes to ensure that all the particles have been broken up and held in suspension within the water. The jar is then placed on a flat surface and left undisturbed for some time. A cloudy mist of very fine particles may stay in the solution indefinitely, held there by Brownian motion, but these are only particles less than 0.0002 mm, (Craig, 1997, p. 7) and can be ignored. All the other fractions should have settled to the bottom within a few days and should be easily distinguishable from one another. A sandy layer should be present at the bottom with smaller particles at higher levels. The particles in suspension fall out of solution according to Stokes' law, which states the larger the particle size the faster the decent velocity and vice versa, (Craig, 1997, p. 6). The clay and silt fractions may not be distinguishable from one another and these can often be combined to yield a simple coarse to fines ratio for the soil sample. The quantity of the different fractions can be found by measuring the depth of particles within each fraction and calculating each fraction as a percentage of the whole settled depth.

A basic analysis of the results found from the glass-jar test can be summarised as follows:

- More than 80% sand and gravel (if present) indicates very low fines content and is considered unsuitable for making SSB's.
- Between 70 – 80% sand shows a low fines content and can be used for SSB's.

- Between 50 – 70% sand shows a high fines content and can also be used for SSB's.
- Less than 50% sand indicates very high fines content and should not be used for manufacturing SSB's.

Shrinkage test: A mould of dimensions 40 × 40 × 600 mm is filled with soil near its liquid limit, (the point at which the soil passes from a solid state to a liquid state). This soil is then left to dry out slowly. The mould walls are treated with grease or a lubricant so that as the bar of soil shrinks in size it slides along the mould walls. The difference between the initial length and the final length is the linear shrinkage. This is usually represented as a percentage of the original length.

All the above field-tests it can be done on a relatively short period of time with simple equipment. The interpretation of the results is where the inconsistencies can arise, especially between different field scientists. Nevertheless these tests are sufficient as a preliminary check for initial analysis and use for even medium-sized projects. Small projects would be classified as the building of one or two dwellings undertaken by an individual or family. Larger projects would have significant funding and could justify further tests to establish soil characteristics more accurately.

3.3 Laboratory tests

The larger projects that require more careful analysis of the soil properties will find that field tests will be insufficient and laboratory testing will need to compliment these to ensure an accurate analysis of the soil present. Testing is usually only justified if a very large amount of soil will be used and an area of land is being surveyed for excavation. The survey will reveal the different properties of the soil in different locations and will help to direct the SSB manufacturer to the best source of soil for making the SSB's. Below is a list of characteristics and the methods for accurately measuring those properties.

Particle distribution: Accurate measurement of the particle distribution has already been hinted at in chapter two, where the particle size distribution or texture of the soil was defined as a physical characteristic of soil. British Standard BS7755: 1998 classification for particle sizes describes a process that separates the soil into different fractions depending on physical dimensions by means of a number of different meshes and sieves. The exact method of this should be referenced from the British Standard as such standards are updated regularly, or its local or national equivalent.

Apparent and specific density: If these two values for the soil are known then the porosity can be measured. Measuring the apparent density is straightforward as only the overall volume and dry mass is required of the soil sample. The water must be removed from the sample before weighing as it will add to the overall mass of the soil and give an inaccurate density of the soil and air mixture. Suggested measurements and calculations are as follows:

Volume of undisturbed soil sample (including air voids that may be partially filled with water) = V

Mass of dried soil sample = M

Apparent Density $\rho_{app} = \dfrac{M}{V}$

The specific density has to be measured in accordance with British Standard BS775:1998 and again this should be referenced to include current changes and modifications.

Porosity/permeability: In simple terms the volume of dispersed air voids within a sample is proportional to the porosity. Porosity can be easily calculated from the specific and apparent bulk densities of the soil sample if they are known. The porosity can be calculated both before and after processing of the soil.

Permeability can be measured as a function of the flow rate of a fluid through a porous substance. Darcy’s empirical law defines the permeability of soil, but this is only limited to one dimensional flow of water through a fully saturated soil, (Craig, 1997), (p. 39).

Darcy's law states: $q = Aki$

where q = the volume of water flowing per unit of time, A = cross-sectional area of the soil corresponding to the flow q, k is the co-efficient of permeability and i is the hydraulic gradient.

Darcy's law can also be written as:

$$v = \frac{q}{A} = ki$$

where v is the discharge velocity of the water through the soil.

4. Effect of moisture on soils

The fundamental problem with building with soil is that it will lose compressive strength when it becomes wet. This is not a desirable characteristic for walls supporting a roof structure with inhabitants underneath it. Consequently it is the responsibility of the designer to ensure that either the weakening effect that moisture has on the soil is greatly reduced, or the possibility of the soil getting wet is removed. For building with soil where there is little or no rain, then the problem is negligible, but for wetter climates it is a serious concern. Techniques used in the past to overcome the problems of building with soil in wet climates have included mechanical and chemical soil stabilisation, wall painting or rendering and use of wide roof eaves.

4.1 Detrimental characteristics

It is important to isolate the characteristics that are most useful for the SSB manufacturer to know about the soil that is being worked with, so that they can be closely monitored. These are usually the characteristics that greatly affect the resistance of the soil to moisture attack. Below is a list of these poor characteristics and how they might be improved for general use.

High porosity/permeability: These are two characteristics of soil that can cause the potential swelling and cracking that is so detrimental to SSB's durability. No matter how much clay is present, if water cannot penetrate then the clay will not swell and integrity can be preserved. Render or paint will provide such protection, but only at significant cost and regular maintenance is always required. A high porosity will permit moisture to penetrate the surface of the block and then subsequently flow into the internal structure of the soil particles distributing moisture to other soil particles. This process causes water to coat the soil particles and by the process of surface tension drive neighbouring particles further apart. This mechanism is particularly severe with the clay fraction of the soil. Reducing the porosity can be achieved by compacting the soil and therefore increasing its apparent bulk density. Porosity of the soil itself can never be reduced to zero, but a significant improvement to the resistance of moisture penetration can result through compaction.

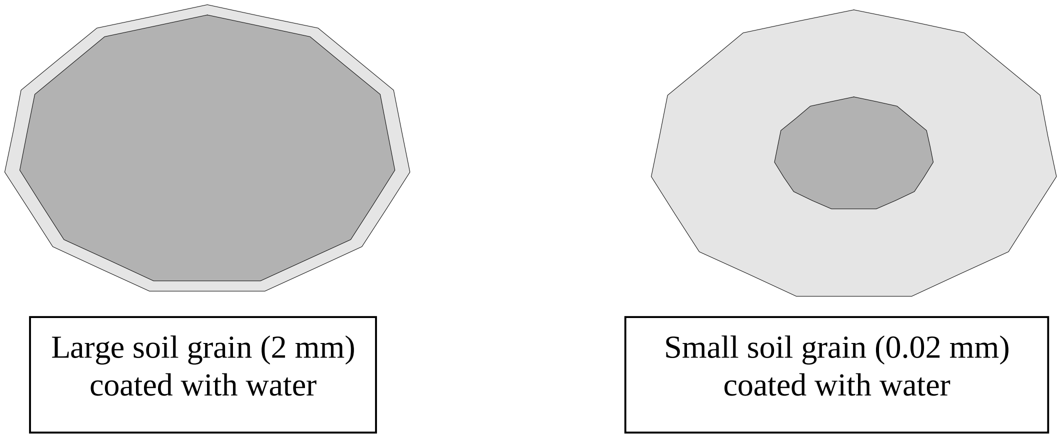
A very high level of porosity in the finished block will mean that the structure will no longer be able to keep out the elements, such as wind, rain and temperature variation. Clearly this is undesirable, as these are some of the most basic functions of a dwelling. As porosity cannot be removed completely from a basic building material such as soil, the level of porosity that is acceptable or even desirable needs to be identified. Taking the other extreme, in a hermetically sealed dwelling, there is no potential for the passage of air or moisture from the inside of the dwelling to the outside world. This is also unacceptable as humidity and oxygen levels from respiration will make the living space uncomfortable. A balance between the two extremes needs to be rationalised.

A major factor that would concern a dwelling designer is the time taken for the building material to respond to changes in climate both inside and outside the dwelling. For example, if outside has a very high humidity and the inside is kept comfortably dry, how long will it take (assuming the conditions are sustained until equilibrium is reached) for the inside of the wall to have the same characteristics as the outside. Perhaps a better analogy is using one of heat. If the outside temperature is 10°C and the inside temperature is 20°C then there is a thermal gradient of 10° between the internal and external faces of the wall. The thermal gradient exists because the wall possesses a thermal resistance and the internal temperature is being sustained by a heat source. If that heat source was removed the temperature of the wall would equalise and the gradient would be reduced to zero. The same principle can be applied to the moisture content of the wall. If the outside is wet due to rain, and the wall is porous then the moisture will migrate to the inside face of the wall saturating the wall. Ensuring that the internal face of a wall can remain dry and the wall itself can survive a 50-year storm would not be an unreasonable request for the average homeowner.

Depth of moisture penetration is another consideration that would concern the SSB builder as a small degree of penetration can be tolerated, but deeper penetration may be unacceptable in the long term. Moisture ingress affects the strength of SSB's but the effect on strength is usually not so significant to cause collapse. A more common mode of failure is spalling of the surface of the blocks as moisture has penetrated, caused expansion and subsequent contraction and cracks have occurred. These cracks permit further moisture penetration and cause more expansion and cracking to occur. These cracks if permitted to continue begin to jeopardise the integrity of the block surface initially and then the structural strength of the block itself. Over time the surface of the block falls away permitting deeper moisture penetration and progressive destruction of the block.

High fines/clay content: The smallest of the particles in the soil are the ones that exhibit the greatest expansion when they become wet. If there is a high percentage of this fraction of soil then the potential expansion will be significant as more particles become coated with water and drive neighbouring particles apart. There are two remedies for the effect of this type of soil, firstly to reduce the fines/clay content by mixing in coarser grains, or to add a stabiliser such as cement in such high quantities that the particles are restrained from moving when water is added. Both will work in practice, but the latter is an expensive exercise and the former should be attempted if possible.

The larger sized fractions of soil are generally unaffected by moisture. They will gain a thin film of water on their surface, but this will be small compared to the grain size. The smaller grain fractions achieve a similar thin film of water on their surface that is of the same order or magnitude or bigger than the grain size. The diagram below illustrates this phenomenon.



The Large soil grains will be coated with a thin layer of water, but this will not increase the size of the particle significantly. If thicker layers of water tried to coat the particle gravity would begin to have an effect and excess water would drip off the bottom of the grain. The surface tension that holds the water onto the grain surface will not be strong enough to create pore pressure that pushes other particles further away from each other. The small grains, on the other hand, will be significantly larger when coated with water and will cause a volumetric expansion of the particles. At this scale the surface tension will be strong enough to move particles further apart and to cause significant overall expansion.

High linear contraction: Again, this linear contraction is due to the presence of clays and fine particles that shrink back together when the moisture around them is removed. The contraction will also depend on the moisture content when the soil is formed and then left to dry/harden/cure. Higher initial moisture contents will result in higher overall shrinkage of the soil. Clearly reducing the initial moisture content will help to reduce initial shrinkage, but ultimately it is the clay content that will determine the amount of expansion and shrinkage. Again the shrinkage can be limited by the addition of cement to the soil. The amount of shrinkage will determine the quantity of cement that will be required to effectively stabilise the soil. As described in (International Labour Office, 1987), (p. 38-39), the cement to soil ratio is as follows for a given shrinkage as determined by the shrinkage test described in chapter 3.2.

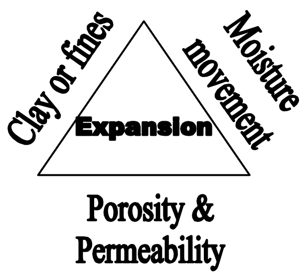
Measured Shrinkage (mm)	Cement to soil ratio
Under 15	1:18 parts (5.56%)
15 – 30	1:16 parts (6.25%)
30 – 45	1:14 parts (7.14%)
45 – 60	1:12 parts (8.33%)

Adhesion: If the soil is moist and has a high adhesion to metallic surfaces, it will cause significant problems when de-moulding. The simplest way of reducing the adhesion exhibited by the soil is to reduce the moisture content when the soil is formed in the mould.

4.2 Significance to making SSB's

The above section detailed the characteristics that the SSB manufacturer would want to avoid. In practise these characteristics are impossible to remove altogether and a compromise needs to be made somewhere along the line. This section aims to explain the effects that the above characteristics have on SSB's produced in the field.

Expansion of a SSB can only occur if three characteristics are present: Clays or fines and Porosity & Permeability and Moisture movement. If any one of those is absent then expansion and contraction will not occur, (ignoring chemical and thermal expansion and contraction). The diagram to the right illustrates the idea.



It is the job of the SSB manufacturer to minimise these characteristics in the blocks that are being produced so that potential expansion is reduced to acceptable levels. External environmental changes will cause the moisture levels to rise and fall over time. This will not have an effect on the SSB unless the moisture levels within the SSB also change. Moisture will only be able to penetrate the SSB if porosity and permeability are present. Swelling and shrinkage will only take place if the moisture reaches a fraction of clay present within the SSB and sufficient cement is not present to resist the potential expansion offered by the clay or fines. Therefore there are three factors that want to be controlled: the clay fraction, the porosity of the material and the rate of moisture movement.

Controlling the clay fraction: Too much clay results in unacceptably high expansion upon wetting or excessive amounts of cement to combat this. Too little clay causes low adhesion between particles and hence causes high breakage rates on de-moulding of the SSB's. Either situation is unacceptable and this can only be achieved by monitoring the clay and moisture content when the soil is to be formed. How closely this has to be done to achieve satisfactory results is not clear. An optimum fines content for making SSB's was suggested by the United Nations to be about 25% of which more than 10% is clay, (Gooding, 1993), (p. 263). From the literature it is unclear how much a change of say ±5% to the clay content will have on the overall performance of the SSB.

Porosity: An ideal for the level of porosity for any type of SSB would have to be zero. Since this is a physical impossibility a small amount of porosity needs to be tolerated. The greater the porosity the more susceptible the SSB will be to the elements and more specifically, the permitting of water penetration. In certain cases it is impossible to avoid water getting onto the face of the block, e.g. blowing rain, but what must be stopped is the water penetrating into the block itself. Water in the block will cause expansion and deterioration of compressive strength unless it is compensated for with a high cement content. If the level of porosity at the surface is much less than deeper into the block then this also is an acceptable situation. If water does not penetrate the surface then it will not matter if the porosity is lower where the water does not reach as this area will be unaffected. Using steel-sided moulds and dynamic compaction give good surface finishes and will result in a slightly lower surface porosity than deeper into the SSB core.

Possible moisture penetration models:

Capillary action without differential pressure - Unknown mechanism but very effective on small pored materials.

Gravitational force pushing water into pores of SSB - Surface water on SSB pushes water into SSB through pores.

Pressure difference flow - Low pressure internally in block with high pressure outside SSB drives moisture from one side to the other.

Moisture levels: More water means more shrinkage upon drying and potentially higher adhesion to metal surfaces, but some moisture is required to keep the soil in a workable state and also to hydrate any cement particles if they are used to help in the stabilisation process. Careful control of the moisture levels is also required to ensure that the soil has adequate adhesion to itself to reduce SSB breakage upon de-moulding. If the moisture level change during the life of the SSB, then moisture movement has occurred. Initially this happens when the block is dried out after it is formed. Subsequent moisture movement should be avoided. Moisture will only be able to enter or level the block if porosity and permeability are present and these can be reduced by adjusting the particle size distribution and the apparent density of the finished block.

5. Conclusions and recommendations

The characteristics that define soil are many and varied. Defining a soil with any degree of accuracy from all the different soils present in the world is a difficult task. With such a variable substance, one can appreciate the difficulties posed to the SSB manufacturer to ensure that the soil that is chosen will be acceptable for the intended task. An even greater problem is determining what effect slight changes to the soil's texture, porosity and moisture content will have on the finished product. This is not helped by the fact that these properties will affect one another as they are sometimes inter-dependant. For example, if the moisture content is high during manufacture then there will be a higher porosity when all the moisture has been removed. If the texture is carefully controlled then this will have an effect on the porosity and apparent density.

Further analysis of how different characteristics affect one another in general should be looked into more closely. A cause and effect chart displaying all the different characteristics and how each is effected by changes in different characteristics would be very helpful. It may be possible to determine that all the different characteristics are linked mathematically and any change in one property will result in changes in a number of others. This model may have to be limited to only a few simple characteristics as the overall variability and complexity of soil may be too difficult to model with any degree of accuracy.

The mechanism through which water penetrated a block is another area where further study should be undertaken. How and why water wishes to permeate a porous substance against the forces of pressure and gravity is a question that needs to be answered. The adhesion of water to surfaces and the internal cohesion that it has with itself are major factors in the situation. How these forces can be hindered so that water is less likely to penetrate a block would be very useful to know if it is possible. Water cannot penetrate certain porous objects because the pores are too small for the water to penetrate into them. At what level this occurs and whether it can be achieved by modifying soil characteristics physically is not known at the moment and should be investigated further.

6. Summary

Some physical characteristics of soil have a major influence in the potential for expansion when it becomes wet. These can be isolated into the clay/fines fraction, the porosity/permeability and the moisture movement. Only with all three factors present will expansion occur. Monitoring the clay fraction and apparent density can be easily done using simple tests, but finding the porosity and hence the potential for moisture movement is a bit more complex. Cement will hinder expansion to a certain degree, but if the root problem can be eliminated rather than trying to constrain the effect of the problem then that would be much more advantageous.

As the SSB's will be in an environment that exhibits changes in moisture and clay is an important component of the block's composition then the only factor that can be reduced is the porosity and permeability of the SSB. The porosity cannot be reduced to zero, but there may be a point at which the SSB becomes impermeable to water. This is the desired condition and this may be achieved by monitoring the particle size and distribution, the moisture content and the apparent density of the final SSB. How exactly this can be done in practice is still open to further discussion. At least now we know the offending characteristics that cause material expansion and consequently we are better equipped to minimise their effects and to deal with their consequences.

Bibliography

1. Craig, R. F. (1997) *Soil Mechanics,* E & FN Spon (UWL - QC 137.7 C7).

2. Gooding, D. E. M. (1993). *Improved processes for the production of soil-cement building blocks*. Unpublished doctoral dissertation, Warwick, University of Warwick.

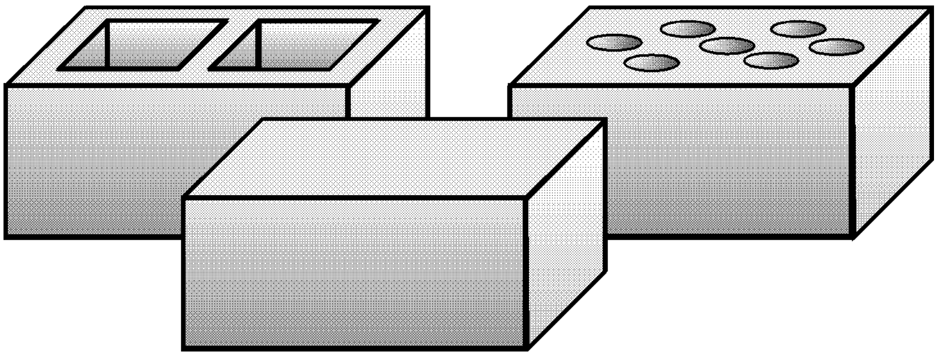
3. Houben, Hugo and Guillaud, Hubert *Earth Construction: A Comprehensive Guide,* Intermediate Technology Publications.

4. International Labour Office (1987) *Small-scale manufacture of stabilised soil blocks*, International Labour Office (UWL - [qto] TP 832 S6).

Development Technology Unit
School of Engineering
University of Warwick
Coventry CV4 7AL UK
Tel (Office) +44/0 1203 522339
Director 523122, Fax 418922
Email dtu@eng.warwick.ac.uk

Web site at http://www.eng.warwick.ac.uk/DTU/

Stabilised Soil Research Progress Report
SSRPR06



Initial critique of existing papers on dynamic compaction of stabilised soil samples

Author: Mr D E Montgomery

April 1999

E-mail: esrsj@csv.warwick.ac.uk

These reports cover 'work in progress' by research students in the Development Technology Unit (DTU) of the School of Engineering at Warwick University. Their primary purpose is internal - a format for recording ideas and data in a way that allows them to be better discussed before their incorporation into theses, DTU Working Papers or external publications. However they also have a secondary purpose, that of facilitating the sharing of our research with other innovators in the field of building with stabilised soil. Each report, after some initial internal discussion and refining will be posted as a title and synopsis on the DTU web pages (home page= http://www.eng.warwick.ac.uk/dtu). Full copies can be obtained from the respective named authors.

Titles of Stabilised Soil Research Progress Reports Produced to date:

[Put printed list of current reports in place of this page.]

A dedication to someone special

Sometimes at the beginning of a publication one finds a dedication to a certain person or member of the family who has been an influence in the author's life either in general or specifically in generating the work in question. There is one person in my life that immediately springs to mind who is worthy of such a dedication. Furthermore, my experience with this person is not unique as millions of others have found him to be a great inspiration, comfort, guide and friend. "What's his name?" you may be asking yourself and, "Why haven't I heard of this incredibly influential person". The sad thing is that you probably have, but you have never accepted him as such or welcomed him into your heart and life. Well, now you have an opportunity to do just that. Please read on.

The man's name is Jesus and although he was born nearly 2000 years ago his testimony still remains and his power to save is just as great. "Save from what?" you may ask, sin and the consequences thereof, or more specifically, your sins and the consequences you face when you die. As humans we demand justice to be done, and justice will be done, but on a perfect scale and to a perfect standard. That leaves us all falling short and without hope when we come face to face with a holy God. But, God in his great love towards us send his only begotten Son into the world that the world through him might be saved. Jesus Christ died for you so that you would not have to be punished for what you have done wrong. You can be spared eternal punishment in hell and enjoy love and peace in the presence of God forever. Today the choice is yours. Reject God's free gift of love at your peril, accept it and who knows you too may have the joy of writing a dedication such as this someday. Please ponder the verses below and make your choice carefully, it will be the most important decision you ever make.

"*For by grace are ye saved through faith; and that not of yourselves: it is the gift of God: not of works, lest any man should boast.*" Ephesians 2:8,9.

"*For God so loved the world, that he gave his only begotten Son, that whosoever believeth in him should not perish, but have everlasting life.*" John 3:16.

"*For whosoever shall call upon the name of the Lord shall be saved.*" Romans 10:13

"*He that believeth on him is not condemned: be he that believeth not is condemned already, because he hath not believed in the name of the only begotten Son of God.*" John 3:18.

"*Jesus saith unto him, I am the way, the truth, and the life: no man commeth unto the Father, but by me.*" John 14:6.

Abstract

Implusion (dynamic) compaction of soil building blocks has been shown to promise certain advantages over block pressing, however previous researchers have already expressed their dismay at the general lack of information in the field of dynamic compaction of soil blocks. This paper reviews what such information is redily available. The information that is available on dynamic compaction mainly comes from the civil engineering industry from ground compaction methods. Whilst these are suitable for gaining a basic understanding of soil compaction, they are not entirely applicable to compaction of blocks confined in a mould. Modelling of the compaction process has been attempted within this field and some mathematical models are described in this report.

Dynamic compaction of soil blocks without the use of cement has been investigated to establish optimum compaction efficiencies when the energy transfer is kept constant. This has shown that between 8-32 blows gives the greatest compaction for the same total energy transfer. The research did not investigate the effect of adding cement to the compaction process, nor did it identify the moisture content to optimise dry block strength. Research done in the civil engineering industry has briefly investigated the effect of moisture on unconstrained compaction as well as the efficiency of different methods of energy transfer. These results are significant but cannot easily be applied to the research done on block compaction.

Several major gaps in the understanding of soil compaction still exist, and these need to be tackled one by one. It is of fundamental importance that thorough testing of dynamically compacted cement stabilised block be carried out in the near future. Optimisation of energy transfer can yield small increases in density, which results in much greater gains in strength. More time spent researching the optimum method of energy transfer would be a valuable exercise especially with the addition of cement which has an effect on the compaction process.

Nomenclature

Bre-pack machine: A high quality 10MPa manual block-making machine as developed in the U.K. for block manufacture in developing countries.

Brick: An object (usually of fired clay) used in construction, usually of rectangular shape, whose largest dimension does not exceed 300mm.

Block: A larger type of brick not necessarily made of fired clay, but stabilised in some way, sometimes with central cores removed to reduce the weight.

Cement: Ordinary Portland Cement (OPC).

Clay: The finest of the particles found in soil, usually of less than 0.002mm in size and possesses significant cohesive properties.

Concrete: The finished form of a mixture of cement, sand, aggregate and water.

Dynamic Compaction: A process that densifies soil by applying a series of impact blows to it.

Fines: General category of silts and clays.

Green: Describing the state of material containing cement and water before it reaches the critical time, after which further plastic deformation hinders the final set strength.

Permeability: Describing a material that permits a liquid or gaseous substance to travel through the material.

Porosity: A measure of the void volume as a percentage of the total material volume.

Sand: A mixture of rock particles ranging from 0.06mm to 2 mm in diameter.

Silt: Moderately fine particles of rock from 0.002mm to 0.06mm in size.

Soil: Material found on the surface of the earth not bigger than 20mm in size, not including rocks and boulders and predominantly non-organic. If soil is to be used for building material it must not contain any organic material and it can be a natural selection of particles or a mixture of different soils to attain a more suitable particle distribution.

Stabilised soil: Soil which has been stabilised (treated to improve structural characteristics) by using one or more of the following stabilisation techniques: mechanical, chemical and physical.

Table of Contents

1. Introduction

There is a small quantity of existing information on dynamic compaction of stabilised soil blocks, but this is limited to a few simple surveys and thesis reports. Much of the work for this project will reference these previous works as they too discovered a lack of information in this field. Other studied have provided information of direct relevance to other fields, but which can only be applied to the field of interest with a small degree of confidence.

Soil compaction is an important area of study within the civil engineering and geotechnics and this is similar to the working being carried out here. Some sources give a bit of detail on a form of dynamic compaction that is used to compact soil prior to construction, or to aid stabilisation of slopes etc. These are of interest especially if any quantitative description of the compaction process is given that would be useful in application to compaction of blocks.

Ground compaction always concentrates on a small area of ground where compaction is desired and the machinery used has to move around the area to ensure thorough compaction of the desired surface. This type of compaction could be considered analogous to the tamping down of soil in a block mould or the compaction of soil between shutters for rammed earth walling. However, simultaneous compaction of the entire block surface is not in the same category as there are no potential slip planes for soil movement under the direct compaction force. Unlike ground stabilisation the compaction force is uniform over the whole surface of the block making the two processes fundamentally different to each other. This makes the information in this field interesting, but not entirely useful. Consequently much of the research into dynamic compaction of soil blocks will be received from previous research done by Dr. Gooding and his thesis.

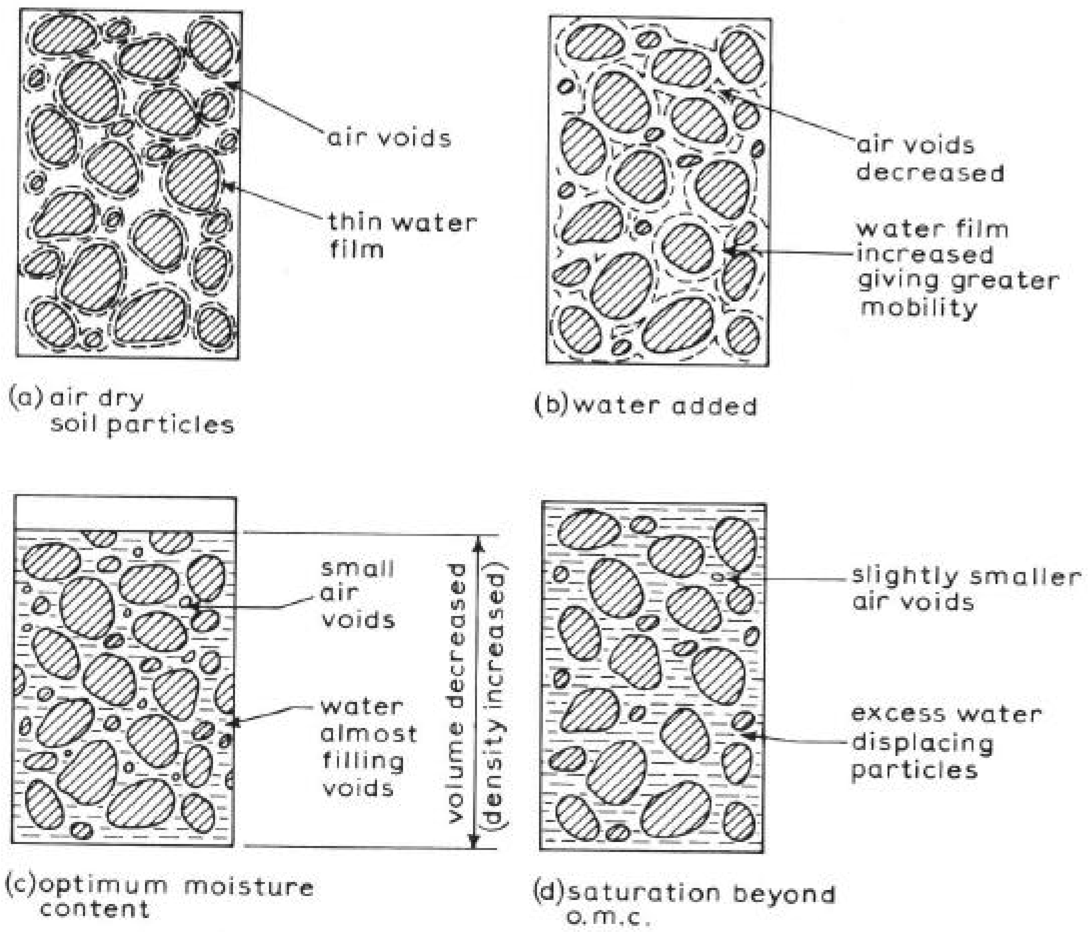
2. Principles of soil compaction

Soil generally consists of a mix of solid, liquid and gas. These are more commonly referred to as the soil particles, water and air. The combination of the volume occupied by the water and the air is called the void volume. Compaction of a soil sample is done to decrease the air voids present in the soil and hence increase the dry density of the sample. Dynamic compaction achieves this by permitting a moving mass to strike the surface of the soil sample and deliver energy into the sample that causes densification. The level of densification that can be achieved relates to a number of different parameters, the most important of which are the moisture content and the compacting energy transferred. Other factors that affect the densification are the number of blows applied to the soil and the momentum of each blow delivered by the falling mass.

2.1 Air void reduction

An air dry mass of soil will have a certain amount of spaces between the soil particles and these spaces are referred to as "air voids". This is sometimes expressed as a percentage of the total volume (air + soil) occupied by the air. Indirectly it can be represented by the "dry density" of the soil, as the weight of air in a soil is negligible compared with the weight of the soil particles. If a soil sample is compacted at it's density-optimum moisture content, by definition it will be at its greatest dry density for that compacting pressure. After such compaction, the volume occupied by the moisture will be virtually equal to the percentage of air voids present in the sample after subsequent drying out. Incidentally the density-optimum moisture content is not the same as the strength-optimum content. We must not use volumetric definition of OMC as it changes (rises) during the compaction process. We use a mass definition. Alternatively we use volume but define when it is measured, e.g. immediately after compaction.

The density-optimum moisture content (OMC) depends on the compacting energy delivered into the sample. The greater the compacting energy the lower the OMC and hence the greater the final dry density. The diagram below taken from Head, 1980, pg. 270, illustrates the particle arrangement of a soil sample at different moisture contents as well as the OMC.



2.2 Compacting methods

Several methods for dynamically compacting a soil sample exist as tests for soil compactability. These involve a mass that is raised to a consistent height above the surface of a soil sample constrained within the walls of a mould. Some impactor designs cover the entire area of the soil sample whilst others are dropped over the surface in a standard pattern. The latter technique could be analogous to tamping the soil down into a block mould, whilst the former is like the dynamic compaction tests as done by Gooding. Both tests are of interest but the former will be more helpful when trying to extend Gooding's research.

2.2.1 Soil compaction tests

The complete description of all the possible compaction tests is not necessary for the purposes of this report. A brief outline of each test is given and their possible relevance to the dynamic compaction research will be suggested. The tests described below are taken from Head, 1980, pg. 281-306.

BS Ordinary Test (or the Proctor test)

This test uses a 2.5 kg metal rammer with a 50mm diameter face that falls into a cylindrical mould of 105mm diameter. The drop height is kept at a constant 300mm to ensure consistent energy transfer between blows. The blows follow a pattern over the face of the sample to ensure repeatability and consistent compaction of the entire sample. Each sample made up of three layers of soil that has passed through a 20mm sieve and each layer is given 27 blows of the rammer. After compaction the sample is trimmed off to a set height that gives a constant volume of 1000cm^3. This is then weighed and the density can be calculated.

BS Heavy Test

This test is virtually identical to the BS Ordinary Test, with the only difference being the mass of the rammer and the drop height. For this test a 4.5 kg rammer is used and it is dropped from a constant height of 450mm above the level of the soil. Compaction is also carried out in five layers instead of three. All other dimensions and quantities remain the same.

Compaction by Vibration

This test uses an electric vibrating hammer operating at a frequency of between 25-45 Hz and a power consumption of 600-700 W. The soil is compacted in a cylindrical mould with an internal diameter of 152mm and a height of 127mm (CBR mould). The vibration from the hammer is transferred into the soil through a steel rod with a circular foot 145mm in diameter, (i.e. that nearly fills the mould). The soil is compacted in three layers by the hammer action and a steady force of 300-400N is applied to the vibrating hammer to prevent it from bouncing up and down on the surface of the soil. The final compacted height is measured using a steel ruler. The mass of the soil and mould is then weighed and weight of the empty mould subtracted from it. From these measurements of height and net weight the density can be calculated.

Dietert Compaction

Of all the compacting methods this one is most similar to the tests done by Gooding. It is a hand-operated device that uses a large cam to lift a mass of about 8kg through a constant height above the surface of the soil. The cam permits the mass to be dropped repeatedly onto a foot that rests on the surface of the soil sample transferring the energy into the soil and causing compaction. This apparatus uses a standard 50mm mould and the foot is fractionally smaller (48mm) to ensure free movement on impact. Density is calculated from measuring the height of the soil in the mould and the mass of the soil that is originally placed into the mould; the number of blows applied is recorded.

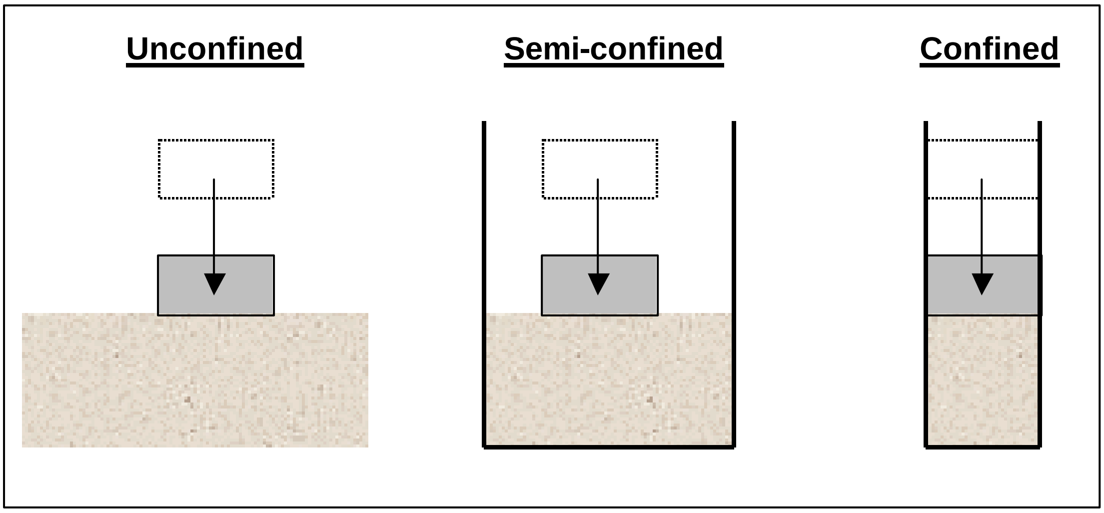
Harvard miniature compaction

In the situation where material for analysis is scarce and the soil particles are finely grained this test may be used. It uses a hand-held spring-loaded tamper and a special mould. The spring ensures that a consistent force is applied to the surface of the soil during each successive 'tamp'. This force equates to 178N and is applied through a tamper rod of 12.7mm in diameter over the surface of the soil. The mould is 33.3mm in diameter and 71.5mm high. This volume yields the useful feature that the mass of soil, in grams, is equal to its density in pounds per cubic foot.

2.2.2 Compaction test analysis

Both the BS Ordinary test and the BS Heavy compaction test show similarities to the compaction process that is of interest because they involve a mass dropping onto the surface of the soil in a mould. To compact the soil sample evenly the rammer must be dropped in a pattern over the surface of the soil. Although the soil is restrained within the sides of the stiff-sided mould it is only semi-confined to a volume. In other words, compaction applied to one area doesn't cause compaction in another and slip planes within the soil can exist. Conversely, confined compaction is similar to the Dietert compaction where the compaction occurs over the entire surface of the soil in the mould, thereby confining the volume and restricting any slip planes in the soil. Both of these compaction methods are very different to the unconfined ground compaction as used in civil engineering.

Now we can separate out any compaction test into three classes groups: confined, semi-confined and unconfined compaction. Of the three, confined compaction is of most interest as it replicates the dynamic compacting process that will be employed for block manufacture during this project. Semi-confined and unconfined compaction may be useful to investigate, but will be limited in their application to this project. Below is a sketch to illustrate the three classes of compaction.



Unconfined compaction is limited to ground compaction as used in civil engineering and no compaction tests have been described above for this case. Semi-confined compaction tests are BS Ordinary and Heavy tests as well as the Harvard miniature compaction test. Confined tests are the Dietert and the Vibrating Hammer compaction tests, although the latter uses a different means of transferring the compaction energy.

It is not advisable to compare compaction methods that use vibration with impact compaction. Vibration expels air from the mixture and does not usually crush soil samples in any way. Instead vibration redistributes particles (largest ones sink) and it does not leave compressed air pockets.

3. Previous dynamic compaction research

It has been suggested already that the information on dynamic compaction of stabilised soil blocks is very scarce. Up till now the author is only aware of two pieces of work that cover this topic, and only one of which he has been able to access. There are however, other publications that deal with the subject of soil compaction, both from a theoretical and practical viewpoint.

3.1 Mathematical modelling

In the unconfined state, a soil sample that receives an impact will compress in the localised area and send shock waves through the surrounding soil. It can be modelled as a highly damped spring with characteristics that depend on the Young's Modulus, Dilation Velocity, Poisson's Ratio, and Elastic Limit of the soil. Scott R. A. and Pearce R. W. give an equation that links these characteristics to the rate of deceleration of a moving mass in order to model the stress and movement at the impact surface.

Scott and Pearce 1976 modelled an unconfined mass of soil that has been hit by a falling weight. They investigate the effect of unsaturated and saturated soils monitoring the elastic properties, surface deflection and stress concentrations. They also suggest a model for a one-dimensional situation that may be analogous to dynamic compaction within a constrained mould. Below is an extract from their paper as found on pg. 23-26 of *GROUND TREATMENT BY DEEP COMPACTION*.

> "Loose unsaturated soils subject to steady localised surface loading deform typically as shown by the curve A of Fig. 2. The deformation is of a generally elastic nature at low stress levels and at these stresses the soils can propagate seismic waves. With increasing stress the slope of the deformation curve falls more or less sharply due to the relative ease with which voids can be collapsed at the higher stress levels.
> If such a soil is subjected to impact by a fast falling weight, the soil rigidity may play a much less important role than the soil inertia in controlling the deceleration of the weight and in absorbing the energy of the impact. An idealised representation of a compactable soil in respect of these inertial and energy consuming effects in the elasto-plastic soil is represented by the curve B of Fig. 2. The stress level of the plateau has been chosen to lie in the region of the reduced slope.

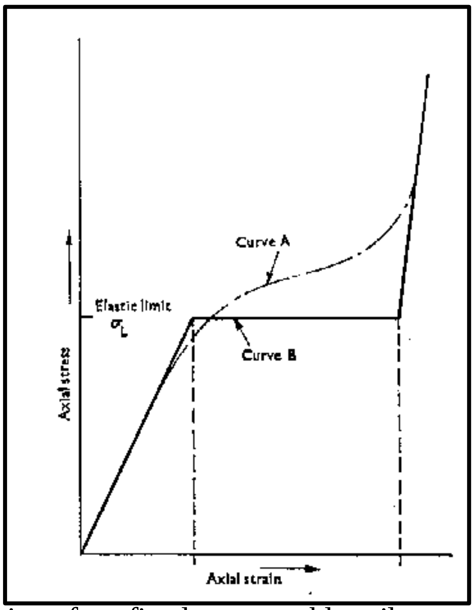


Fig. 2. Axial deformation of confined compactable soil

A three-dimensional treatment of the reaction of the soil underlying the contact is impracticable as the strains are generally so large that the shear restraints due to flanking regions of soil are not easy to quantify.
However, when the impact momentum is high the weight will punch through the upper soil layers and carry down a growing zone of compacted material of a generally cylindrical shape. For present purposes of illustration we shall discount the inevitable lateral spread of the compacted zone and use a one-dimensional description based on the approach mapped out for example by Salvadori (1960).
Immediately upon impact the stress level rises because of stress wave reaction due to the elastic nature of the first small movements of the soil at the contact surface. When the stress level has reached the level σ_L of the plateau, the soil particles at the surface have acquired a velocity v associated with a radiating stress wave which travels downwards into the medium with the seismic dilation velocity c appropriate to initial elasticity. The wave is accompanied by a pressure front in which the axial stress is given by a form of equation (1) that is,

$$s_L = rcv$$

The radiation of the stress wave is followed almost immediately by a further acceleration of the surface particles such as to bring the surface to the same instantaneous velocity V as the weight.
If z is the instantaneous position of the front of the steadily lengthening compacted material (Fig. 3) the retarding stress applied at the bottom surface of the weight is

$$-m\frac{d}{dt}(\dot{u}-v) = r_c\frac{d}{dt}\left[(z-u)\frac{d}{dt}(u-vt)\right] + s_L \qquad (8)$$

where m is written for the ratio M/pa^2 and ρ_c is the compacted density. The distances z and u can be shown to be related by the expression $z=k(u-vt)+vt$ where $k=rc/(rc-r)$. This relation can be used to eliminate z in equation (8), with the result that

$$-m\frac{d}{dt}(\dot{u}-v) + k_r\frac{d}{dt}\left[(u-vt)\frac{d}{dt}(u-vt)\right] + s_L = 0 \qquad (9)$$

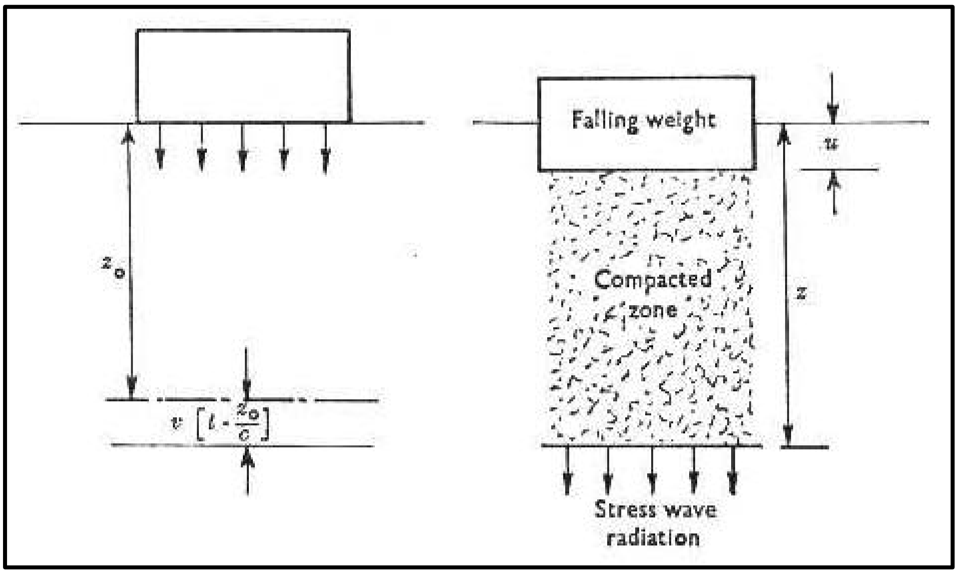


Fig. 3. One-dimensional compaction

The displacement u of the surface is obtained by solving equation (9) hence

$$u = vt + m(F-1)/k_r \qquad (10)$$

where

$$F = \left(1 + \frac{2k_r}{m}t - \frac{\sigma_L}{m^2}t^2\right)^{1/2}$$

The surface stress in the soil is then given by

$$\sigma = \frac{\sigma_L + k_r(V-v)^2}{F^3} \qquad (11)$$

Surface motion ceases after a time given by $t=m(V-v)/\sigma_L$ and at this time the final depth h of the compacted zone is given by evaluating $(z-u)$ and therefore by

$$h = \frac{m}{\rho_c}\left\{\left[1 + \frac{k_r(V-v)^2}{\sigma_L}\right]^{1/2} - 1\right\} \qquad (12)$$

It should be observed that while the stress just ahead of the compaction zone is at the elastic limit stress σ_L the stress at the surface may be considerably higher, especially at the early stages of compaction."

The author does not confess to understand all of the above nor what approximations have been made to develop such a model. Further reference to other source texts will be attempted to try and establish an appropriate model for a fully constrained soil. This model would then need to be checked with actual readings taken from the dynamic compaction process to verify its consistency. Both of these have yet to be done, but they are included in the scope of this project.

Another theoretical analysis of the impact method was found in Parsons pg 199 as follows:

"Theoretical analysis of the factors influencing the performance of dropping-weight compactors

12.27 To give an indication of the important factors to be considered in the design of impact compactors in general, and dropping-weight compactors in particular, Lewis (1957) produced a simplified theoretical analysis of the impact pressures produced on the surface of soil by a rammer. The experimental dropping-weight compactor shown in Plates 12.3 and 12.4 was used to verify the theoretical analysis.

12.28 From the well known equations of motion:-

$$V^2 = 2fx \qquad (1)$$

And $pA = Mf \qquad (2)$

where V = velocity of rammer on impact
f = deceleration of rammer on striking soil
x = deformation of soil during impact
p = pressure generated on surface of soil by the impact
A = area of rammer base
M = mass of rammer

Hence:- $$p = \left(\frac{Mk_s V^2}{2A}\right) \qquad (3)$$

where $k_s = \frac{p}{x}$

=dynamic modulus of deformation of the soil

In the case of a rammer falling freely from a height h:-

$$p = \sqrt{\frac{Mhgk_s}{A}} \qquad (4)$$

If the acceleration of the falling weight is less than g as a result of frictional losses:-

$$p = \sqrt{\frac{Mhg' k_s}{A}} \qquad (5)$$

where g' = actual acceleration of the falling rammer.

$$p = \sqrt{E_s k_s} \qquad (6)$$

where E_S = specific energy

12.29 These relations indicate that the impact pressure is a function of the energy per unit area of the rammer base (specific energy) and the deformation properties of the soil under dynamic conditions of loading. The latter factor is also likely to be a function to some extent of the area and shape of the rammer base, but little information was available on that aspect at the time that the analysis was made. If it is assumed that the dynamic modulus of deformation behaves similarly to the static modulus of deformation in that the modulus is often found to be inversely proportional to the square root of the loaded area, then:-

$$k_s = \frac{C}{\sqrt{A}} \qquad (7)$$

where C is a constant

The expression for the impact pressure developed can then be written:-

$$p = \sqrt{\frac{MV^2C}{A2\sqrt{A}}} \qquad (8)$$

where C = constant for the particular soil conditions.

Thus, if the rammer area is changed, the compaction energy provided by each blow per unit area of rammer base (specific energy) $\left(\frac{1}{2}M\frac{V^2}{A}\right)$ would have to be kept proportional to the square root of the area of the rammer base $\left(\sqrt{A}\right)$ for a constant pressure to be developed."

The author can apply these formulae to his results from the dynamic compaction of full-sized blocks that was done in 1997. The table below shows the increase in energy that was delivered by the impactor as the soil block was compacted. It also indicates the total transfer of energy into the block after a certain number of blows.

Impactor stroke (m)	0.1364	0.1571	0.1661	0.1748	0.1814	0.1866	0.1913
Energy(J) / blow		55.5	58.7	61.7	64.0	65.9	67.5
Energy increase		7.3	3.2	3.1	2.3	1.9	1.6
Energy transferred after blows (J)	0 blows 0	1 blows 55.5	2 blows 104	4 blows 221	8 blows 468	16 blows 980	32 blows 2035

Between the initial resting place of the impactor and the resting place after one blow there is a distance of (0.1571 – 0.1364) = 0.0207m. This is the deformation of the soil during impact (x). The velocity of the impactor prior to impact can be assumed to be $V = \sqrt{2gh} = \sqrt{2 \times 9.81 \times 0.1364}$ = 1.64 m/s …etc.

Below is a calculation table with the rest of the calculations for multiple blows during a compaction cycle using the above formulae.

	1 blow	2 blows	(3 blows)	(4 blows)	(8 blows)	(16 blows)	(32 blows)
Velocity prior to final impact (m/s)	1.64	1.76	1.81	1.83	1.88	1.91	1.94
Stopping distance (m)	0.0207	0.0090	0.0043	0.0044	0.0016	0.0006	0.0003
Mean deceleration (m/s²)	64.6	171	375	384	1070	2800	6380
Calculated stopping time (s)	0.025	0.010	0.005	0.005	0.0018	0.0007	0.0003
Pressure generated (MPa)	0.057	0.152	0.332	0.341	0.948	2.488	5.656
Dynamic mod of deformation	2.768E+6	1.687E+7	7.635E+7	7.835E+7	5.743E+8	3.828E+09	1.925E+10
Mean force in tonnes (final impact)	0.233	0.616	1.35	1.38	3.85	10.1	23.0

N.B. The velocities and stopping distances for the blow numbers in brackets have been linearly estimated from compaction data for multiple blows. These figures are probably accurate to ±10% and despite not being spot on experimentally they do show the continued trend.

Two things are immediately obvious from the table of results above. Firstly, the dramatic increase in force that is applied during impact between the first blow and much later ones. Secondly, the dynamic modulus of deformation for a soil compacted in a confined manner will increase as it becomes compacted. Therefore the characteristics and behaviour of the soil will change during the compaction process. This will make accurate modelling the compaction significantly more difficult than an unconfined soil with a constant dynamic modulus of deformation.

Another thing to consider from these results is the magnitude of the force that can be delivered using a bigger dynamic compaction machine. For example: a 50kg impactor with a maximum velocity of 2m/s stopping in 0.0001m will deliver an instantaneous force of 100 metric tonnes! Delivering forces of this magnitude will necessitate a secure foundation for the machine, perhaps even larger than originally anticipated.

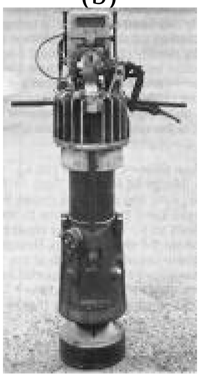
3.2 Dynamic compacting equipment as used in civil engineering

Within the field of civil engineering there are many different types of equipment that have the capacity of compacting a mass of soil. Many of these will not be of interest as they possess very little dynamic properties that help to compact the soil. Even smooth vibrating rollers and vibrating sheep's foot rollers are outside of the field of interest as compaction via vibration is quite different to dynamic compaction.

Of the remaining equipment that is regularly used in civil engineering there are no devices that compact soil in a confined fashion. At a stretch of the imagination, one could say that some pneumatic and power rammers could be classed as being semi-confined if they were compacting soil in a trench. The dynamic compaction equipment almost always compacts the soil in an unconfined state, and there are several examples of these that can be looked at.

(a)

(b)



(c)

(d)

(e)

Vibro-tampers

These devices are essentially an engine driven reciprocating rammer that bounces up and down on the surface of the soil with its location controlled by an operator. They range from 50 – 150 kg in weight and vibrate at a frequency of around 10 Hz. The amplitude of vibration can vary depending on the machine anywhere between 10 – 80 mm. A picture of a Vibro-tamper can be seen in part (a) of the above diagram.

Power rammers

A controlled explosion of a petrol/air mixture is used to force a piston ground-wards. This causes the power rammer to jump up into the air compressing the soil beneath it and compacting the soil on its descent. A photo of a power rammer can be seen in part (b) of the above diagram, and a power rammer in use can be seen in part (c). Power rammers typically have a mass of about 100 kg with a circular base of about 250 mm in diameter. These rammers are manually controlled and guided around the ground surface. They jump between 300 – 360 mm into the air and deliver a blow of between 315 – 370 J/blow. This equates to an energy transfer per unit area of compacting base of between 6.3 – 8.1 kJ/m².

A much larger variety of power rammer is the frog rammer, typically around 600 kg with a 750 mm compacting base. This machine ‘hops’ along the surface of the soil compacting it with each ‘hop’. It also moves forward with each ‘hop’ in order to reduce the directive force required by the operator. The operator turns the rammer into the direction that (s)he wants it to travel and the rammer hops along in that direction. Must be a fascinating machine to watch! Although this machine delivers 1835 J/blow it delivers a smaller 4.3 kJ/m² than the other type of rammers.

Multi-dropping weight compactor

A picture of this machine is included in part (d) of the diagram above. The unit is towed behind a suitable traction unit and is designed to provide adequate compaction in a single pass over the surface. It uses an arrangement of six 200 kg cast iron weights that are lifted and dropped onto the surface of the soil by rotating cams driven by an on board diesel engine. Each weight is lifted through 330 mm and delivers around 515 J/blow. The base of the rammers are 330 ×305 mm and therefore have a specific energy of about 5.1 kJ/m².

Mobile dropping-weight compactor

This machine is called the Arrow D500 dropping-weight compactor and is self propelled with a hydraulically lifted impactor at the front of the machine. A picture of the machine can be seen in part (e) of the above diagram. This device can lift the impactor through a variable height up to a maximum of 2.2 m. A 36 kW diesel engine drives a pump for the hydraulic system to lift the 588 kg mass to the desired height. This can then deliver a maximum of 11167 J/blow, and with a 305 × 305 mm base this equates to a considerable specific energy of 120 kJ/m^2.

All of the above information is taken from research carried out at the Transport Research Laboratory as reported by A. W. Parsons 1992. At TRL tests were carried out using the above machines on different types of soil and their different compaction abilities were noted. Some of the different types of soil that were used were; heavy clay, sandy clay, well-graded sand, gravel-sand-clay and silty clay. Different machines within the same class of compactors were assessed relative to each other in the different soil types. TRL also developed an experimental falling weight compactor that was used to help determine the efficiency of the other falling weight compactors that were available.

3.3 Research done by Gooding for his PhD

This source of information has proved to be highly valuable in the planning of future research in this field. Gooding has been the sole available reference for dynamically compacted soil samples that are compacted in a confined manner. Although Gooding thoroughly investigated the dynamic compacting process, he didn't actually stabilise any of the dynamically compacted samples with cement. The characteristics and effectiveness of the combined processes was not looked into. Other samples were stabilised using both compaction and cement but in these circumstances quasi-static compaction was always used.

3.3.1 Quasi-static compaction

Before Gooding began to investigate dynamic compaction, he looked into the process of quasi-static compaction (i.e. pressing). His research included varying the cement content, the applied pressure, mould taper, double and single sided compaction, pressure cycling and mould wall roughness. Throughout his tests he used a fabricated soil called *soil A* with a constant moisture content of 8%.

Gooding looked at the relationship between pressure verses wet compressive strength, cement content verses wet compressive strength and developed a model to estimate the wet compressive strength of a sample with known cement content and applied pressure. This model was based on actual experimental results taken from tests carried out using a range of pressures and cement contents. A small cylindrical mould specified in BS1924 was used for all of these tests. All the cylinders had their wet compressive strength tested after seven-day curing and subsequent soaking for 16 hours.

The model that Gooding developed suggests that a sample of *soil-A* with 5% cement and a compaction pressure of 10 MPa should have a wet compressive strength of around 1.6 MPa. Initial tests by the author using the Bre-pack machine have yielded blocks with compressive strengths of slightly less than this value, (1.5 MPa for a block with 4.9% moisture content). This apparent similarity has to be discounted for two reasons. Firstly, the test specimen the author used was a 100 mm cube instead of a 50 mm cylinder. And secondly, the difference in moisture contents would lead to considerably different results. Whereas Gooding was able to test the compressive strengths of the finished cylinders, the author found it more advantageous to cut the full size blocks into two 100 mm cubes. This resulted in generating two tests for the same block and it also uses a standard sample size, as used in the concrete testing procedures.

3.3.2 Dynamic compaction

Gooding investigated the efficiency of impact compaction using <u>un</u>stabilised *soil – A*. Consequently the wet compressive strength of compacted stabilised soil samples could not be measured as unstabilised soil breaks down when immersed in water. Instead each sample received the same energy but by different impact arrangements and the achieved density was measured. Density was calculated by measuring the final cylinder height (±0.05mm) and mass (±0.1g) on ejection from the mould. Each cylinder received a constant 279 J/kg and the mass of each cylinder was kept at around 1.66 kg. Other factors such as the number of blows and momentum of impactor were varied to find any optimum parameters for this technique.

Each sample received one of 1, 2, 4, 8, 16, 32 or 64 blows. The optimum number of blows (number that yielded the greatest density) was found to be at 16 blows, but it was also noted that only a 3-4% reduction in compaction efficiency occurred when this was varied from 8 to 32 blows for each of the different masses.

If different number of blows and different masses were used to compact the samples then the height through which each mass was lifted had to be varied. A lighter mass had to be raised higher to transfer the same energy per blow as a heavier mass dropped from a lower height. Similarly, if less blows were being applied then the mass had to be raised higher to transfer the same total energy. This has the effect of changing the momentum of each blow applied as momentum depends on the mass and the velocity of the mass prior to impact and velocity depends on the distance through which the mass falls. Three different masses were used in the experiments on the samples (23.35, 35.00, 46.80 kg) and it was noted that the bigger masses dropping at slower speeds were more effective. Yet, the 23.35 kg mass and the 35.00 kg mass were only 0.4% and 0.2% less efficient respectively at the 16 blow configuration than the 46.80 kg mass.

This area needs to be further investigated using cement and doing proper compressive tests to suggest better accuracy for the environment in which the samples will finally be placed.

3.3.3 Other research that was done

Gooding 1995 was involved in producing "Survey of the potential for cement-stabilised building blocks as a building material in developing countries". During this field survey of many countries he encountered a couple of structures that were made out of cement stabilised dynamically compacted material. He compares them with other structures in the area, constructed using similar appropriate techniques, with some interesting observations. Below is an extract from that survey, pg 58 covering Botswana.

> One soil-cement house is of particular interest. In 1985 a soil-cement house was constructed at the Camphill Community Centre in Otse using the Ranko Block Maker. This is a manual machine which uses impact to compact the soil-cement to high pressure. It was designed by Agas Groth, a Botswanan national. The house has now been standing for ten years without any maintenance work having been carried out and is in excellent condition. The blocks were produced with a cement ratio of 1:16 and having been well cured and laid in the wall were rendered with a low-cement bagwash. This should be compared with houses constructed by BTC for their experimental staff housing project using imported quasi-static machinery; the Hydraform and Ceratec machines which cost 60,000 P (£14,000) and 100,000 P (£24,000) respectively. The blocks were produced with a cement ratio of 1:10 and powered mechanical soil sieving and mixing were used. These houses have now been standing for only two years but are already deteriorating. In the case of the Ranko block walling production was estimated to cost between 20 and 30 % less than the prevailing price for sandcrete blocks (Enyatseng 1987). In comparison the blocks produced using the Ceratec machine were found to be 18% more expensive than stock cement bricks and 46% more than sandcrete blocks (BTC 1995). The high cost of the Ceratec blocks was attributed to the low productivity of the machine. Although this machine was capable of producing 1200 blocks/hour this figure was never achieved as two motorized mixers would have been required to continuously supply the machine with soil. If a lower cost machine were available, capable of high pressure compaction but with a useable maximum output then the economics of production would be significantly improved.

The author is currently trying to get a copy of the research work done by Agas Groth to compare it with Gooding's investigations.

3.4 The author's previous research

As part of an undergraduate degree programme the author had to do some research on a subject that was suggested by one of the resident lecturers. The author discovered that Gooding had a small project that would be suitable both for the project requirements and for the author's abilities. This project was subsequently undertaken and labelled "Design and realisation of a test rig to research the production of full size dynamically compacted soil-cement blocks". This project was completed in 1997 and achieved the following results. A full size dynamic compaction test rig was designed and manufactured. The design chosen was suited to the level of appropriate technology available in developing countries. Several blocks were produced and their densities and surface resistance was measured. Two blocks were stabilised using cement, but these were not used in the experimentation as they were only intended to be demonstrator blocks. This means that up to date there has not been any research done on dynamically compacted cement stabilised soil blocks.

Gooding quasi-statically compressed a block to 9.7 MPa and noted that it achieved a density of 2038 kg/m³. This compaction pressure equated to a transfer of 279 J/kg. By comparison, the author dynamically compacted a full size block to a density of 2040 kg/m³ by applying 32 blows to it from a 36 kg impactor. This block received a total of 2035 J from the falling impactor. For a 10-kg block this equates to approximately 204 J/kg, some 26% less energy required than the quasi-statically compressed block, which is still a significant saving. This research indicated that the savings in energy that Gooding had found could be extrapolated onto full size blocks and warranted further research.

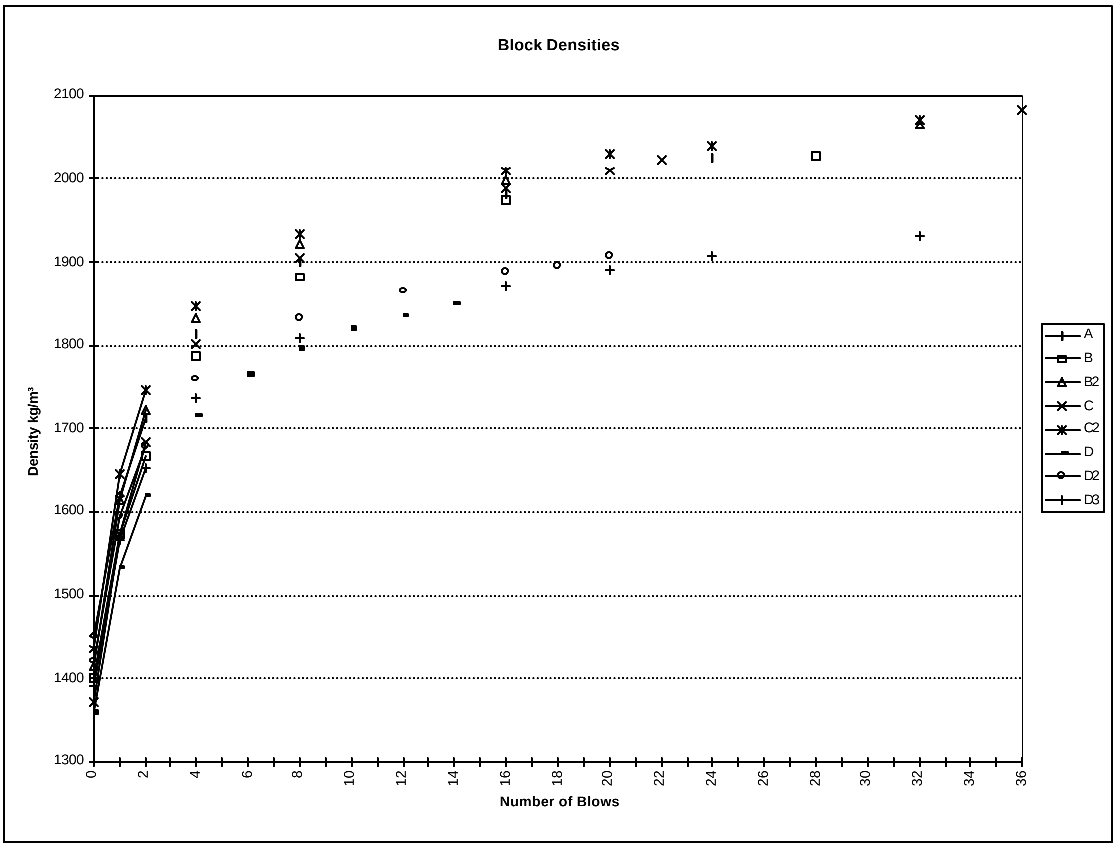
The author also did not stabilise any of the full size dynamically compacted blocks as these were trials to test the feasibility of full size compaction. Consequently there are not any known characteristics of the produced blocks apart from a handful of penetrometer tests done on the freshly demoulded block. These give little indication of the core strength and only sought to establish the level of uniformity of density throughout the block.

4. Discussion of research

The experiments done by the Research Transport Laboratory, Gooding and the author can in some way be compared with each other. The experiments described in section 3.2 can be compared to the tests carried out by Gooding, but only with the soil that is closest to the soil used by Gooding during his research, which are the sandy clays. Even this soil has a much higher percentage of clay than the soil used by Gooding, but the other soils are vastly different. It can be noted from these compaction results that greatest compaction was achieved with the experimental rig when it delivered 4, 5 or 7 blows with the same total energy transfer at the optimum moisture content as discovered by the 2.5 kg rammer test (described in section 2.2.1). The compaction was about 4% better in this configuration than the big multi-weight machine (described earlier), and about 8% better than the experimental rig delivering 2 blows, (40% of the energy as transferred compared to the 4,5 or 7 blow arrangement).

The author during his previous research also noted the slight reduction in compaction from a massive reduction in energy transfer. The graph below shows several blocks that were made by dynamic impact. Each blow had approximately equal energy after the first few blows so 40% energy of a block that received 32 blows should equate to about 12 blows. Block C2 achieved a density of around 1975 kg/m³ after 12 blows, but its density only increased to 2070 kg/m³ after a further 20 blows. Thus a decrease of 60% in energy transfer only led to a decrease in density of less than 5%.

However a small drop in density can have a significant effect on the final compressive strength of a compacted block. From Gooding's research it can be noted that a cylinder stabilised with 5% cement that was compacted to a density of 2124 kg/m³ achieved a cured wet compressive strength of 1.63MPa. Another cylinder compacted to 2032 kg/m³ (a drop of less than 5% in density) only yielded a cured wet compressive strength of 1.20MPa, (a drop of over 25% in strength). This trend of high gains/losses of strength for small increases/decreases in density fits throughout the results that Gooding received from his experiments.



Graph showing block density against number of blows received

From the above results that have been highlighted for comparison, there are a few trends that can be noted and will help in further research. Final cured strength of cement stabilised blocks is highly dependent on the final compacted density. It is also true that small changes in density can only be achieved by much greater changes in energy transferred into the block. Dynamic compaction has proved to be a more efficient compaction process than quasi-static and it also has the added advantage that it is relatively easy to increase the energy transfer by simply applying more blows.

Any quasi-static compaction machine will have a working limit and will be unable to compress to a higher compaction pressure than that. Gooding suggested that pressure cycling would yield a small increase in final density and subsequently a higher strength, pg. 137, but this is time consuming and is still highly limited. Dynamic compaction would only be limited by the time required to produce each block, and even then the impact time could be reduced by modifying the machine design. Dynamic compaction, therefore, has a much greater potential for increasing the energy transfer and consequently increasing density and final cured strength.

Furthermore there is agreement among all the sources that compaction via multiple blows is more effective than with a single or a few larger blows. This characteristic is highly advantageous with dynamic compaction as larger numbers of blows can deliver the same energy into a block as a much larger impactor falling from a greater height. This method of energy transfer is much easier to design into a machine than a very large compactor falling through a great height.

5. Conclusions and recommendations

There is still more information that needs to be found and investigated. This will continue throughout the project and will be written up in due course. Several other sources are already being sought and they will help to shed new light on this relatively undocumented field of research.

The limitations of the existing information are significant and these need to be tackled during the project if a better understanding of the dynamic process is to be achieved. Dynamic compaction of cement stabilised samples needs to be undertaken, both for cylinders and for full size blocks. These will need to be tested according to the seven day wet compressive strength test and their performance noted. It is known that dynamic compaction provides better and more efficient compaction, but it is not clear if these will in turn reap significant benefits when the addition of a cement stabiliser is included in the stabilisation process. Will a dynamically compacted and quasi-statically formed block perform similarly if they both achieve the same density and contain the same amount of stabiliser?

In order to achieve a higher density a significant amount of extra energy has to be transferred into the block. Reducing this energy transfer, or changing the way it is transferred has a marked effect on the final density that can be achieved. Small changes in density have large repercussions with other important characteristics of the finished block, such as the compressive strength and porosity. Consequently the greatest factor in the production of a cement-stabilised block is the final density and maximising this characteristic should be done in wherever possible. If cement is the expensive commodity and this has been reduced to an absolute minimum, then the application of extra energy in the most efficient manner is surely justifiable.

Optimisation of the number of blows for small cylindrical samples was done by Gooding for a constant moisture content. This optimisation needs to be extrapolated onto full size blocks to determine if there are any better arrangements for delivering a fixed amount of energy into a sample of stabilised soil. The moisture content has not been altered with respect to the cement content and this may be of significance. A lower moisture content may not yield the block with the greatest density, but it may yield a block with a higher wet compressive strength and durability. Parsons reported small changes in moisture content around the optimum to try and discover if the different compacting method yielded a different optimum moisture content. A similar exercise needs to be done with confined stabilised soil samples.

Gooding never used cement to stabilise his dynamically compacted cylinders and consequently nothing is known of the effect that the presence of cement has on the compacting process. It has been suggested in that cement will hinder the compacting process when the cement crystals are forming. Furthermore, compaction of the soil during crystal growth will be detrimental to final strength as bonds that have already formed will be broken and will need to be reformed again. It is the author's experience with the two dynamically compacted blocks that were stabilised with cement that slightly lower densities were achieved using similar compaction regimes. It was also noted that the ejection of the block from the mould was considerably more difficult than with the blocks formed without the use of cement.

6. Summary

The dearth of information on dynamically compacted soil blocks has only been a preliminary setback for the purposes of this research project. In one sense it gives complete freedom to explore any other area of the field that may be of interest as very little has been done before. The specific areas that have been covered provide adequate information and analysis and leave the author in no doubt of their accuracy. These areas do not need to be covered again, but they need extending to include other areas within the research field.

Dynamic compaction has been studied mainly for use within the civil engineering industry for ground compaction. This research gives helpful pointers to the behaviour of soil when it is compacted by an impact blow and also provides examples of equipment that are used within the industry. This research does not fit the same model as the fully constrained soil that would be used for dynamic compaction of soil blocks, but much of the data for impact delivery, energy transfer and soil deformation can be applied to this situation.

The understanding of what happens to the soil during an impact blow is still in infancy. It is dangerous to assume linear deceleration during the impact as the calculations in the latter part of section 3.1. This is probably not the case as the soil will act as a highly damped spring with variable damping and spring constants. A thorough investigation of the actual energy dissipation and resistive forces applied by the soil on impact may not be possible within the scope of this project. It would be good to know a bit more about this mechanism and the author intends to try and work this out, but he feels that the substance contained with such a study may warrant the commitment of a whole project on its own.

Bibliography

Gooding, D.E.M., *Improved processes for the production of Soil-Cement Building Blocks*, 1994, PhD Thesis.
{Loc: M.R.C. ([res] DIS 1994 345)}.

Gooding, D.E.M., Thomas, T.H., *The potential for Cement-Stabilised Building Blocks as an Urban Building Material in Developing Countries*, 1995, The Overseas Development Administration (Project No. D141).
{Loc: Contact T.H. Thomas at University of Warwick}

Head, K.H., Manual of Soil Laboratory Testing Volume 1: Soil classification and compaction tests, 1980, Pentech Press.
{Loc: U.W.C.L. (QC 137.7 H3)}.

Montgomery, D.E., *Design and Realisation of a Test Rig to Research the Production of Full Size Dynamically Compacted Soil-Cement Blocks*, 1997, University of Warwick 3rd Year project undergraduate report.
{Loc: Author's personal copy}

Parsons, A.W., *Compaction of Soils and Granular Materials: A Review of Research Performed at the Transport Research Laboratory*, 1992, HMSO.
{Loc: U.W.C.L. ([qto]TE 208 S2)}.

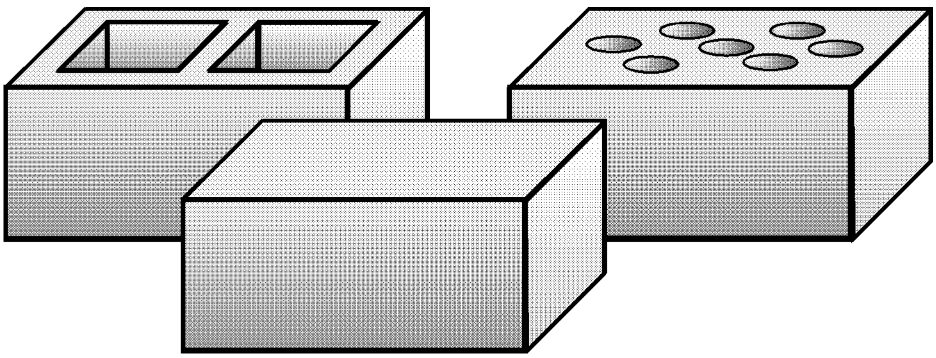
Scott, R.A., Pearce, R.W., *Soil compaction by impact*, 1976, Institution of Civil Engineers: Symposium report.
{Loc: U.W.C.L. (TA 775 G7)}.

Development Technology Unit
School of Engineering
University of Warwick
Coventry CV4 7AL UK
Tel (Office) +44/0 1203 522339
Director 523122, Fax 418922
Email dtu@eng.warwick.ac.uk

DTU

Web site at http://www.eng.warwick.ac.uk/DTU/

Stabilised Soil Research Progress Report SSRPR8



Minimising the cement requirement of stabilised soil block walling

Author: Mr D E Montgomery & Dr T H Thomas
March 2001

E-mail: d.e.montgomery@warwick.ac.uk

These reports cover 'work in progress' by research students in the Development Technology Unit (DTU) of the School of Engineering at Warwick University. Their primary purpose is internal - a format for recording ideas and data in a way that allows them to be better discussed before their incorporation into theses, DTU Working Papers or external publications. However they also have a secondary purpose, that of facilitating the sharing of our research with other innovators in the field of building with stabilised soil. Each report, after some initial internal discussion and refining will be posted as a title and synopsis on the DTU web pages (home page= http://www.eng.warwick.ac.uk/dtu). Full copies can be obtained from the respective named authors.

Titles of Stabilised Soil Research Progress Reports Produced to date:

[Put printed list of current reports in place of this page.]

A dedication to someone special

Sometimes at the beginning of a publication one finds a dedication to a certain person or member of the family who has been an influence in the author's life either in general or specifically in generating the work in question. There is one person in my life that immediately springs to mind who is worthy of such a dedication. Furthermore, my experience with this person is not unique as millions of others have found him to be a great inspiration, comfort, guide and friend. "What's his name?" you may be asking yourself and, "Why haven't I heard of this incredibly influential person". The sad thing is that you probably have, but you have never accepted him as such or welcomed him into your heart and life. Well, now you have an opportunity to do just that. Please read on.

The man's name is Jesus and although he was born nearly 2000 years ago his testimony still remains and his power to save is just as great. "Save from what?" you may ask, sin and the consequences thereof, or more specifically, your sins and the consequences you face when you die. As humans we demand justice to be done, and justice will be done, but on a perfect scale and to a perfect standard. That leaves us all falling short and without hope when we come face to face with a holy God. But, God in his great love towards us send his only begotten Son into the world that the world through him might be saved. Jesus Christ died for you so that you would not have to be punished for what you have done wrong. You can be spared eternal punishment in hell and enjoy love and peace in the presence of God forever. Today the choice is yours. Reject God's free gift of love at your peril, accept it and who knows you too may have the joy of writing a dedication such as this someday. Please ponder the verses below and make your choice carefully, it will be the most important decision you ever make.

David E. Montgomery

"*For by grace are ye saved through faith; and that not of yourselves: it is the gift of God: not of works, lest any man should boast.*" Ephesians 2:8,9.

"*For God so loved the world, that he gave his only begotten Son, that whosoever believeth in him should not perish, but have everlasting life.*" John 3:16.

"*For whosoever shall call upon the name of the Lord shall be saved.*" Romans 10:13

"*He that believeth on him is not condemned: be he that believeth not is condemned already, because he hath not believed in the name of the only begotten Son of God.*" John 3:18.

"*Jesus saith unto him, I am the way, the truth, and the life: no man commeth unto the Father, but by me.*" John 14:6.

Abstract

The monetary cost of low-cost walling in developing countries is greatly dependent on the expensive additives that are used to manufacture the building units and the cost of transportation of raw materials or finished products to the site of construction. Another cost associated with the production of anything is an energy cost and that can give an approximate overall measure of environmental impact. Within this paper several different types of existing walling materials are investigated for their overall cement and energy consumption. The purpose is to see how favourably they compare with high-density compressed and stabilised soil blocks using these suitable comparative measures. Assessment of suitability of local and on-site production will also be indicated for each of the materials in this study.

The study indicates that only three of the materials examined utilise less than 15kg/m² of cement, two of those are unsuitable for local production and the third uses about three times the energy in production. High-density compressed and stabilised soil blocks use slightly more than 15kg/m² of cement but have a low energy requirement for production. The other sections of this paper deal with the possible methods of further reducing the cement requirement of high-density compressed and stabilised soil blocks to a value below 15kg/m².

Several different cement-reducing methods are outlined within this paper. These include: placing voids in the block, incorporation of a cement rich-skin (either within the block itself or applied as a render), interlocking blocks requiring very little or no mortar and taller blocks that reduce the number of block courses needed for mortaring. In isolation each method does not reduce the cement demand below 15kg/m². However, it is possible to apply several of these methods together that safely brings the cement requirements to well below the target of 15kg/m² with a low energy cost.

Nomenclature

Brick: An object (usually of fired clay) used in construction, usually of retangular shape, whose largest dimension does not exceed 300mm.

Block: A larger type of brick not necessarily made of fired clay, but stabilised in some way, sometimes with central cores removed to reduce the weight.

Cement: Ordinary Portland Cement (OPC).

Clay: The finest of the particles found in soil, usually of less than 0.002mm in size and possesses significant cohesive properties.

Concrete: The finished form of a mixture of cement, sand, aggregate and water.

Dynamic Compaction: A process that densifies soil by applying a series of impact blows to it.

Fines: General category of silts and clays.

Green Strength: The strength present in a freshly formed block prior to curing.

Sand: A mixture of rock particles ranging from 0.06mm to 2 mm in diameter.

Silt: Moderately fine particles of rock from 0.002mm to 0.06mm in size.

Soil: Material found on the surface of the earth not bigger than 20mm in size, not including rocks and boulders and predominantly non-organic. If soil is to be used for building material it must not contain any organic material and it can be a natural selection of particles or a mixture of different soils to attain a more suitable particle distribution.

Stabilised soil: Soil which has been stabilised (treated to improve structural characteristics) by using one or more of the following stabilisation techniques: mechanical, chemical and physical.

Table of Contents

1. Introduction

Cement (opc) is the normal material used to stabilise soil in compressed block walling. It gives them a ‘wet strength’ they would otherwise lack. Other stabilisers are possible, but few meet the requirement of being readily and economically available in the target area for low-cost house walling, namely developing countries. However work at Warwick on micro-silica (both in its classical form and as a product of low temperature rice-husk processing) has led us to investigate its advantages as an opc additive in block-making. Interestingly, at clay contents below 15% Kaolin equivalent, lime has not proved to be a useful substitute for opc in soil block manufacture.

Cement is expensive in some countries (e.g. over $0.2 per kg in Uganda) and the ratio of (50kg) cement cost to daily wage exceeds 5 in most developing countries. It is currently uneconomic to use much cement – say more than 15 kg cement per m^2 of walling. Additives like micro-silica, while they are only used to substitute a small part of the opc, are considerably more expensive per kg and therefore even more restricted in their concentrations.

If we take as a norm a wall thickness of 140 mm, and assume mortaring consumes 30% of the available cement, then we are restricted to about 4% opc by weight within the blocks, or less than 4% if a costly additive is included. Even with very high moulding pressures (10 MPa), or high impact energies, it is difficult to produce really durable blocks with so little stabiliser.

There are however some paths we might follow that would allow us to use denser stabilisation without exceeding this cost target (of 15kg opc per m^2). One is to produce hollow or indented blocks that use less material per unit area of walling. Saving 50% of the material would allow a doubling of the cement:soil ratio. A second path is to employ non-homogenous material, increasing the concentration of stabiliser in the block faces (where deterioration is focussed) and reducing it in the block interior. A third path is to reserve much of the cement for a render, placed over hardly-stabilised blocks. A fourth is to employ dimensionally tight interlocking blocks requiring little mortar to lay.

The purpose of this Research Progress Report is to discuss the advantages, disadvantages and practical implications of following each of these paths.

2. Summary of existing materials for building

In this document we cannot provide an exhaustive list of building materials, just some of the more popular methods of providing walling at tolerable cost. Hollow and aerated concrete blocks, clamp and kiln fired brick and compressed and stabilised soil blocks (hereafter CSSB) are the main materials for consideration. Some of these materials require a thicker level of mortar to compensate for the irregularities of the blocks. Furthermore certain materials need further protection from the elements if they are to last for tolerable periods and this is usually done by applying a render to the external face of the building. Sometimes this is only done for visual reasons, but for the purposes of this investigation we will assume that aesthetics are not the primary concern and certainly not worth extra expense.

Possibly one of the most striking differences between different types of building materials is their width. Some concrete hollow blocks are 250mm (10”) wide whist the clay fired brick is usually only 103mm (4”) wide. A wider block is more stable and can be used to build taller walls with a high slenderness ratio, (width/height). A single skin 103mm wall is not considered to be stable enough except for in-fill walling between columns and beams or for relatively small structures. In our analysis of single skin brick construction we have included a buttress pillar of two bricks at 1-metre centres, which increases the brick and material requirement by 25%. It is more common to make a single skin brick wall of closer to 150mm (6”) thick and this practice can be extended to two storey construction.

2.1 Hollow concrete blocks

These are expensive due to their need for graded sand and large amounts of cement (12-17% by weight). If manufactured properly they can have a very high strength and have excellent durability. Cost reduction is achieved by removing material from the block core thus making it lighter as well. Machinery for production requires a vibrating table to settle the cement mix into the mould. Sometimes a heavy hinged lid slammed a couple of times or low pressures are applied to compress the material.

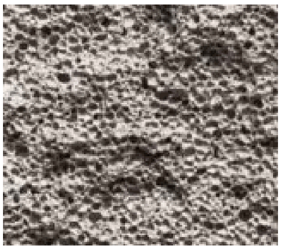
High-pressure compaction of these blocks is highly uncommon and is well out of the scope of low-cost building materials.

Good dimensional accuracy means that these blocks can be laid on a 10mm mortar joint. However, due to the voids in the block much mortar falls down these holes and is wasted. (In calculating the required mortar we have assumed that the mortar actually used is closer to the total surface area of the entire top surface of the block rather than just the edges where a joint is made with the neighbouring block.) These blocks are sometimes rendered for aesthetic reasons, which we will omit from any calculations for the time being.

2.2 Aerated concrete blocks

Aerated concrete is a much lighter form of concrete that omits the use of coarse aggregate and includes a high percentage of air voids in the material. A cement rich mixture has a foaming agent applied to it before the material is pumped or can be cast into suitable moulds (Neville, 1995). This material has been developed into a high performance building material and is currently marketed as aerated concrete blocks (Thermalite, 2001). The large proportion of air within the block reduces the density to around 500kg/m^3.

Although these blocks are not considered suitable for heavy-load bearing conditions, (over 7MPa), they are highly favourable to low-rise structures such as typical homes. Other features such as high wall area per block, low thermal conductivity, easily shaped by hand tools and low moisture penetration make this a highly attractive material. The production costs are reasonable as the main ingredient is coal ash from power stations, (which itself is a pozzolanic material that helps the cementitic process), but the complexity of the process makes it relatively unsuitable for small-scale manufacture. Moreover coal-burning power stations are not present in all countries

A – Aircrete B – Thermalite block C – Thin mortar joints

The above photographs show the structure of aircrete (A), it's ease of handling (B) and the high dimensional accuracy required for thin mortar joints (C). The textured surface of the blocks help to bond the to the block mortar, (if desired as it is not necessary on external walling).

2.3 Kiln fired brick

Over the centuries the process of burning clay to make brick has become more and more automated, sophisticated and complex, but not necessarily more cost effective, particularly in developing countries. (Parry, 1979) very eloquently and persuasively describes two methods of brick production in terms of cost and shows quite clearly that where labour costs are low, kiln-fired brick production would be unsuitable. Kiln-fired brick production requires a high capital investment and a significant amount of infrastructure to support production. A greater degree of material selection must be employed, staff needs to be highly skilled, spares and servicing is highly specialised and energy requirements are considerable. Production output is very high, typically 10,000 - 30,000 bricks per day and needs to be continuous if to achieve high efficiency and to achieve the greatest return on investment.

The characteristics of such kiln-fired bricks are highly desirable as the material has a high wet-compressive strength and does not deteriorate rapidly over time even in the harshest of climates (Hanson, 2001). The material is pleasing to the eye and is sought after as an attractive material for home building.

2.4 Clamp fired brick

Can be inexpensive in monetary terms because the raw materials are dug from the ground and the energy required firing the brick could come from collected firewood.

Clay fired blocks need good sources of clay for production and like graded sand must be obtained from a suitable source nearby. Forming the blocks requires a wooden or metal mould and after forming they are laid out to dry. After drying they are stacked into a clamp where fires are burnt inside (Parry, 1979). These fires raise the temperature of the blocks to the point where the particles bond together (Stulz & Mukerji, 1993). Thorough burning is necessary to fire all the blocks properly and this takes several days to achieve. The finished blocks can be quite badly misshapen and this requires a thick layer of mortar between the blocks, sometimes as thick as 20mm. Furthermore, if the blocks are poorly fired then in order to achieve adequate durability they may need to be rendered as well. Fired blocks are considered attractive and so they are not generally rendered unless necessary.

This is a particularly poor example of clamp-fired bricks and thick poorly used mortar. The result is unattractive and wasteful of cement.

However, due to the high cement content of the wall and the fired brick used it will probably achieve adequate durability.

The poor dimensional accuracy of the bricks can be clearly seen in this photograph.

2.5 Compressed and Stabilised Soil Blocks

These blocks use the same parent material as unstabilised mud but offer the significant advantage of wet compressive strength. One of the methods of stabilisation is to compact a soil sample to reduce the voids in the finished block. Compaction is achieved by applying some mechanical effort to the soil, which in turn drives out some of the air voids. Increasing the density of the material gives it a higher compressive strength and also reduces the potential for ingress of moisture into the block (Houben & Guillaud, 1989), (Norton, 1997). CSSB are further stabilised with the addition of a chemical stabiliser that helps to bind the particles together. Cement or lime are expensive additives but are generally available and although the practice of adding them to soil is reasonably popular the results can be disappointing unless it is done carefully.

Here is a good example of a wall made of stabilised soil blocks.

The blocks are approx. 0.4 × 0.2 × 0.125m and may have some voids through the centre. No render has been applied to the wall and no significant roof eaves have been used.

A solid cement rich foundation had been used to build the blocks onto. This is a high quality construction and would have been quite costly but not as much as hollow cement blocks.

CSSB can be compacted using low or high-pressures or dynamically compressed using falling weights. The greater the level of compaction the greater the compressive strength of the block and the more effective any added stabiliser becomes, (Gooding, 1993). CSSB compacted to higher densities are also usually more dimensionally consistent and therefore can be laid using a thinner mortar layer of around 10 – 15mm. Some CSSB need to be rendered in order to enhance the protection from the elements, but this can be avoided with higher levels of compaction and or higher quantities of stabiliser. Making a hollow CSSB can be done by straight-through perforations or deep and shallow frogs (Houben & Guillaud, 1989). Each of these reduces the material present and therefore reduces the stabiliser quantity necessary for each block. Removal of material from the core must be done carefully as it decreases the maximum supportive load of the blocks.

3. Criteria for comparing walling materials and assessment of current materials

There are a number of criteria we could use for comparing walling materials. For our present purposes we would like to hold ‘performance’ constant so that we can meaningfully compare some of the costs of production. Whilst different building materials have can have very different characteristics, we can suggest a minimum standard that all the materials must comply with. We have therefore chosen the following performance specification:

- bulk wet crushing strength = 1.5 MPa
- exterior surface wet crushing strength = 3.0 MPa

Blocks with this performance should be wholly adequate for low-rise housing construction up to a roof-ridge height of 8 meters (for which the bottom-course pressure < 0.15 MPa). It has been suggested that blocks that have a wet compressive strength of over 3.0 MPa can be used in tropical environments without the need for external render. We will therefore consider that a block with a similar surface strength will exhibit adequate durability for most circumstances.

Market cost is the most familiar criterion for materials comparison, but is not easy to use in situations where part of the building process is performed within the subsistence economy. A fairly universally applicable measure of resource-use in walling is ‘primary energy consumed per square meter’. This is the sum of primary energy required in the extraction and manufacture of the materials, in their transportation to site and in their final erection on site. However as the last item is comparatively small and also very difficult to estimate, we have chosen to neglect it. Transport energy obviously depends upon distance and we have chosen, for various reasons, to estimate distance as 25% of the mean spacing between points of production in one country (namely Uganda), i.e. 100km for cement and less for other materials.

For those types of walling for which cement (opc) is the main bonding agent, or is the only purchased material, cement content provides another comparative measure. The energy required to produce the cement will also be included in the energy calculations. The cement literature suggests that the energy requirements for material extraction, processing, firing and grinding for cement production is approximately 6MJ/kg.

For low-cost housing in developing countries, there is an additional criterion for comparing materials. It is their ease of access (geographical or socio-economic) to potential users. Thus a material that can sensibly be manufactured 'locally' – say on a scale of under 10,000 m^2 walling per year – is more likely to be available in an area of poor transport, and more likely to receive production investment, than a material requiring a trans-national scale of capital.

An even more severe constraint arises where the production of housing does not fall wholly within the monetary economy, i.e. where the tradition has been for householders to construct their own housing out of 'free' local materials. Actually few traditional materials meet the *wet* strength criteria listed above. However there remains a strong householder interest in making *some* use of local or on-site materials or of employing artisanal members of their own community in materials production.

3.1 Assessment results

We have therefore chosen to assess the most commonly used walling materials according to the four measures:

- Primary energy consumption in MJ per m^2 walling
- Cement usage in kg per m^2 walling
- Ranking for suitability for small-scale ('local') production
- Ranking for suitability for on-site production using mainly on-site materials

To limit the number of materials we have chosen those most prevalent in humid areas of East Africa and South Asia (excluding stone and timber walling) and added one high-tech material namely foamed PFA blocks. These are compared with two well-established variants of stabilised-soil blocks, namely low-density low-cement CSSB and high-density very-low-cement CSSB.

The table and associated notes below is a summary of a spreadsheet used to make the calculations.

Material	Dimensions (l x b x h)	Note	Energy	Cement	Suitability for production	
					'Locally'	On-site
	mm		MJ/m^2	kg/m^2	Ranking (1 = best)	
High-density CSSB	290 x 140 x 90	1	290	18.7	2	1
Low-density CSSB	290 x 140 x 90	2	420	34.1	1	1
Brick (kiln-fired)	215 x 105 x 65	3	430	8.1	2	3
Brick (clamp-fired)	215 x 105 x 65	4	1340	11.4	1	2
'Cement' block (hollow) N	300 x 150 x 200	5	430	27.0	1	2
'Cement' block (hollow) F	300 x 150 x 200	6	590	27.0	1	2
Foamed PFA-cement block	440 x 140 x 215	7	230	12.4	2	3

Notes

1. High-density (2000kg/m^3) solid blocks manufactured on-site from local soil/cement mix (5% cement), laid with 10 mm of soil/cement mortar (20% cement) and no render, (Cement transported 100km).
2. Low-density (1700kg/m^3) solid blocks manufactured on-site from local soil/cement mix (10% cement), laid with 15 mm of soil/cement mortar (20% cement) and 15mm render, (Cement transported 100km).
3. Kiln fired brick (3000MJ/1000 bricks) laid with 10 mm of sand/cement mortar (20% cement) and no render, double brick buttress column at 1m centres, (Cement transported 100km).
4. Clamp fired brick (16000MJ/1000 bricks) laid with 15 mm of soil/cement mortar (20% cement) and no render, wall has double brick buttress column at 1m centres, (Cement transported 100km).
5. Hollow (50% voids) cement blocks made from 10% cement mixed with gravel and sand from nearby source, with a 10mm mortar joint, (sand/cement, 4:1 ratio). Cement transported 100km.
6. Hollow (50% voids) cement blocks made from 15% cement mixed with gravel and sand transported from 50km away, with a 10mm mortar joint, (sand/cement, 4:1 ratio). Cement transported 100km.
7. High-tech aeration process using coal ash mixed with cement (15%) to make a very light (480kg/m^3) material. Laid with a 3mm mortar joint using cement rich paste (50% cement). Blocks transported 50km.

Of the materials listed above only three of them use less than the desired 15kg of cement per m^2 of walling, two of which are unsuitable for local production and the third has an extravagant energy requirement. High-density CSSB is the only material that uses a modest amount of cement, a low energy requirement and is suitable for local and on-site production. The following chapters will discuss other methods that may further reduce the cement requirement of High-density CSSB to less than the desired 15kg per m^2 of walling.

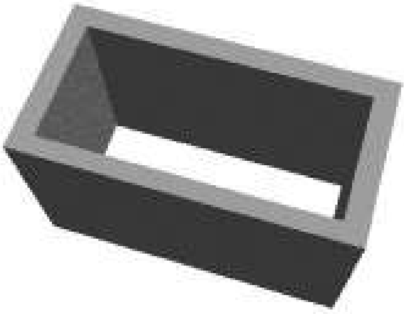
4. Perforated and indented blocks

When considering to place indentations or to perforate a block it is a good idea to determine the reasons behind existing block shapes to see what can be learnt from them. There are a huge variety of block shapes and sizes available and we shall not investigate them all. For the purposes of this study we will limit the possible shape of the blocks to those that specifically remove material from the block core to reduce material. These will include perforated blocks, deep and shallow frog indentations.

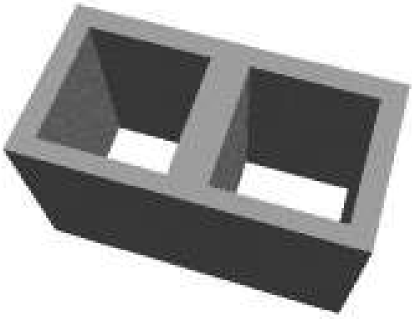
The design of a block can vary a great deal depending on the application. The standard clay fired brick includes a shallow frog that aids the process of keying the brick into the underlying mortar. This purpose of the frog is not really to reduce the overall material of the brick, but this is a beneficial result of the technique. In a similar fashion hollow concrete blocks are hollow typically for two reasons. Firstly due to their size a solid block would be much too heavy for easy manual movement and placement in a wall. Secondly the hollowness of the blocks permits the inclusion of reinforcement for more massive structures to gain sufficient strength even in areas with seismic activity.

In order to remove significant amounts of material from the centre regions of a block there must be sufficient block width to accommodate the voids left behind. Also the minimum material thickness needs to be carefully chosen so that the material does not become too weak to support the necessary loads. The drawback to including any perforations or voids in a block is that it increases the mould complexity and reduces the ease of block manufacture, particularly block ejection.

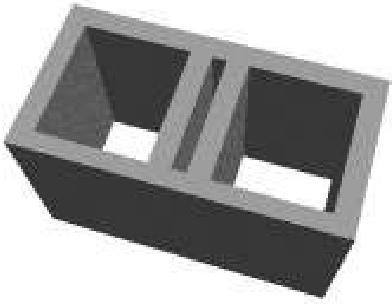
Below are a series of images depicting different types of concrete blocks with different shapes.



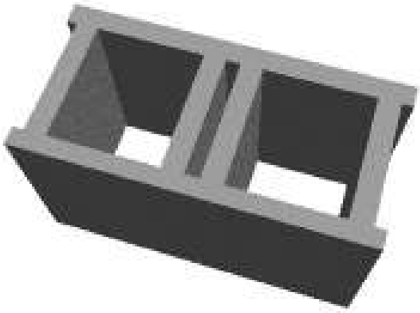
A



B



C



D

<u>NOTES</u>

Block A has the least material but mortar joints on the top and bottom surfaces are limited to the front and back face.

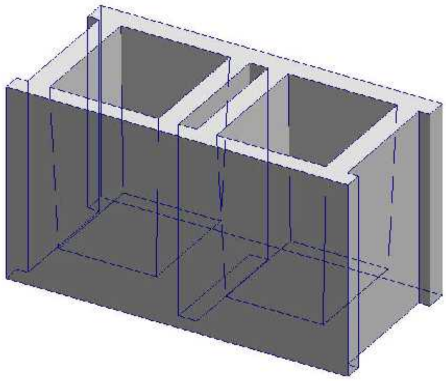
Block B overcomes this problem by putting a central web so that block tessellation can occur and good mortar contact is achieved.

Block C incorporates a double web so that the block can be more easily split into halves for wall ends etc.

Block D adds a few further flanges on the ends of the block to reduce the mortar contact area and also to help with more accurate tessellation of the blocks.

With all of the above blocks there is a significant problem with mortar falling into the holes when it is laid. A better block would have a flat surface onto which a thin layer of mortar could be placed. This idea follows the deep frog concept where a significant internal void is achieved but without going though the entire block.

An even better example of block is like the one shown below, where the internal voids don’t go all the way through. The thin lines indicate the outline of the material, more clearly showing the internal voids.



If mortar is being used to join the blocks together then a deep frog arrangement is better than the hollow section as less mortar is wasted. Chapter five of this document deals with the proposal of interlocking blocks that have no need of mortar between the courses.

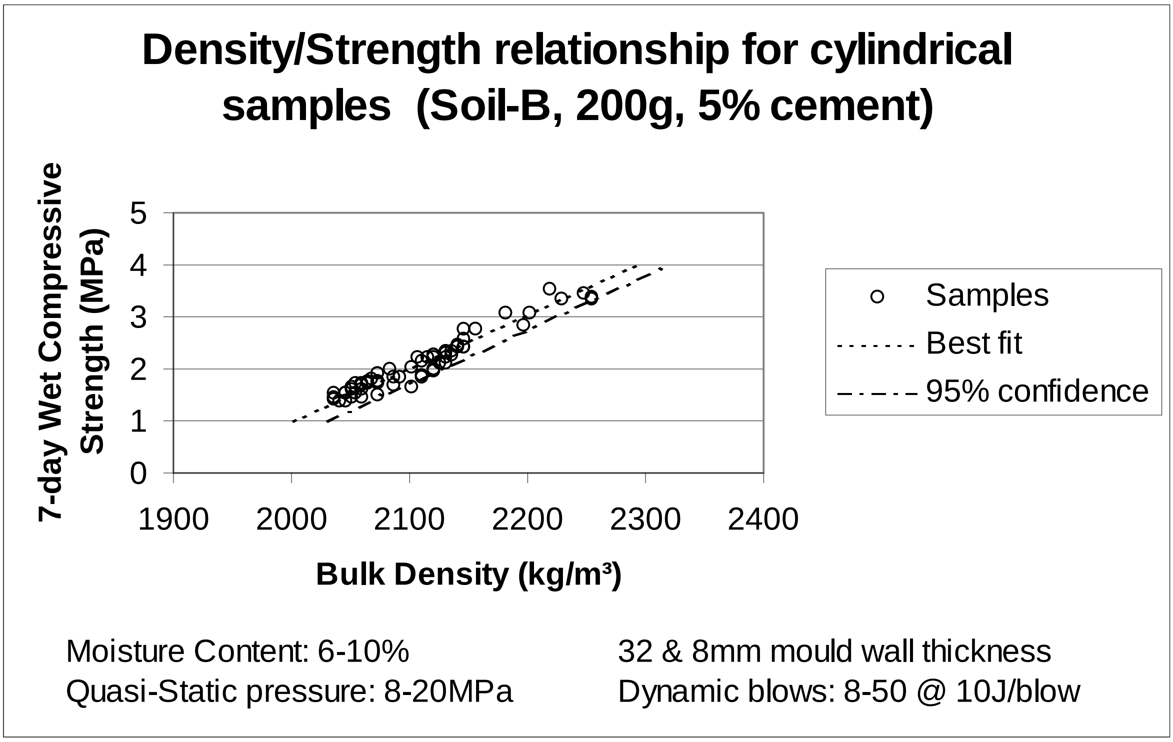
4.1 Results of material removal

In the production of low-cost building materials cost reduction is paramount, and if the cost can be reduced without jeopardising the strength of the material beyond acceptable limits then this would be a significant advantage. One obvious way of achieving this cost reduction is to reduce the amount of raw materials that are necessary to make the block, or more specifically to reduce the expensive raw materials that are necessary to make the block, i.e. opc. If the material mass of each block could be reduced by 50% then that would constitute a saving of opc of 50% in the block itself. If the material strength remained the same then the maximum load that could be applied to the block would also be approximately 50% less.

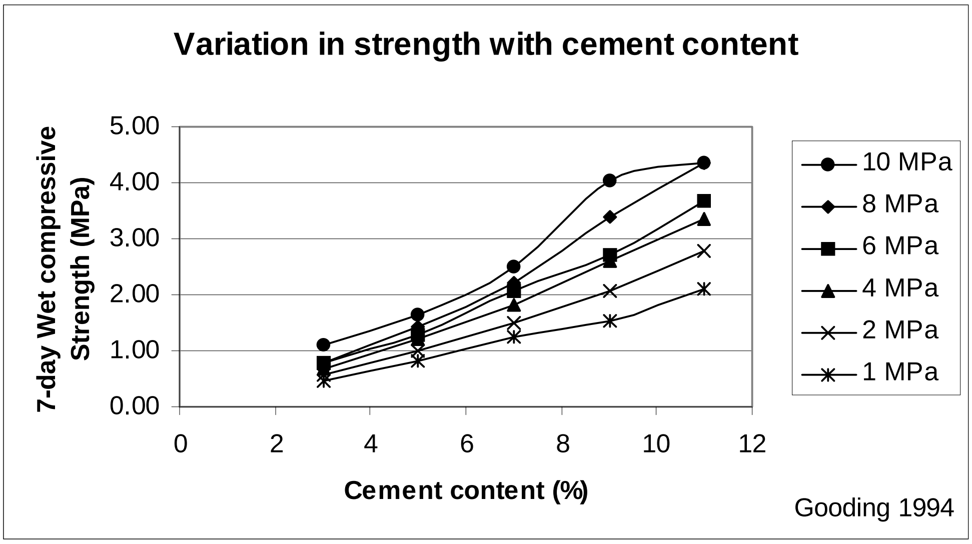
Now that we have seen a number of different types of hollow and indented blocks, we now need to assess the effectiveness and notice any implications that the addition of indentations will have. Indentations will quite clearly remove material from the core of the block and therefore reduce the amount of material required to produce each block. This removal of material also reduces the maximum load of the block itself and this should be taken into account when designing the structure.

Strengths of materials are usually given in compressive strength terms in MPa or equivalent N/mm^2. Thus block strength is directly proportional to the surface area on the compression face. In the case of a hollow block removing 50% material reduces the compressive surface area by 50%. This means that the same material will only be able to support 50% of the load. Fortunately the reduction of material from the cores reduces the mass of the block so that the mass of walling is also reduced and therefore a similar height wall can still be accommodated. In order for the block to support the same load it will need to have an increased compressive strength.

Significant indentations can only really be accommodated if the material strength is high enough and this may require the addition of more stabilisation, (compaction and/or opc). The strength of the material is dependent on the amount of compaction and the amount of opc present in the material. This relationship is not a linear one either for the compaction or for the cement. For a certain range of densities it has been found that an increase in density by 10% yields a 100% increase in compressive strength. Furthermore the doubling of the cement content has the effect of more than doubling the achieved strength of the material.



The above graph shows research results from the author's work that indicates the relationship between density and strength. It clearly shows that a small increase in bulk density can yield a significant increase in strength for the same cement content. Both the dynamic and quasi-static methods of compaction were used to make these samples the latter being much more difficult to increase the compactive effort if it is necessary.



This graph shows the change in compressive strength with extra pressure and extra cement. For the low pressure samples (1 and 2 MPa) as the cement content doubles the strength also doubles. For the higher pressure samples the fractional increase in strength for the same increase in cement is greater. This clearly indicates that the effectiveness of the cement present increases as the level of compaction is also increased.

With the combination of increasing the cement content and increasing the level of compaction it would be possible to remove significant amounts of material from the centre regions of a block without jeopardising the strength and gaining an overall reduction in costly opc.

4.2 Analysis of material removal

According the graph above a sample with 10% cement and compressed to 4MPa has a wet compressive strength of 3MPa. A standard block 0.29 × 0.14 × 0.09m and an approximate bulk density of 2060kg/m³ would have a cement mass of around 0.7kg present in it. If the level of compaction was increased to 10MPa the cement content drops to 8% to achieve the same 3MPa compressive strength. The same block has now reached a bulk density of 2160kg/m³ and would have a cement mass of around 0.58kg present in it. A more than two-fold increase in pressure results in only an 18% drop in cement content. This has already been shown to be a false economy in quasi-static compaction because this extra moulding pressure seriously increases the machinery rental and labour costs of production.

Now if half of the material present in the block is removed then the cement mass would naturally drop to 0.28kg per block which is less than half of the original value. This removal could be achieved by the inclusion of voids in the material. The higher density of the material would yield sufficient strength for forming and handling and whilst the absolute load that the block could sustain would be less, the compressive strength would still be within the required limits. This option would not be possible with blocks of lower densities, as they would not be strong enough to have such large voids placed in them and still keep strong enough for forming and handling.

4.3 Note on strength of rival walling materials

If house walls 'fail', it is usually by surface erosion, by overturning of by internal material changes like swelling. To prevent erosion we require adequate surface properties such as hardness or wet-compressive strength that are unaffected by whether or not the building blocks are hollow. To prevent overturning we look first to architectural measures such as providing adequate foundations, connecting perpendicular walls or constraining the outwards thrusts from roofs. However the block properties also affect a wall's ability to resist horizontal forces applied to its top. Increasing both block mean density (=ρ) and wall thickness (=Block width b) are beneficial. Although there are various overturning failure modes, almost all have a force threshold determined by ρb^2. For example the formation of a hinge at the wall bottom (assuming the mortar has no tensile strength) occurs when $F=\rho g b^2/2$ where F is the outward force per unit length of a wall.

The table below compares different materials by this criteria. Employing hollow blocks instead of solid ones lowers F because it lowers the mean block density ρ

Material	**Wall Thickness (b)**	**Mean Density (ρ)**	**Failure Force (F)**
	m	kg/m^3	N/m
Single skin brick	0.105	1350	74
Double skin brick	0.220	1350	327
Solid cement block	0.150	2200	248
Hollow (50%) cement block	0.150	1100	124
Foamed cement block	0.140	480	47
Low-density solid CSSB	0.140	1700	167
High-density solid CSSB	0.140	2000	196
High-density hollow (30%) CSSB	0.140	1400	137

5. Cement-rich skin

As mentioned in Chapter 3, there are several different methods that are under investigation for cement and energy reduction in the production of low-cost building materials. This chapter will briefly assess the method of putting the greatest stabilisation in the region of the block were it is needed most. This can be accomplished by either incorporating a cement-rich layer in the external face of every block, or by adding a cement render to the surface of a block which has a very low cement content.

5.1 Non-homogeneous blocks

As an alternative to reducing the cement content of the block to perilously low quantities, it may be possible to concentrate the cement in the area where it is needed most, i.e. the exterior surface. This cement rich layer would effectively be acting as a built in layer of render protecting the more fragile material behind it from the elements. For example instead of having 5% cement throughout the block one could put 10% cement in the first 20mm and have the rest of the block stabilised with only 3% cement. Providing that the cement rich layer did not suffer from de-lamination from the rest of the block, (which is doubtful if the block contains cement and the courses of blocks are joined with a cement based mortar), then this could reduce the cement demand for each block.

The production of such blocks with this cement rich layer greatly increases the complexity of the block production and construction processes. A very clear means of identification would be necessary to indicate which face of the block was cement rich, and furthermore the staff erecting the structure would need to be trained to lay the blocks in the correct manner. Homogenous blocks would also be necessary for the corners and any exposed edges, that adds another type of building material to the construction. The calculations carried out on this type of construction shows that the saving in cement is not terribly significant, (approximately 13%).

5.2 Rendered soil block construction

There are a number of reasons why a wall might be rendered. Leaving aside the aesthetic reasons, rendering is usually done to protect the walling from the elements or other forms of attack. Unstabilised walling has to be protected in humid areas and this can be achieved through the application of render. Cement-based renders do not work with an unstabilised soil wall as the coating is much too stiff to accommodate the movement of the soil wall as it absorbs and rejects moisture from the atmosphere. Lime-based renders are more suitable for this purpose. However, with more stable forms of walling, cement render is acceptable providing a good bond can be achieved to the surface of the block.

It may be possible to achieve a sufficiently high degree of surface hardness to negate the need of a render altogether as discussed earlier. This is indeed the most favourable option as the cement render is an expensive component of the walling construction if it has to be applied. In the application of render there are only really three different variables that are of interest if cost reduction is the main objective. They are the render thickness, the cement content of the render and the surface area that needs to be rendered.

The size and shape of the blocks under the render don’t have a direct effect on the quantity of render needed to cover the wall, (providing the external surface is flat). This is a pity, as larger blocks need less mortar per m^2 than smaller blocks. The same gains however are not achieved when it comes to rendering. If we assume that the thickness of the render has already been minimised and also the cement/lime content has also been minimised then the only variable left to work with is the surface area that the render covers.

A practise that has been used in developing countries is to restrict the rendering to the areas of the walling most prone to attack. This generally constitutes render application to the corners of the building and the first 300 to 600mm of walling above ground level. The vast majority of the walling is then left in barefaced brick.

This photograph shows an earth walled house with a limited amount of rendering at the lower level.

A small amount of render has also been applied to the corners of the building.

Notice too the significant roof eaves that have been constructed to protect the walls from precipitation.

If the cement was concentrated in the render on the external surface of the wall and a very small amount was used to stabilise the blocks behind the render, then this might provide a saving in cement use. If we assume that the entire wall needs to be rendered to achieve adequate durability then we can calculate the cement requirements for this type of construction. Unfortunately the calculations do not suggest that this is a favourable method of reducing the cement quantity need per square metre. Applying a 15mm render (20% cement) to a 3% cement stabilised block actually increases the cement used per m^2 by 3%. If portions of the wall could be protected by some other means and the area of rendering reduced, then this method might yield greater saving in cement use.

6. Mortar reduction methods

In the previous chapters we have seen some suggestions for minimising the cement content in the blocks and in the render, but we haven't yet discussed the mortar that joins the blocks together. The mortar used in wall construction can account for a significant part of the cement cost and if this can be minimised then this would be an added saving. Fortunately the mortar required is dependent on the size and shape of the blocks used to construct the wall and even in some circumstances the mortar may be omitted entirely like with interlocking blocks.

Mortar is necessary to carry out two basic functions, one is alignment and the other is cohesion. Due to the surface irregularities of blocks a certain amount of mortar is needed to ensure that the two faces of adjoining blocks sit well together and spread the load over the entire surface of the blocks. This layer of mortar also permits a degree of alignment so that the wall can be built to conform to a vertical face. The mortar between the blocks also creates a physical joint that will help to keep all the separate block units in the wall bonded together.

6.1 Variables affecting mortar quantity

Apart from making the mortar thinner one can reduce the mortar requirements by changing the size and shape of the block. These adjustments change the mortar requirements for each block, but also change the requirements per m^2 of walling. If the block size is increased then the mortar necessary per block increases, but the overall mortar requirement for the walling goes down. In order to determine the most important variables in wall production a spreadsheet was drawn up. It calculated the changes in cement demand for small changes in every variable that could be altered in block design and wall covering.

The table below shows all of these variables and the sensitivity that a change of that variable of ±10% gives to the overall cement requirement per square meter of walling.

	Cement (kg/m²)			
Variable	+10%	0%	-10%	Sensitivity
Block Length	34.5	34.5	34.6	0.02
Block Breadth *	37.6	34.5	31.4	-0.98
Block Height	34.3	34.5	34.7	0.06
Block Density *	36.9	34.5	32.1	-0.76
Cement content of block *	36.9	34.5	32.1	-0.76
Cement content of mortar	35.2	34.5	33.9	-0.19
Mortar Thickness	34.8	34.5	34.2	-0.08
Render Thickness	34.9	34.5	34.2	-0.11
Voids fraction of block *	32.1	34.5	36.9	0.76
Combination of four * variables	46.5	34.5	25.5	-0.89

The four main variables are 'block breadth', 'block density', 'cement content of material' and 'voids fraction'. Reducing the block breadth is not an option and we want to achieve the highest density possible to give the greatest strength.

Below is a table showing the four block variables that exhibit the greatest sensitivity to changes of 10% with the effect that each one has on the different areas of cement use, namely in the material, the mortar and the render.

	Cement mass required (kg)					Cement
Variable	Material	Mortar	Render	Total	Sensitivity	kg per m²
Standard block configuration	0.731	0.197	0.108	1.035	N/A	34.5
10% decrease in breadth	0.658	0.177	0.108	0.943	-0.98	31.4
10% decrease in material density	0.658	0.197	0.108	0.962	-0.76	32.1
10% decrease in cement in Mat.	0.658	0.197	0.108	0.962	-0.76	32.1
10% decrease in material (voids)	0.658	0.197	0.108	0.962	-0.76	32.1
Combination of all variants	0.479	0.177	0.108	0.764	-0.89	25.5

By concentrating on the variables that have a significant effect on the cement in the material and the mortar use we can suggest a combined scenario that may give a tolerable cement usage. A suggestion is shown in the table below.

	Cement mass required (kg)					Cement

Variable	Material	Mortar	Render	Total	kg/m²	Reduction (%)
Standard block configuration	0.731	0.197	0.108	1.035	34.5	N/A
25% increase in height	0.974	0.212	0.140	1.326	34.0	1
Removal of render	0.731	0.197	0.000	0.927	30.9	10
50% decrease in cement in Mat.	0.365	0.197	0.108	0.670	22.3	35
25% decrease in material (voids)	0.548	0.197	0.108	0.853	28.4	18
Combination of all variants	0.365	0.212	0.000	0.577	19.2	44

Instead of getting the cement per m² down to as low as 15 kg/m² the minimum suggested here is only 19.2kg.

Note:

Standard block to be considered as the following:

External dimensions (L × B × H) = 0.29 × 0.14 × 0.09m

Material = 2000kg/m³ (dry) with 10% cement, (NB different to Table in Ch 3)

Mortar = 1800kg/m³ (dry) with 20% cement and 0.01m thick

Render = 1800kg/m³ (dry) with 20% cement and 0.01m thick

Internal void volume = 0

This value for the cement demand per m² of walling is still too high. It is estimated that the Thermalite blocks described in Ch 2 use approximately 12.4kg of cement per m². If the mortar could be removed entirely from the example above then the total cement demand would reduce to 12.2kg per m², below our target cement consumption per square metre.

6.2 Mortarless blocks

Interlocking blocks have been available in a number of different styles for quite some time now. The designs differ but the basic principles are the same. Some form of indentation and protrusion on facing blocks form a mechanical link between the two building units. The difficulty with producing mortarless blocks is that you no longer have any freedom of adjustment during the laying process. Any alignment errors present in the lowest block course will be present in every subsequent layer on top. Not only does the bottom layer have to be set very accurately, but also every block must have a very high dimensional accuracy. An error of only 1mm in the height of a block between the internal and external face of a 150mm wide block will generate a vertical alignment error of 13mm at the top of a 2m wall. A 2mm error leads to 26mm, etc. Alignment errors approaching 25% of the block width would be of great concern as the wall stability is significantly reduced.

The other role that mortar plays is one of ensuring that the load is spread over the entire face of the block. Mortar removes any stress concentrations that would otherwise be there if the blocks were laid on top of one another. Even blocks with very high dimensional accuracy will still suffer from this. However, the process of production of stabilised soil blocks may offer a solution to both the problems of alignment and stress concentrations found in mortarless construction.

6.3 Block alignment while 'green'

When a cement-stabilised block is formed it has what is referred to as a "green strength". It is this strength that needs to be there for the block to be handled immediately after production. This strength enables the formed block to be moved from the production machine to a place of curing where blocks may even be stacked one on top of another to conserve curing floor area. Full strength of the material is not achieved for some time and the strength of the blocks at the time of production is a small fraction of the final compressive strength.

This low initial strength could be an advantage in mortarless construction. If the blocks are formed and placed directly into the wall then this may solve the two problems with mortarless construction. (The process of building walls from freshly made mud bricks is currently in use in the U.S. but these walls are not stabilised and an external render is applied to protect the bricks.) The construction of the wall in its green state enables a degree of flexibility with the material itself. As the material has a small amount of "give" to it, the different courses could be laid and as the blocks settle and begin to harden they will be taking the shape of their neighbours and therefore greatly reduce the chance of stress concentrations.

The other issue that has been raised is the one of alignment. If dimensional accuracy of ±0.5mm can be achieved then the maximum 'out of plumb' for a single storey wall would be a tolerable 13mm. However, this is a very high level of accuracy and probably not possible with CSSB production techniques. Blocks that are laid in their green state will accommodate a degree of manipulation and this may be all that is necessary to ensure that the blocks are being built in alignment with the vertical. Very slight adjustments could be made to the finished blocks during the construction process that would be impossible to do once the block has fully cured.

Even if removal of the mortar entirely is not a feasible option, the reduction of its thickness will generate great savings in cement. The thermalite blocks described in Ch 2 use a very thin mortar joint of around 3mm. The mortar is more of slurry consistency than a paste and is almost poured into position. Such a system could easily be incorporated into wall erection if the dimensional accuracy was as high as described able. Even a tolerable ±1.0mm error in the block height would still be able to use this very-thin mortar technique.

6.4 In wall curing

The process of building a wall of green uncured material generates a fresh problem of achieving a high strength successfully to an exposed material. Once the blocks are placed into the wall there is almost maximum exposure to the air and therefore to the blocks drying out. If cement is the stabiliser of choice then this drying out process must be hindered and even stopped if possible. While the blocks are on the ground or in tidy piles it is much easier to cover them and keep them moist than if they are already made into a wall.

Curing the blocks in the wall may be achievable if something could be draped over the wall that protects the block from the wind and the drying of the sun. If plastic sheets are used then this would be acceptable although would incur a greater cost even if the sheets are reused a number of times. Keeping the blocks under plastic sheets in the direct sun would have the effect of raising the temperature of the blocks and the cement within the block would achieve a higher strength faster then through normal temperature curing. This early higher strength caused by the higher temperatures would result in a slightly lower overall strength in the block once curing is finished.

The effect of temperature on pure cement is well documented and the following information just summarises some of these that may be of use in curing the blocks in the wall. The table below clearly shows that if a small reduction in final compressive strength can be accommodated then curing the material at a higher temperature can increase early strength. This may be a rather desirable side effect to construction in the humid tropics and one that should be exploited if possible.

Extracted from graph found in (Neville, 1995)

Time	Strength in MPa at curing temperature (°C)				
	20	35	50	65	80
6 hours	30	55	100	135	140
1 day	140	165	165	165	165
7 days	220	210	200	190	180
28 days	250	235	230	210	195

If three times the strength could be achieved in the material after 6 hours if it was being cured at 50°C then construction could proceed at a faster rate as well. The only problem is that we do not know if these results still apply to a material that has such a small amount of cement as would be used in CSSB production.

6.5 Tall thin blocks

The ratio of a block’s height to width is its’ slenderness ratio (height/width), (Norton, 1997), (Keable, 1996). Typically this slenderness ratio is not more than 1 but with some more advanced materials at can be as high as 2. If the height of the block is large then this will reduce the number of blocks necessary to fill the same area of walling. Another measure that we can have to assess the shape of a building block is the number of blocks required per square meter of walling. In order to maximise the use of the material therefore we want to have a high slenderness ratio and a large surface area of the external face of the block.

As mentioned in the pervious section the larger the external surface area of a block is the smaller the number of blocks needed per square metre and consequently less mortar is required. Increasing the height of the block therefore doesn’t so much as reduce the cement requirement per block as reduce the mortar requirement to lay the same area of walling.

In section 6.1 a suggested block arrangement was drawn up to try and reduce the overall amount of cement. One of the variables that were changed was the block height. This increased the amount of cement required in the material, mortar and render per block, but actually decreased the overall cement requirement per square metre. Although the decrease was quite small, if that is then applied to blocks with less cement in the material, laid with thinner mortar and without any render then significant savings can be made.

7. Conclusions and recommendations

The high-density compressed and stabilised soil block seems to be a reasonable contender in low-cost building materials. It requires less energy than all of the available competitors and slightly less cement than most of them. Variants on the CSSB can reduce the cement still further making it even more acceptable to a wider range financial capacity. Furthermore the ability for the CSSB to utilise local materials and be manufactured either on-site or very locally makes the material more suitable to cottage industries and self-build schemes.

The table below summarises the different possible variants that can be accomplished with the CSSB and how each one performs with reference to the unmodified CSSB. By combining several of these variants into a single block the material can theoretically achieve a tolerable cement requirement, (less than 15kg/m²), without excessive energy consumption. The tall, hollow, interlocking block as described below even uses less cement then the clamp fired bricks outlined in Ch 3. As this is one of the more common and more wasteful methods of making satisfactory building materials, this confirms that this variant of CSSB is a real contender.

Material	**Dimensions (*l* x *b* x *h*)**	**Note**	**Energy**	**Cement**	**Suitability for production**	
					'Locally'	**On-site**
High-density CSSB	Mm		MJ/m^2	kg/m^2	Ranking (1 = best)	
Normal	290 x 140 x 90	1	290	18.7	2	1
Hollow	290 x 140 x 90	2	220	15.1	2	1
Cement-rich skin	290 x 140 x 90	3	270	16.3	1	2
Interlocking	297 x 140 x 97	4	270	15.4	2	1
Tall	290 x 140 x 90	5	280	17.6	2	1
Rendered	290 x 140 x 140	6	300	19.3	2	1
Tall, Hollow, Interlocking	297 x 140 x 147	7	190	11.0	2	1

Notes

1. High-density (2000kg/m^3) solid blocks manufactured on-site from local soil/cement mix (5% cement), laid with 10 mm of soil/cement mortar (20% cement) and no render, (Cement transported 100km).
2. As 1. but with 30% material remove from the block core.
3. As 1. but with 10% cement in first 20mm of exterior block surface and 3% in the body of the block.
4. As 1. but with thin mortar of 3mm required.
5. As 1. but with increased block height to 140mm to reduce mortar per square metre.
6. As 1. but with 15mm render on a block with only 3% cement in the body of the block.
7. As 1. but with a combination of tall, hollow and interlocking arrangements.

Many different variants of the CSSB have already been successfully made. However, the author is not aware of any specific manufacturer that can produce the tall, hollow, interlocking CSSB variant that seems so frugal in its cement use. It is hoped that the application of compaction by impact can yield such a material without the addition of expensive machinery, but has yet to be confirmed.

Tests need to be conducted to see if such a variant of CSSB can indeed be made successfully. Following that it would need to be tested to determine whether or not it exhibits the necessary level of durability for use in the humid tropics. If these proved successful, then a pilot scheme would need to be implemented to disseminate the information and necessary technology to a suitable area where low-cost housing is needed.

Bibliography

1. Gooding, D. E. M. (1993) *Improved processes for the production of soil-cement building blocks*, Unpublished doctoral dissertation: Warwick, University of Warwick.

2. Hanson. (2001) Hanson Bricks [Web Page]. URL www.hanson-brickseurope [Accessed: 2001].

3. Houben, Hugo and Guillaud, Hubert *Earth Construction: A Comprehensive Guide,* Intermediate Technology Publications.

4. Keable, Julian (1996) *Rammed Earth Structures: A code of practice,* Intermediate Technology ([qto] TH1421 K3).

5. Neville, A. M. (1995) *Properties of Concrete,* Longman.

6. Norton, John (1997) *Building with earth - a handbook*, Intermediate Technology Publications London. (UWL - TH 1421 N6).

7. Parry, J. P. M. (1979) *Brickmaking in developing countries*, Building Research Establishment UK. (UWL - [qto] TP 827 P2).

8. Stulz, R. and Mukerji, K. (1993) *Appropriate building materials,* SKAT (UWL - TH 145 S8).

9. Thermalite. (2001) Thermalite [Web Page]. URL www.thermalite.co.uk [Accessed: 2001].

20th WEDC Conference Colombo, Sri Lanka, 1994

AFFORDABLE WATER SUPPLY AND SANITATION

Disseminating ram-pump technology

Dr. Terry Thomas, Warwick University, UK.

FOR BOTH IRRIGATION and domestic supply, gravity feed is not always possible: water often needs lifting. The power to lift a flow of water can conveniently be expressed as

$$\text{power} = \frac{\text{constant x mean flowrate x height lifted}}{\text{duty x efficiency}}$$

where 'duty' is a time fraction (pumping hours per day) and 'efficiency' is a product of the efficiencies of the hydraulic circuit, the pump and the prime mover. Pipes are sized to give tolerable hydraulic efficiency and pumps are chosen to match the hydraulic conditions and the energy source available. Duty can also be varied to achieve better matching of the prime mover to the hydraulic circuit: high duties such as continuous 24-hour operation result in low power requirements and cheap piping (see Box on next page).

Whilst in general the power for water-lifting can come from engines, electrical mains, animals, humans or renewable (climatic) sources, in the particular context of rural areas in poor countries the choice is more constrained. In many such countries there are virtually no rural electrical mains, engines pose problems of both fuelling and maintenance, draught animals may be unavailable or difficult to apply to water lifting, renewables are erratic, complex and import intensive. Therefore human-powered lifting and transporting of water is still common, despite the very high cost of human energy (US$ 2 to 20 per kW hour).

Of the renewables, water power has the longest history, and under favourable conditions is the easiest to use. Several Asian and Latin American countries have developed the capability of building hydro-power systems. Although sites where power can be economically extracted from falling water are rather rare, they generally occur in the same terrain (mountainous) as the greatest water-lifting needs. The use of water power to pump water is therefore an interesting option. Figure 1 shows the main ways of doing this and illustrates the relative simplicity of the hydraulic ram-pump system. A typical such system is shown in Figure 2.

Ram-pumps (invented 200 years ago) are still manufactured in over ten countries and were once commonplace in Europe, The Americas, Africa and some parts of Asia. They have however been largely displaced by motorised pumping in richer countries, whilst in developing countries their use is concentrated in China, Nepal and Colombia. Ram-pump technology is not trivial: designing systems that are reliable, economic and durable (e.g. against flood, theft, silt ....) takes some experience. Generally, in rural areas of developing countries, this skill has been lost since about 1950, and the intermediaries that used to connect ram-pump manufacturers to pump users have disappeared. Old systems lie broken for lack of fairly simple maintenance: new systems are few.

Figure 1. Different configurations for water powered pumping.

For various reasons, discussed later, the potential for using ram-pumps seems to be increasing worldwide. Working, primarily in Africa, since 1985 the Development Technology Unit of Warwick University has identified several obstacles to this potential being realised, and has been trying to remove them. This paper records that experience.

Figure 2. Typical ram pump system.

The niche of the ram-pump

In suitable terrain, ram-pumps can be used to provide low-power unsupervised pumping. Typical individual ram-pumps can deliver 10 to 200 watts for lifting water; several small pumps can be operated in parallel to feed a single delivery pipe, larger pumps are available from some manufacturers. The power requirements of rural water lifting are illustrated by the following examples, which all assume pipe head losses are 10% of lift. The powers quoted are 'water watts' assuming 24 hours pumping.

domestic supply to a prosperous house (500 litres per day lifted 75m)	5W
village supply (l0,000 litres per day lifted 50m)	62W
irrigated garden (0.5 hectare) (35,000 litres per day lifted 20m)	87W

As the ram pump's system efficiency including its drive pipe is 50% to 75%, the hydro-power inputs for the examples above need to be up to twice the figures shown. The ram-pump is therefore well power-matched to these applications. These inputs are obtained at comparatively low drive heads - typically 10% of the delivery head - so the drive flows to ram-pumps are typically twenty times their delivery flows. (In the examples above the drive flow would be typically 7, 140 and 500 litres per minute respectively). This high flow requirement is clearly a constraint on location. On the positive side, however, no ram-pump user can extract more than a small fraction (e..g. 5%) of any source flow, the bulk of it being passed on downstream to other users: this has some social advantages.

Three other technical constraints require mention. Firstly there is only a limited range of head ratios (delivery height divided by drive head) of 5 to 30 over which a ram-pump is efficient and economic. Secondly neither drive head nor delivery head should exceed the particular pump's rating (often 20m and 100m respectively, but much less for cheap plastic ram-pumps). Thirdly it must be acceptable that the water lifted is derived from - and hence is of the same quality as - the drive water: a ram-pump cannot derive energy from a dirty stream to pump water from a different (cleaner) source.

Disregarding social and organisational factors, we can therefore describe the technical niche of the ram-pump as moist hilly rural areas where there is no mains electricity but a need for lifting water from streams or springs. The source must be of adequate quality and have a flow many times that to be lifted.

The problem of minor technologies

One of the more accessible concepts from 20th Century physics has been that of 'critical mass'. If the mass of a radioactive material or the size of an organisation is below some threshold its activity dies away; above that threshold the activity sustains itself and may even grow. For most technologies there is similarly a critical scale of application below which the activities needed to sustain it may die away. Such activities include manufacture of components, training of new users and specialist maintenance.

In the case of ram-pump systems, specific skills are needed in manufacture, system design, installation and operation. The skills are not especially high and overlap those needed to manufacture, install etc. other devices. Sometimes such skills are preserved in inanimate form. Thus many ram-pump manufacturers employ steel castings whose foundry patterns were made decades ago. Documents preserve design procedures. Existing installations are available as models for new systems. The critical throughput to sustain commercial manufacture is perhaps 50 pumps per year, it is usually achieved via selling into more than one country. A throughput of only one or two new systems a year might sustain system design and installation skills in a general water contractor. However, a specialist installer might need to put in at least 20 pumps a year to survive.

Box Continuous pumping versus discontinuous pumping

Water and wind powered pumps, and some electric pumps, are best operated for 24 hours per day. Solar, human and animal-powered pumps are limited to about 8 hours per day. Diesel pumps are typically run for only 2 hours per day in rural areas because they are usually over-powered for their applications. These differences in duty (load factor) have implications for pipe and storage costs. Pipes are sized so as to give a 'tolerable' friction head loss (FHL). What is tolerable depends on the means available to supply this head loss, for example pump power or the slope of the pipe. We know that, for a given length of pipe, FHL is proportional to Q^2/D^5, where Q is flowrate in the pipe and D is its diameter. Also pipe cost (per meter) is typically proportional to D^2. These relationships give the table below, which is based on a specified daily flow.

	Pumping for 24 hours/day (taken as datum)	Pumping for 1 x 8hrs/day	Pumping for 2 x 1 hr/day = 2hrs/day
Power to overcome FHL	1	x27	x1728
Energy to overcome FHL	1	x9	x144
Pipe D for constant FHL	1	x1.5	x2.7
Pipe cost for constant FHL	1	x2.4	x7.3
Typical storage ÷ daily throughput	0.4	0.5	0.4

In reviving an old technology or introducing a new one, the 'critical mass' throughputs need to be estimated. If they are higher than the area of sales or of installer operation can sustain, any intervention to promote the technology will ultimately fail. More important, if the likely demand is thought to be close to such a threshold of sustainability it is worth effort to lower the threshold.

With the technology of hydro-electricity we are used to having separate organisations making turbines, designing systems, building them and operating them. Maintenance may require a fifth agency. Even though some of these organisations operate internationally via local agents, such complexity entails uncertainties that tend to raise the critical size for each of them. Micro-hydropower utilisation has lagged behind its apparent economic potential for these reasons in most countries. Ram-pumping faces similar difficulties.

Often there is a key agency that effectively leads the others involved in a technology. For example a manufacturer of equipment may set up training for its installers, users and maintainers; alternatively a consultant may co-ordinate and supplement the existing skills of the other parties. A low value rural technology does not lend itself to the latter approach.

Experiences in Africa

The author and his DTU colleagues have been trying to revive ram-pump usage in Africa since 1985. An early analysis suggested that foreign (e.g. European) manufacturers selling a few pumps a year via agents could not and would not provide adequate training for local installers. Moreover imported pumps are expensive and difficult to source spares for. In colonial times there were few technical alternatives for water lifting to plantations, mission hospitals and large schools and it was worth the cost of bringing a ram-pump installer from another continent. Today that is an unacceptably expensive option for a village or farm needing pumped water or for a small-scale pumped irrigation scheme.

In the absence of a design consultant (again unlikely for this scenario), the options for sustainability appeared to be

either to build up the design capability of installation contractors

and/or to encourage local manufacture by an organisation also capable of providing back-up to installers.

The DTU chose the 'and' option, first spending several years in developing simple and cheap pump designs suitable for provincial manufacture and codifying system design and installation procedures. Since 1990 the DTU has been training both producers and installers from nine African and one Asian country, usually using its demonstration centre in the Eastern Highlands of Zimbabwe. There is an ongoing debate about what is the right level of manufacturing technology (hand tool, workshop with electricity, factory), whether manufacture and installation should be undertaken by the same organisation, whether low-lift irrigation or high-lift water supply should be given priority, whether installer training should be directed towards governmental, NGO or private organisations and what fraction of possible sites are 'easy' sites suitable for beginners to tackle.

The results have been mixed. Easy sites (with modest lifts, plentiful water, favourable stream geometry and well-organised customers) are perhaps only a few percent of technically feasible sites. The process of system design has proved intimidating to technicians for whom even sizing a pipe for gravity flow is at the limit of their understanding. The input of (expatriate) man and woman power to bring an installation organisation up to the level of competence and confidence to stand alone with this technology has been expensively high. The 'successes' have been with unusually well-resourced NGOs. Commercial manufacture, for example in Kinshasa (Zaire) and Mutare (Zimbabwe) has been started but self-sustaining manufacturer-installer arrangements have not been developed. Of some 30 pumps installed, too many have been 'demonstrations' rather than built to meet real water needs.

Clearly training on courses alone is not enough. Installers and manufacturers need to be visited and helped / encouraged with production of their first systems. A ram-pump has a certain 'something-for-nothing' magic about it that impresses onlookers and causes any installation to yield many enquiries from neighbouring villages or farms. However the technology's uncertainties, using very cheaply produced pumps in the hands of novice installers, makes it much easier to apply to individual 'rich' farms or institutions than to villages or communal dry-season gardens.

Ram-pump technology has a fascination for enginers and users out of proportion to its current commercial importance. The DTU's 1992 book on system design must have sold more copies worldwide than there have been new systems built! A 1993 day school on ram-pumps in Sri Lanka attracted fifty engineers but so far has resulted in no new systems.

Prospects

Ram-pumping will never be a major technology comparable with motorised pumping from rivers or hand pumping from boreholes. Its particular niche is described above: worldwide there is a potential for between perhaps 10,000 and 200,000 systems. Much of that potential lies in areas where there are currently no system design skills. Availability of pumps need not be a major problem (despite the DTU's local manufacture strategy in Africa), since even though good imported machines cost over \$10,000 per kilowatt the pump itself rarely accounts for more than 40% of system costs.

Certain trends worsen the prospects for ram-pumps. Worldwide, water sources are becoming both dirtier and weaker. Some historical ram-pump systems no longer operate because of declining drive flow. Clean spring water is usually associated with very low power levels - in Rwanda for example, the DTU had to design for 80 metre lifts from drive flows under 10 litres per minute, which is on the limits of the technology.

Factors increasing likely demand are the movement of rural populations uphill (under population growth pressures), the expansion in micro-irrigation, the introduction of local ram-pump manufacture (especially in S. America) and the availability, apparently for the first time in decades, of both trustable handbooks and training courses.

In Africa the prospects for ram-pump usage seem to depend largely on the confidence of potential installers. Despite much individual innovation there, Africa is not a continent where organisations readily take risks with unknown technology. Elsewhere in the developing world continuation of the current slow expansion of ram-pump usage will depend upon developments in photo-voltaic pumping, its most immediate rival.

The scope for technical improvement of a simple device already used for 200 years is rather small. However, modern materials may permit the pressure vessel (required to smooth the pulsating flow through the delivery valve into a steady flow up the delivery pipe) to be replaced by a pressured bladder. This will allow pumps to be operated slightly under water which has advantages for both efficiency and reliability. Understanding of the causes of erratic pump behavious and of inefficiency is now better than in the past, which designers of pumps and 'trouble-shooters' of systems can draw upon. It is not possible to totally design away temperamental behaviour, during for example system start-up, but its incidence can certainly be reduced.

For the ram-pump to fully occupy its niche, efforts must continue both to simplify the design of reliable systems and to propogate design skills. Although water-powered pumping will never attain the simplicity of "drop the suction pipe in the stream and switch on" that motorised pumping offers, as users of a renewable energy source, ram-pumps may have time on their side.

References

The following books explain how ram-pumps work and provide system design assistance. They also contain addresses of (Northern) manufacturers.

Meier V. *Hydram Information Package*, SKAT, St. Gallen, 1990.

Knol H. *The Fall and Rise of the Hydraulic Ram-pump*, Drachten (Netherlands), 1991.

Jeffery T. et al *The Hydraulic Ram-pump*, London, IT Pubs, 1992.

Please contact the DTU, Warwick University, Coventry, CV4 7AL, UK for drawings of pumps designed for local fabrication.

Partially Below Ground (PBG) tank for rainwater storage

Instructions for Manufacture

Prepared by Dai Rees

Development Technology Unit
School of Engineering
University of Warwick
Coventry CV4 7AL
Tel: +44 (0)1203 522339
Fax: +44 (0)1203 418922
dgr@eng.warwick.ac.uk
http://www.eng.warwick.ac.uk/DTU/

November 2000

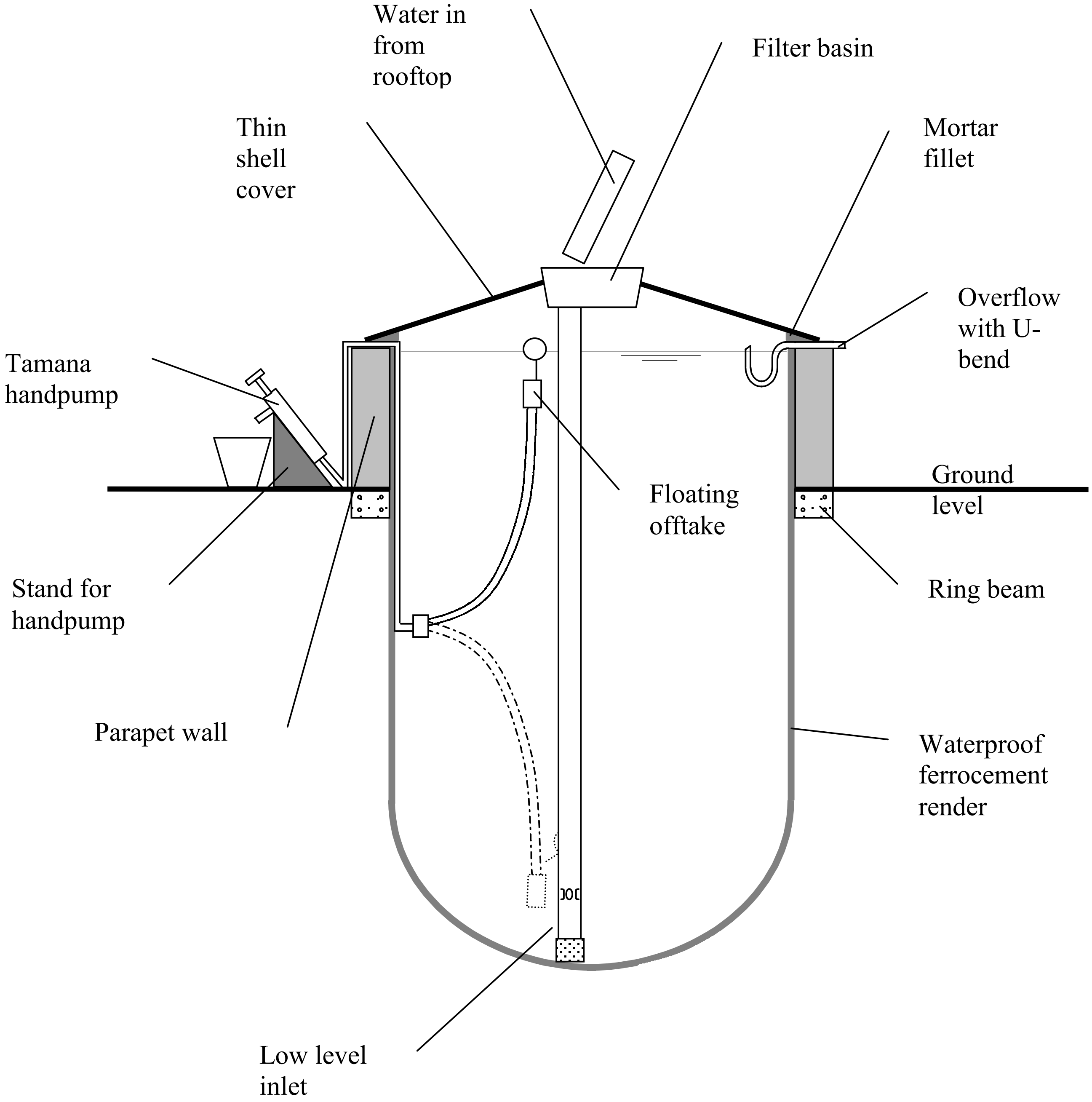
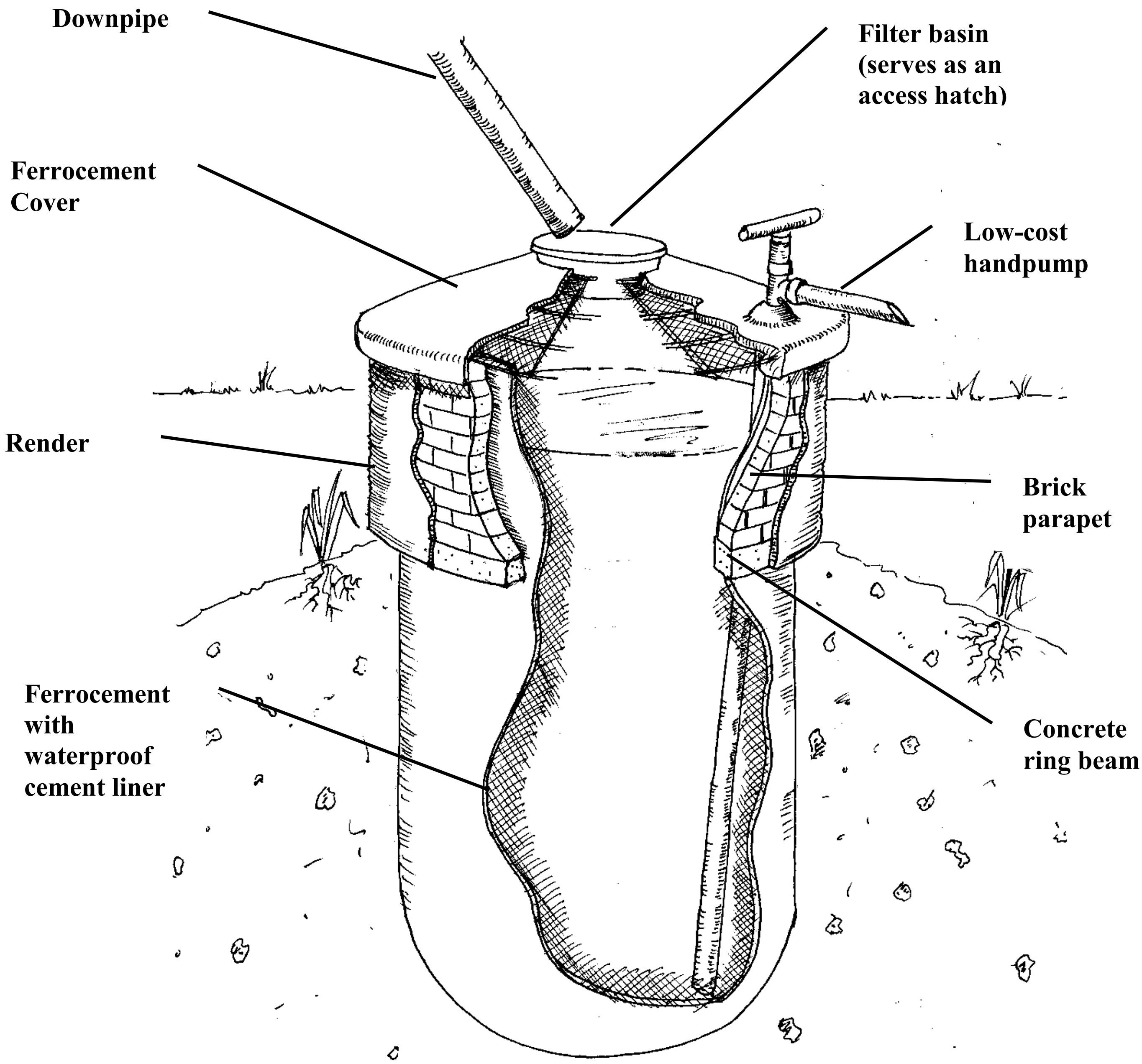


Figure 1- The Partially Below Ground (PBG) Tank



Partially below ground tank sketch

Introduction

Tanks for rainwater storage come in many shapes and sizes. The main distinction between tank types are size, shape, material and whether they are sited above or below ground.

Some of the relative merits and drawbacks of above and below ground tanks are listed in Table 1 below.

Table 1. Pros and Cons of Tanks and Cisterns

	Tank	Cistern
Pros	• Above ground structure allows for easy inspection for cracks or leakage • Many existing designs to choose from • Can be easily purchased 'off-the-shelf' in most market centres • Can be manufactured from a wide variety of materials • Easy to construct for traditional materials • Water extraction can be by gravity in many cases • Can be raised above ground level to increase water pressure	• Generally cheaper due to lower material requirements • More difficult to empty by leaving tap on • Require little or no space above ground • Unobtrusive • Surrounding ground gives support allowing lower wall thickness and thus lower costs
Cons	• Require space • Generally more expensive • More easily damaged • Prone to attack from weather • Failure can be dangerous	• Water extraction is more problematic – often requiring a pump • Leaks or failures are more difficult to detect • Contamination of the tank from groundwater is more common • Tree roots can damage the structure • There is danger to children and small animals if tank is left uncovered • Flotation of the cistern may occur if groundwater level is high and cistern is empty heavy vehicles driving over a cistern can cause damage

The partially below ground (PBG) tank incorporates the merits of both above and below ground tanks in one simple, low-cost design. The PBG tank takes advantage of the support given by the soil, to do away with the need for a structural component below ground level. At the same time protection is given against contamination by surface runoff and damage by vehicles.

To date (November 2000), about 20 of these tanks have been built in SW Uganda using render linings, often without the chicken mesh. The reports from the field have been good, and feedback suggests that the tanks are easy to construct by masons with some training, at a reasonable cost. A training course for masons was held Kyera Farm, Mbarara, Uganda in June 2000 and 8 local masons were trained in the art of constructing the PBG tank. The instructions given here for manufacturing the tank were developed for this training course.

WARNING: The PBG tank is suitable for construction where the soil conditions are stable e.g. in the lateritic soils of East Africa. If there is any doubt about the stability of the soil, then seek further advice. Working in soils that are unstable can be very dangerous, even fatal.

Instructions are given in a step-by-step guide and these should be followed carefully, especially the tips relating to curing and quality of workmanship. It is worth bearing in mind, however, that materials availability varies from place to place, and so where a given material is not available, a substitute can usually be found and the necessary amendments made.

The instructions given below are for the construction of the PBG tank without the cover or handpump (although the cost of the cover is included in Table 2). **Instructions for the manufacture of the cover and the handpump are given is separate documents – TR-RWH 04 and TR-RWH 09 respectively.**

Neither is any detail given for sizing the tank in terms of demand and supply. This information can be found in many RWH texts or on the DTU Web Page at http://www.eng.warwick.ac.uk/DTU/rainwaterharvesting/index.html

Tools and materials required

Tools – the tools listed below are for the construction of the PBG tank without the cover or handpump.

- 2 shovels
- pick
- 2 buckets
- masons hammer
- wheelbarrow
- plastering tools (if plastic liner is not being used)
- bricklayers trowel
- spirit level
- handsaw for making wooden profile
- bucket on a rope for lifting out soil
- wooden mallet (can be home-made) for tamping walls

Materials
The following are the materials required for a 10.8 cubic metre, ferrocement lined tank.

Table 2 - Materials requirement for the 10.8 cubic metre ferrocement lined PBG tank										
Item	Tank component									
	Ground leveling and ring excav'n	Ring beam	Excav-ation	Parapet wall	Parapet - external render	Internal render - first coat	Internal render- second coat	Cover	Placing cover	**Totals**
Cement (OPC) - kg		30		50	25	100	200	50	10	**465**
Sand - kg		60		300	125	300	600	150	50	**1585**
Aggregate (<50mm) kg		120								**120**
Bricks (angled) - no				300						**300**
Chicken wire (0.9m wide) - m							24			**24**
Staples - kg							1			**1**
Waterproof agent - kg							4			**4**
6mm rebar - m								20		**20**
8mm rebar - m								20		**20**
Coffee tray mesh - (0.9 wide) -m								4.8		**4.8**
Binding wire - kg								1		**1**
Basin for filter								1		**1**
Labour (skilled) - days		0.25	1	2		2	2.5	1	0.5	**9.25**
Labour (unskilled) - days	1	0.5	10	2		2	2.5	1	0.25	**19.25**

Table 3 - Costs for the PBG outlined in Table 2				
	Number reqd	Unit cost (Ugandan Shillings)	Total cost (Ugandan Shillings)	Total £
Cement (OPC) - kg	465	300	139500	62.84
Sand - kg	1585	20	31700	14.28
Aggregate (<50mm) kg	120	25	3000	1.35
Bricks (angled) - no	300	52	15600	7.03
Chicken wire (0.9m wide) - m	24	1667	40008	18.02
Staples - kg	1	2500	2500	1.13
Waterproof agent - kg	4	4000	16000	7.21
6mm rebar - m	20	230	4600	2.07
8mm rebar - m	20	385	7700	3.47
Coffee tray mesh - (0.9 wide) - m	4.8	4350	20880	9.41
Binding wire - kg	1	2000	2000	0.90
Basin for filter	1	1000	1000	0.45
Labour (skilled) - days	9.25	5000	46250	20.83
Labour (unskilled) - days	19.25	3000	57750	26.01
		GRAND TOTAL	**388488**	**175.00**
		Total materials	284488	128.15
		Total labour	104000	46.85

Exchange rate as of July 2000 (£1.00 = USh2220)

Instructions for manufacture

1. Finding a suitable site
   1.1. The first step is to find a suitable site for the tank. Some pointers for what constitutes a good site are given below:
   - Close enough to the dwelling to avoid long lengths of guttering and downpipe (some suggest siting the tank mid way along the length of a building to reduce gutter size– this is fine if water from one side of the building only will be fed into the tank)
   - Reasonably flat where possible – otherwise the ground will have to be levelled before marking out
   - Away from areas where surface water will gather (i.e. depressions)
   - Away from trees – the roots of trees can be problematic
   - Away from areas where animals will wander – or else the tank should be fenced off
   - Not so close to the dwelling that the foundations of the dwelling are undermined
   - Somewhere convenient for extracting water e.g. close to the kitchen area

   1.2. The ground should be suitable for digging and for siting such a tank. There should be no large stones, bed rock or sheet rock close to the surface, and one should be sure that the groundwater table in the area is several meters below the bottom of the tank. This information can often be gleaned from locals who may have tried digging wells, sinking boreholes or digging garbage pits.

Figure 2 - Showing the cleared ground and the markings in place for the ring beam

2. Deciding what depth the tank will be
   2.1. As mentioned earlier, the sizing of the tank in terms of supply and demand is not given in this document. It is assumed that the sizing of the tank has been carried out correctly.

   2.2. The nominal diameter of the tank is given as 2m. All the sizes given in these instructions are for a tank of 2m nominal diameter. The actual internal diameter is slightly less than this.

2.3. The actual volume of the tank is dependant, therefore, upon the depth of the tank. Table 2 below shows the volume of the tank for a number of given depths (the depth is total depth from the top of the parapet wall, which is 1.0m high).

Table 4 - depths and volumes for PBG tank			
Volume (cubic metres)	Depth of cylindrical section to be dug	Add 1m for the parapet wall and 0.95m for the hemisphere	Total depth of tank (given from top of parapet to base of hemisphere)
8.0	0.55	1.95	2.5m
9.4	1.05	1.95	3.0m
10.8	**1.55**	**1.95**	**3.5m**
12.2	2.05	1.95	4.0m

3. Casting the ring beam

3.1. Mark out two circles with inner and outer diameters of 1.9 and 2.2m respectively (i.e. radii of 0.95 and 1.10m respectively). This gives a ring beam of width 0.15m (150mm)

3.2. Dig between the lines to a depth of 150mm keeping the trench neatly trimmed and clean. A machete can be used for trimming the walls of the trench to get a good, clean finish. When completed clean out any loose earth from the trench.

Figure 3 - Casting the ring beam

3.3. Make a concrete mix of **1:2:4** (cement : sand : aggregate), using the quantities shown in Table 5. Be sure the concrete is well mixed and then place the mix into the trench being sure that any air voids are removed by 'vibrating' the concrete with a stick. Remember that wet concrete mixes will have lower final strength, so keep the mix workable but not too wet.

Table 5 - Quantity of material required for the ring beam	
	Approximate quantity required
Material	**Kg**
Aggregate (<50mm)	120
Sand (clean, graded)	60
Cement (Ordinary Portland)	30
Water (clean)	Enough to make the mix workable

3.4. The ring beam should be covered and cured for 7 days before any further work is carried out (keep the concrete wet during the curing period). Keep the beam covered with polythene during this time and wet the concrete at least twice a day.

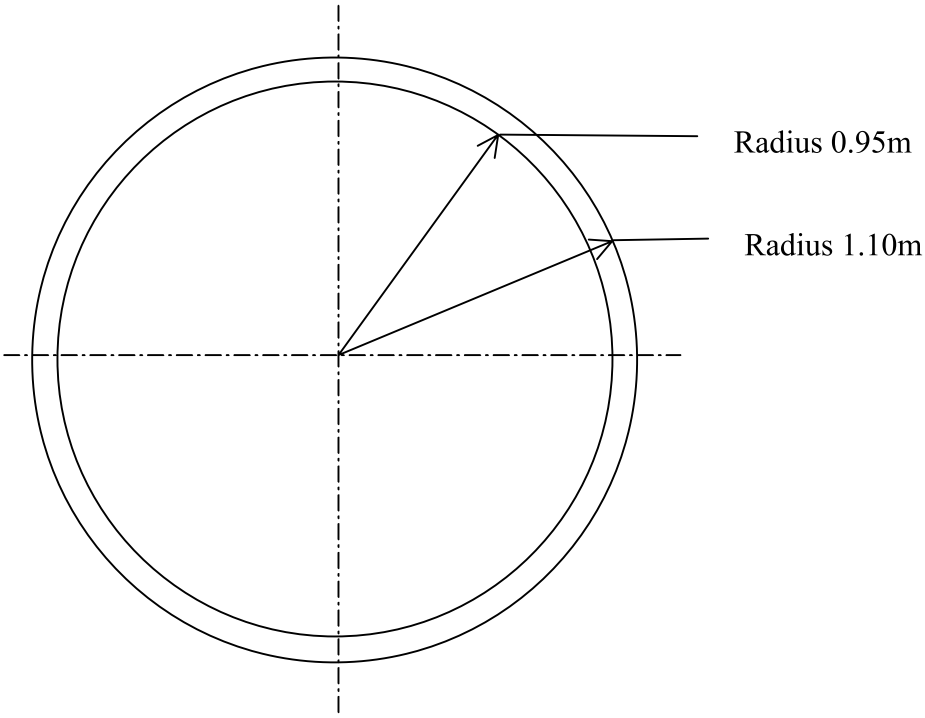


Figure 4 – Dimensions for marking for the ring beam

4. Excavating the hole
    4.1. When the beam has cured the hole can be dug. Use Table 4 to decide what depth of hole is required. The sides of the tank should be kept reasonably vertical. This can be checked occasionally using a plumb line or a masons spirit level.

    4.2. As a rough estimate of the time and manpower required to dig the hole, use the figure of 1 person-day per cubic metre of excavation.

    4.3. The bottom of the hole is shaped like a hemisphere or an inverted dome. This shape is easily dug with a shovel. A rod can be placed centrally in the ground and a piece of string used as a guide if there is any difficulty.

Figure 5 – Excavation of the tank

Figure 6 - Plumbing the walls to keep them vertical

5. Building the parapet wall
   5.1. The parapet wall is built to a height of 1.0m. It is recommended that bricks be cast and fired especially for the construction of the tank to the dimensions shown in Figures 7 and 8 below. Where this is not possible, it is recommended that a standard 100 x 75 x 225 fired house-brick is used, although other (larger) sizes can also be used.

Figure 7a – Showing the mould used for casting the bricks used for the parapet wall

Figure 7b - The bricks with profiled ends

5.2. The number of bricks required depends on the actual brick size, but assuming the dimensions are as given above, then 24 bricks per course will be required. For a height of 1.0m this will be a total of 240 to 270 bricks (10 or 11 courses depending on thickness of mortar used). This represents 6m^2 of brickwork.

5.3. Bricks should be laid with a mortar mix of 1:6. When placing the final course of bricks two small gaps should be left, one for handpump pipe, which will be placed later, and the other for the overflow pipe. This should also be cast into the wall at this stage.

5.4. The overflow trap (see Figure 9) allows overflow water to escape from the tank while preventing mosquitoes from entering the tank. The U-trap is filled with water and so mosquitoes cannot pass. A mosquito mesh also prevents mosquitoes entering the overflow pipe. The trap is made from 50mm (or similar) plastic pipe. If the U-trap is not used then mosquito mesh should be fitted over the end of the overflow pipe. The tank owner should be advised to replace this if damaged or removed.

5.5. When complete, the parapet wall can be externally rendered. This is not essential but makes the tank look better.

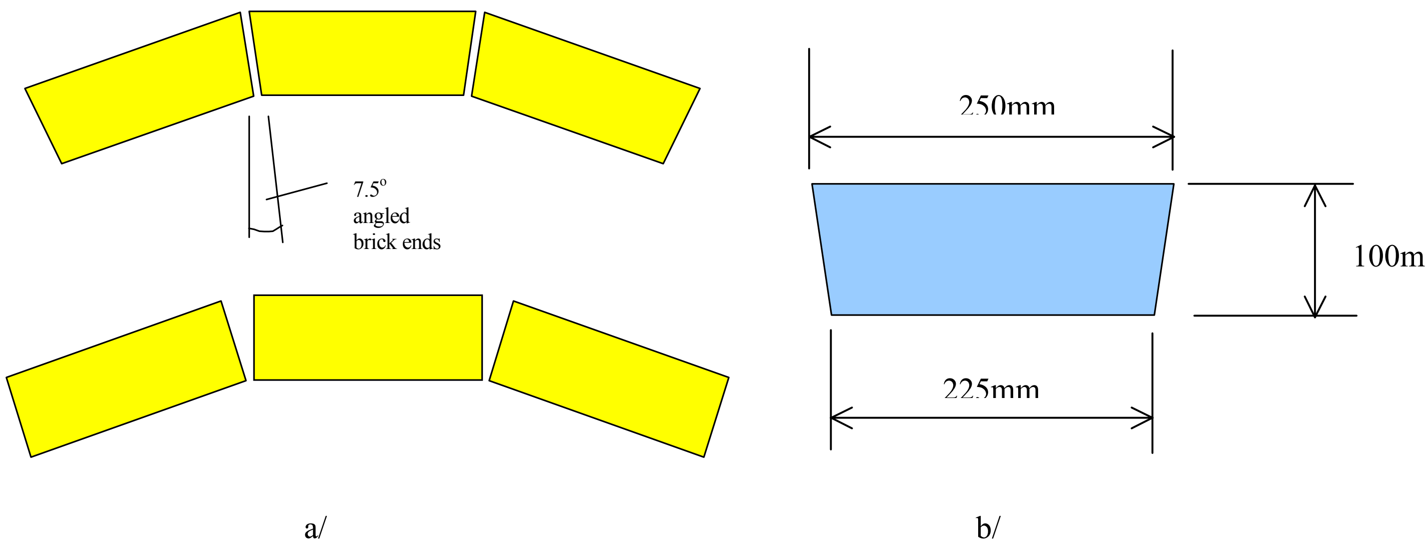


Figure 8 - a/ where bricks are cast and fired specially for tank construction, they can be cast with a slight angle at each end. To minimise the amount of mortar used the bricks should be cast with an angle of 7.5° on each end of the brick b/ showing actual dimensions of brick cast for tank construction

5.6. Material requirement for wall

Table 6 – Quantity of material required for the parapet wall		
	Approximate quantity required	
	Wall	**Render**
Material	**kg**	
Sand (clean, graded)	300	25
Cement (Ordinary Portland)	50	125
Bricks	300	
Water (clean)	Enough to make the mortar workable	

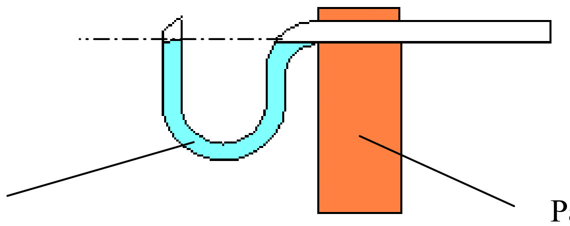


Figure 9 – The U-trap

5.7. Allow 2 skilled person-days and 2 unskilled person-days for building and rendering the parapet wall.

Figure 10 - showing the construction of the parapet wall

6. Lining and water-proofing the tank
   6.1. Any sharp stones or roots protruding from the wall should be removed. Where holes have appeared due to stones having been removed, these should be filled with clay soil and tamped hard and level with the surrounding surface.

   6.2. It is very important to achieve a waterproof lining for the tank. As mentioned earlier, one of the drawbacks of an underground tank is that it is difficult to detect leaks. We, therefore, need to be sure that the tank is well lined and will not leak under normal usage. Good workmanship here is essential.

6.3. Ferro-cement render. This technique is based on the well-known ferrocement technology that has been well documented (see Watt, 1978 and Gould and Nissen Peterson, 1999). The technique involves using a composite of cement render and galvanised chicken wire mesh. A water-proofing compound (readily available in most countries) is added to the cement render. The procedure for application is given here:

- A thin coat (~ 1cm) of 3:1 cement render is applied evenly to the wall of the tank. When the render has started to set (after about 30 minutes), score the render lightly to provide a key for the next layer.
- This first coat is allowed to cure for 2 days. The top of the tank should be covered with a plastic sheet during this time and the walls regularly and liberally sprinkled with water.

Figure 11 - showing the rendering in progress

- A layer of 1” chicken wire mesh is then applied to the render. This mesh is fixed to the render using galvanised fencing staples. Care should be taken to lay the mesh as flat as possible onto the render.
- When the chicken wire is in place the second coat of render can be applied. This is again a 1:3 mix, but includes a waterproofing agent, which is added during the mixing. Any proprietary waterproofing agent can be used and the manufacturer's instructions followed regarding the quantity to be added. The second coat of render should be applied in a similar fashion to the first – about 1cm in thickness (although it may be thicker in places to cover the chicken mesh). This second coat of render should be cured for 7 days as described above.
- The gap around the overflow pipe should be sealed using a waterproof mortar. This is done before the cover is fitted.

Scaffolding or ladders should be used when rendering the walls of the tank. See Figure 12

Material requirement for render:

Table 7 – Quantity of material required for the ferrocement lining (1:3 mix – approx. 21m^2)	
	Approximate quantity required
Material	**kg**
Sand (clean, graded)	600
Cement (Ordinary Portland)	200
Water (clean)	Enough to make the mix workable
Waterproofing agent e.g. Leak Seal Waterproofing Compound @ 2%	As per manufacturers instructions
Chicken mesh (0.9m wide)	24m length
Staples	1 kg (fencing staples)

Figure 12 - Ladders and boards used to provide a working platform

7. Fitting the handpump
   7.1. More detail of the handpump is given in another DTU publication – TR-RWH 09 "Low-cost handpumps for water extraction from below-ground water tanks - Instructions for Manufacture". The fitting and fixing procedures are dependant on the type of pump used.

8. Making and fitting the cover.
   8.1. The manufacture of the cover is discussed in a separate document TR-RWH 04 " Low-cost, thin-shell, ferrocement tank cover - Instruction for Manufacture". The cover is made independently of the tank and is fitted when complete.

Figure 13 - The tank with cover in place

8.2. The cover is easily lifted into place by 4 – 6 people. It weighs in the region of 200kg and the lifting height is 1m. Care should be taken when lifting the cover and proper safety precautions should be observed.

8.3. The cover will be mortared into place. A 2cm thick mortar is placed on top of the parapet wall and the cover is placed onto the mortar. The mortar is smoothed to seal the cover onto the wall. The mortar is also smoothed on the inside of the wall / cover joint to form a continuous waterproof lining.

9. The filter is part of the cover. When the cover is manufactured, a plastic basin is used to form the access opening. This basin is then left in place, filled with coarse gravel and covered with a fine cloth. The cloth filters out any coarse debris and should cleaned when dirty and replaced when damaged. It is easy to monitor the condition of the cloth as it is in clear view on top of the tank. The owner of the tank should be made aware of this.

Figure 14 - Showing two types of handpump (DTU and Tamana), filter basin and externally rendered parapet wall, on this demonstration tank at Kyera Farm, Mbarara, Uganda. The guttering has not yet been fitted.

10. Guttering and pipework
    10.1. The guttering and downpipe are not specified. This is due to the wide variation in the styles available. This is left to the discretion of the installer.

    10.2. In the general diagram in Figure 1, a low-level inlet is shown. This helps to prevent disturbance of the water and directs sediment to the bottom of the tank. The floating intake then takes water from just below the water's surface, where is cleanest. This arrangement is not essential but it is desirable, especially where the water is used for potable supply. The fitting of the low-level inlet again is left to the discretion of the installer. The floating intake is discussed in TR-RWH 09.

11. Maintenance. The maintenance of the tank is quite simple. The following steps should be followed:

- The tank should be cleaned annually - at the end of the dry season, the tank should be emptied and any debris in the bottom of the tank removed.
- The filter should monitored to make sure that it does not become blocked. If the cloth becomes damaged it should be replaced.
- The overflow should be covered with mosquito mesh at all times.
- The tank and associated guttering and pipework should be kept in good general repair. Any damage or faults should be rectified as soon as possible.
- The maintenance of the pump is discussed in TR-RWH 09.

Single-skin, externally reinforced, brick tank

Instructions for manufacture

Development Technology Unit
School of Engineering
University of Warwick
Coventry CV4 7AL
Tel: +44 (0)1203 522339
Fax: +44 (0)1203 418922
dgr@eng.warwick.ac.uk
http://www.eng.warwick.ac.uk/DTU/

DTU

October 1999

Introduction

The single-skin, externally-reinforced brick tank is designed to minimise material input. The single-skin brick wall, normally having insufficient hoop strength to withstand the stresses imposed by the internal water pressure, is externally reinforced with packaging strap to give adequate hoop strength to the tank. This reduces the quantity of brick required to build such a tank, with brick only, by more than half. The capacity of the tank is approximately 6 cubic metres, with an internal diameter of 2.0m and a height of 2.0m.

These instructions are based on the procedures used to manufacture a tank at the Vipassana Meditation Centre in Herefordshire, UK, during the summer of 1999. Figure 1 shows the main components of the tank (the external render is not shown in order that the strapping and brickwork configuration can be seen clearly). These instructions go through the construction procedure step by step, but it should be remembered that where identical materials are not available (e.g. using different brick sizes) then allowances may have to made to compensate for this.

Tools required for the construction process

Tool	**Number needed**
Shovel	2
Bucket	2
Trowel (large)	1
Trowel (small)	1
Float	1
Spirit level	1
Hammer	1
Tape measure	1
Ladder (trestle type)	1
Wheel barrow	1

Total time required for tank construction:

Skilled - approx 4 days
Unskilled - approx 4 days

Also needed

- Clean area for mixing mortar (preferably hard surfaced)
- Plastic sheeting for curing (5m x 5m)

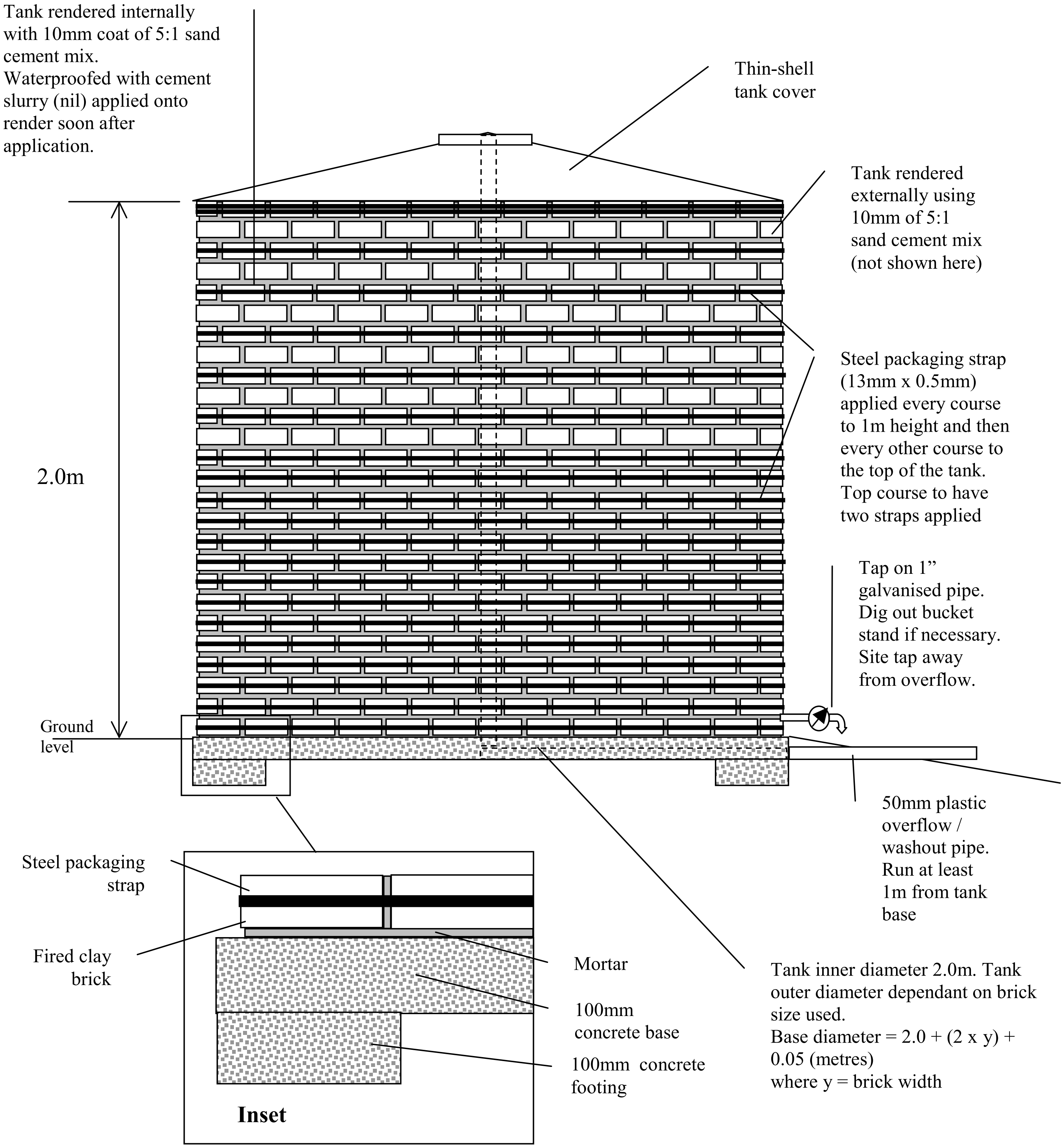


Figure 1. Sketch (not to scale) of the SSER tank, showing the main components

Laying the foundations

Material requirement (approximate)

- hardcore (for base) 1.5 tonnes
- sand 0.5 tonnes
- aggregate 1.0 tonnes
- cement 5 x 50 kg bags
- 75 mm plastic pipe 2m
  1 x 90^o elbow

(we used 63mm solvent weld rainwater downpipe, but any similar pipe will do – it has to be of sufficient diameter to act as overflow)

Time required: skilled 0.5 days
unskilled 0.5 days

Find a suitable location for the tank i.e. close enough to the catchment area to conveniently transport the water, an area with suitably firm ground with no risk of subsidence, etc. If the ground is sloping slightly take advantage of this to site overflow pipe later. If the ground is flat the base may have to be laid above ground level to allow overflow water to run off. If the latter is the case, some form of shuttering will be required (bricks can be used and reclaimed for use in the wall later).

Assuming there is sufficient slope to take the overflow pipe out as required, dig a circular hole to a depth of 250mm and to a diameter that is two metres plus twice the width of the brick being used and then add 50mm (0.05m) (see note below).

Diameter of base = 2.0 + (2 x y) + 0.05 metres

where 2.0m is the internal diameter of the tank
y is the width of the brick being used
0.05m gives a 25mm border around the base

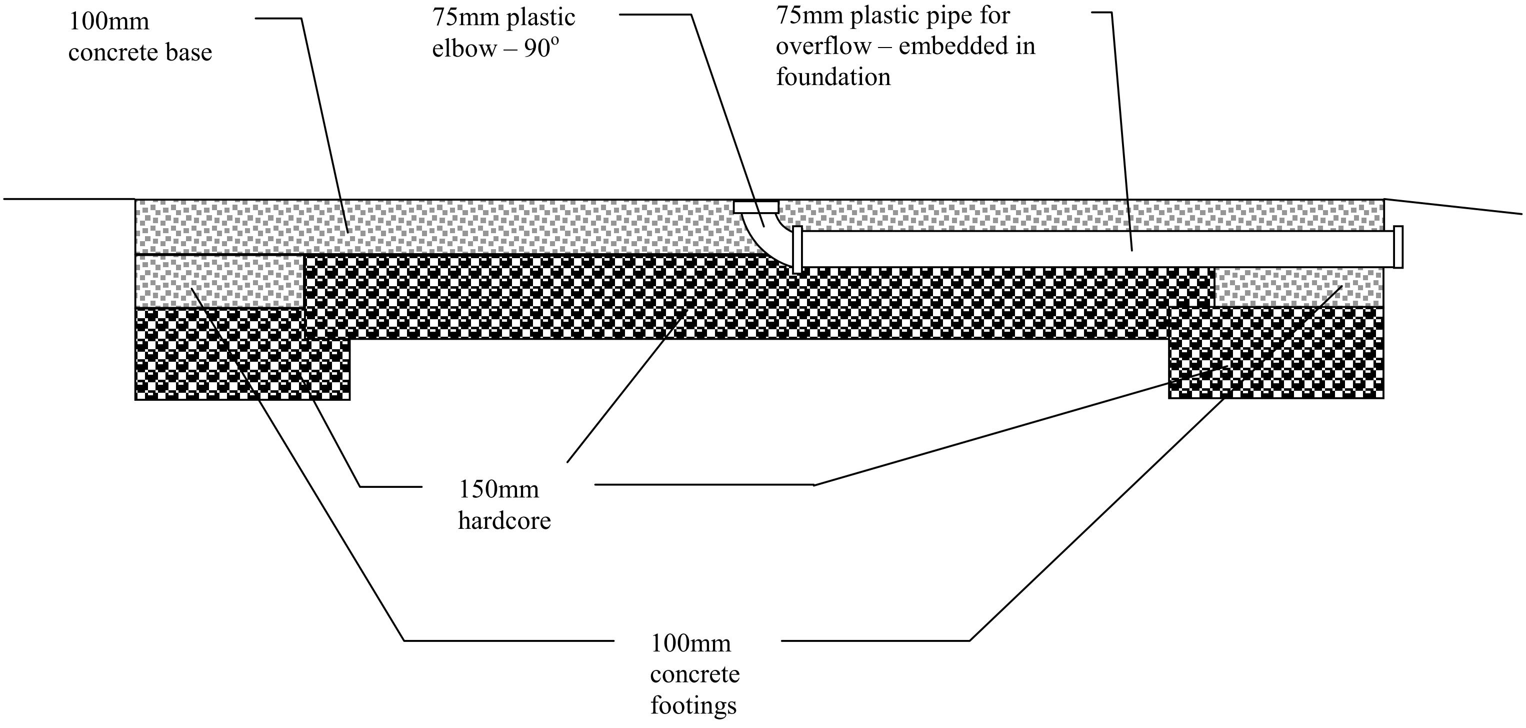


Figure 2 – details of the tank base

The footings are then dug to a further 100mm depth. The width of the footings again depends upon the width of the brick being used – allow for the 25mm border around the tank, add 50mm inside the tank wall and also make an allowance for the stone that will make up the under-base. Lay the stone to a depth of 150mm.

The overflow pipe is now placed such that it will sit partially in the stone (one third of its depth) and partly buried in the concrete (two thirds of its depth). There should be a very slight gradient on the pipe. The elbow is at the centre of the tank. Make a good seal between the elbow and the pipe.

Then peg out the area ready for the concrete. A peg is placed at the centre of the tank, next to the elbow. Pegs are then placed at regular intervals around the perimeter of the tank at the same level.

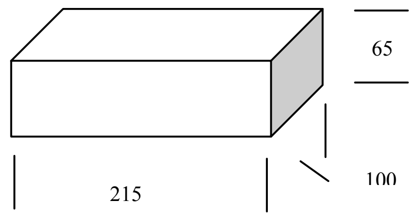
Make the base in one session. Use concrete of mix 4:2:1 (aggregate: sand: cement). Level using a tamping board using the pegs as a guide.

Building the tank wall

Material requirement:

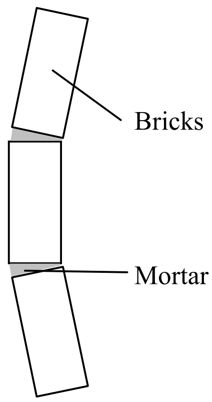
- bricks 800 (using brick dimensions shown)
- Cement 3 x 50 kg bags
- Sand 1 tonne

Time required: skilled 1.5 days
unskilled 1.5 days



Brick dimensions used on prototype

The tank wall is built simply by forming whole bricks into a circle. The bricks are not cut to shape. This leaves a slight angle between bricks but this causes no problems and is compensated for when rendering. The verticality and cylindricality of the wall can be maintained in one of two ways:



1. By using a spirit level – if the wall is set up at the base to be round and the walls are kept vertical then the tank will be perfectly cylindrical.

2. By placing a length of pipe in the elbow and bracing it in the vertical position. A length of is then loosely tied around the pipe and measured to give the desired length. This is then used as a guide for the tank construction.

Note: if the first method is used the elbow should be protected to prevent mortar falling in during construction.

The wall is built up to the required height i.e. 2 metres and a trestle ladder is used to pass mortar into the tank for building. Working from the inside is easiest – stock up enough bricks inside to finish the wall at an early stage.

The wall should then be properly cured by covering the whole with polythene sheet for 7 days and wetting the walls daily.

Applying the steel packaging strap

Kimarakwija – you will have to source a supply of packaging strap and purchase a tensioning tool and crimping tool from Kampala. Quote for the cost of the trip to Kampala, as well as the tool and crimps, in your tender. The full kit for this includes:
- steel strapping (comes in rolls of several hundred metres)
- tensioning tool (for pulling the strap tight around the tank)
- crimping tool (for crimping the strapping once in place)
- dispenser (optional tool for easily dispensing the strap – makes this job a lot easier)
- crimps (usually in a box of 1000 – for crimping the strap)

Time requirement:

- skilled 0.5 days
- unskilled 0.5 days

The strap is applied to the brick masonry specimen using a manually operated tensioning tool. Once fully tensioned the strap is crimped using specially designed crimps and crimping tool and then the tensioning tool is removed. It can be seen from Figure 8 below that the tensioning tool holds the strap away from the wall in order to allow access for the jaws of the crimping tool. When the tensioning tool is removed there is some loss of tension in the strap and so packing is placed under the strapping (pieces of broken stone can be used) before the tool is removed to prevent this loss of tension.

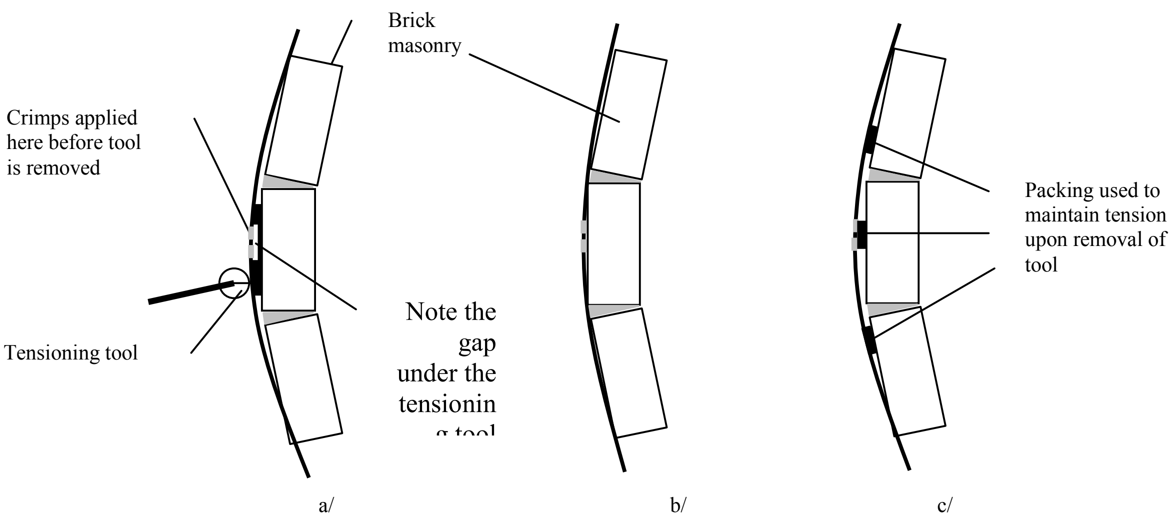


Figure 8 – showing tensioning and crimping arrangement for steel strapping a/ during the tensioning and crimping process b/ when crimping is complete and the tensioning tool has been removed and tension reduced c/ maintaining tension by using packing

The strapping is placed on every course of bricks for the lower one metre of the tank and then every other course for the upper metre. There can be some difficulty in applying the strapping on the lower two courses because of the difficulty of access for the tool. This can be overcome by digging a small hole in the ground where the tool access is required. This can be filled later.

Covering the tank
The tank is covered with a thin shell ferrocement cover. This is mortared into place. (this has been quoted for separately)

Rendering the tank
Material requirement:

▪ Cement	2 x 50 kg bags
▪ Sand	400kg (mix 4:1)
▪ Mortar plasticiser	1 litre (where available)
▪ Water	as required

Time:

▪ Skilled	1.5 days
▪ Unskilled	1.5 days

The tank is rendered both internally and externally to a thickness of 10mm (which varies due to the uneven surface caused by the angled bricks). A 5cm fillet is built up between the wall / base junction. The mix is 4:1 (sand:cement). The internal surface is then painted with a cement slurry.

Other (with detail to follow)
- A filter can be placed in the tank cover. This can be the usual bucket of gravel.
- A 2.5 metre length of pipe is required to run from the elbow in the base to the cover. This is slotted at 2m height to act as the the overflow (more detail to follow).
- Tap and galvanised pipe – I haven't yet given any information about the siting of the tap and the dugout for tap stand. Please make an estimate for this in the quotation. More information will follow

Strap dispenser

Strap showing crimps

Tank with straps in place (note the double thickness at the bottom – we do not want this any longer). The straps are painted with red oxide paint in this photo to prevent rusting as the tank will sit without render for some time during the testing phase.

Low-cost, thin-shell, ferrocement tank cover

Instructions for manufacture

Development Technology Unit
School of Engineering
University of Warwick
Coventry CV4 7AL
Tel: +44 (0)1203 522339
Fax: +44 (0)1203 418922
dgr@eng.warwick.ac.uk
http://www.eng.warwick.ac.uk/DTU/

November 2000

Introduction
The thin-shell ferrocement tank cover is designed in such a way that it can be manufactured without the use of a mould or shuttering. It can also be manufactured remote from the tank to which it is to be fitted and moved into place once complete. The aim is to reduce the cost of the tank (cover) by eliminating costly shuttering or moulds and by reducing the quantity of material used to manufacture the cover. It also means that the cover can be removed at a later date for maintenance, refurbishment or cleaning, should this be a necessity. The cover can be manufactured by two persons (one skilled and one unskilled) in a single day (with some time required after that for curing) using tools required for the construction of a simple cylindrical ferrocement tank.

The design is based on a frame known as a reciprocal frame, that has spokes that, when loaded, put little radial loading onto the structure on which it sits. The frame is covered with a wire mesh that is then rendered with a sand cement mix.

Details of the construction process are given here for a 2.0m diameter cover that has an inspection chamber opening of approximately 0.5m. The cover pitch is 25^{o}. Strength tests have proved acceptable up to this diameter. No guarantee is given for greater diameters. The spoke angles have to be recalculated for different diameters – this is one disadvantage of the cover design.

Benefits of the thin-shell ferrocement tank
- ♦ low cost – reduced use of materials
- ♦ no shuttering or mould required
- ♦ strong and lightweight – the tank cover is designed to be strong (through good quality control) and light at the same time
- ♦ good quality control can be achieved through easy working environment
- ♦ can be manufactured by two people in a single day (one skilled and one unskilled)
- ♦ no clambering on top of tanks required during construction
- ♦ can be cured easily – in the shade and at ground level
- ♦ can be batch produced at one site

Tools and materials required

Tools
- hacksaw
- pliers
- tin snips
- masons trowel (small)
- masons trowel (large)
- plasterers float
- shovel
- buckets (2)
- wheel barrow (optional)
- vice (handy if available)
- workbench (again, handy if available)

Table 1 – Materials required for cover

Item	Quantity
Cement (OPC) – kg	50
Sand – kg	150
6mm rebar – m	20
8mm rebar – m	20
Coffee tray mesh* - (0.9 wide) -m	4.8
Binding wire – kg	1
Basin for filter	1
Labour (skilled) – days	1
Labour (unskilled) – days	1

* or chicken mesh (twice the quantity required)
Other – plastic sheet 4 x 4m – (reusable)

Table 2 – Costs of materials (based on manufacture of cover in Uganda, July 2000)

	Number req'd	Cost per unit Ush	Total cost Ush	Total £
Cement (OPC) – kg	50	300	15000	6.76
Sand – kg	150	20	3000	1.35
6mm rebar – m	20	230	4600	2.07
8mm rebar – m	20	385	7700	3.47
Coffee tray mesh - (0.9 wide) -m	4.8	4350	20880	9.41
Binding wire – kg	1	2000	2000	0.90
Basin for filter	1	1000	1000	0.45
Labour (skilled) – days	1	5000	5000	2.25
Labour (unskilled) – days	1	3000	3000	1.35
		GRAND TOTAL	62180	28.01
		Total materials	54180	24.41
		Total labour	8000	3.60

Instructions for Manufacture

Stage 1 – making the frame

- Choose a location with plenty of space to work. The procedure requires bending long lengths of reinforcing steel and so a clear working area is essential. Also a ground space of 2m diameter will be needed where no other activity will be carried out for a week (while the cover is cured). Preferably choose a covered area, so that curing can take place out of the sun and/or rain.

- The first step is to set up a jig for bending the reinforcing bar. The jig is made up of two steel pegs or nails about 5cms long, set about 5cm apart. The steel is placed in the jig and bent as shown in Figure 1. The jig needs to be fixed so that it cannot move when the steel is bent. A workbench is ideal where the pegs can be put into the vice. Alternatively the pegs can be driven into a heavy piece of timber and this arrangement can be used effectively. Steel re-bar (8mm) can be used to form the pegs, but slightly heavier steel is better.

Tip:
When bending the re-bar it does not bend exactly where it makes contact with the jig peg. The bending takes place a cm or two on the pulling side. This has to be allowed for when bending. The bending radius can be quite large because of the thickness of the steel. This doesn't present any real problems here.

♦ The next step is to bend the 8mm reinforcing steel into hoops. Four hoops, diameter 0.55m, 1.0m, 1.5m and 2.0m are required. To make the procedure easy, a peg can be knocked into the ground and used as a centre around which the four circles can be drawn using string and a marker (also mark the positions of the 8 spokes at 45^{o} intervals for later use). The steel can then be bent gently in the jig to match the circles. The hoops ends are tied with two or three pieces of tie wire. For this the steel is cut slightly oversize to allow for tying. The cutting lengths are given in Table 3. Where the cover is to be fitted to an existing tank the outer hoop should be bent to fit the mean radius of the top of the tank wall and any irregularities in the shape should be taken into consideration.

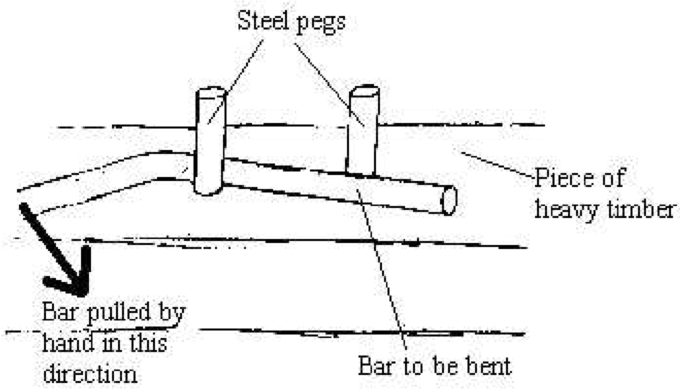


Figure 1 – Jig for bending steel reinforcing bar

Figure 2 – Workbench fitted with bending jig – the angles for the spokes are marked on the jig to make the process quicker and easier

Figure 3 – Marking out the hoop diameters before bending

Figure 4 – Tying the hoops using tie wire and a nail

Figure 5 – One of the spokes

Table 3 – Cutting list for steel hoops (8mm steel)	
Diameter	Steel cutting length (add 0.2m for overlap for tying in all cases)
0.55m	1.72m (1.92m)
1.0m	3.14m (3.34m)
1.5m	4.71m (4.91m)
2.0m	6.28m (2.48m)

- At this point all but the outer (largest) hoop can be put aside until later.
- The next step is to bend the spokes. These are from the **6mm steel** bar. There are eight in number and are bent in the jig to the dimensions shown in Figure 7. The cutting length is 1.33m. To aid the bending, the angles can be marked out on the jig (see Figures 1 and 2) beforehand and then the bent steel can be matched against the marked angles. The angles to mark are:

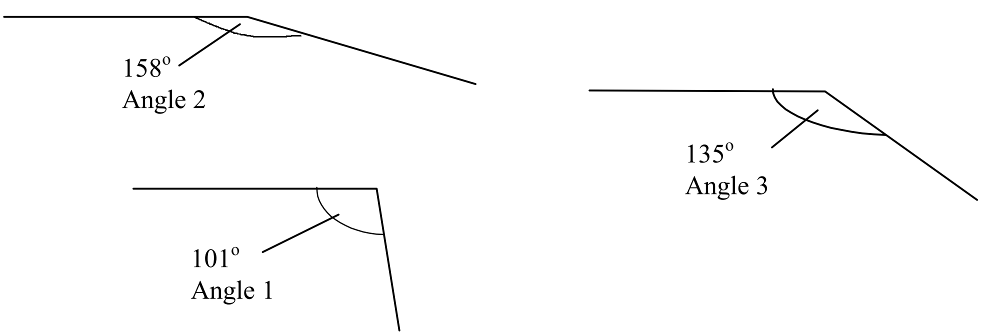


Figure 6 – Angles for spoke bending

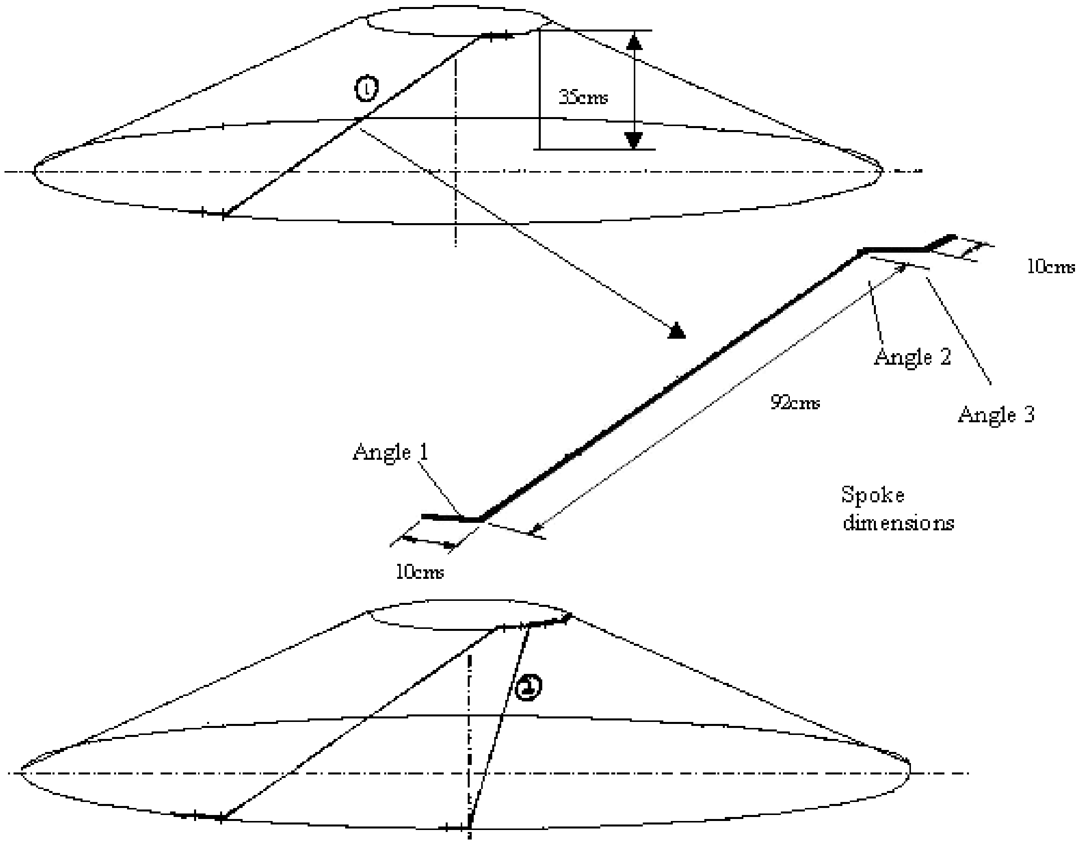


Figure 7 - Dimensions and locations of spokes

- ♦ It is recommended that Angles 1 and 3 are bent first. These are bent in the same plane. The spoke is then turned through 90^{o} and Angle 2 is bent.
- ♦ Eight secondary spokes are also cut from 6mm steel bar to a length of 75cm. These are wired to the frame as shown in Figure 8 and support the mesh to reduce the 'panel' size.

Figure 8. Assembly of the frame

- Now the spokes are placed one by one inside the outer hoop (as shown in Figure 8.) to slowly form the cover frame. It is convenient to have the outer hoop sitting on the ring marked out earlier with the position for the 8 spokes marked also. THE INNER RING IS NOT USED AT THIS POINT – There is no inner ring. This is made up as the separate spokes are joined together. (See Figure 9). Spoke one is placed on a support (a box or piece of wood) which is 35cm high. This is the height of the frame from the ground to the plane of the circular access hatch.

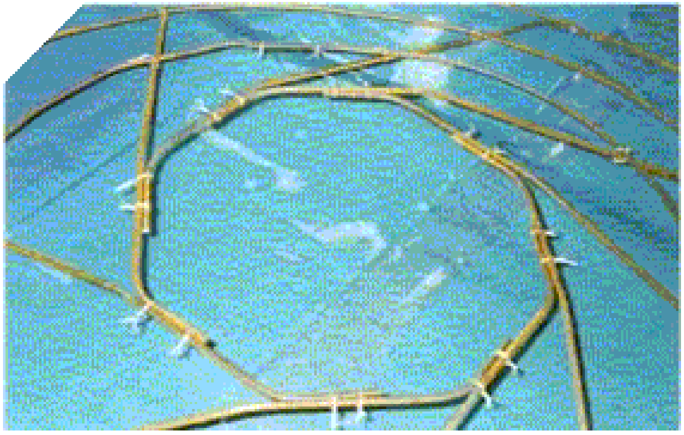


Figure 9. Showing the formation of the inner ring from individual spokes

- Tie the first spoke to the inner side of the outer hoop as shown in Figure 10.

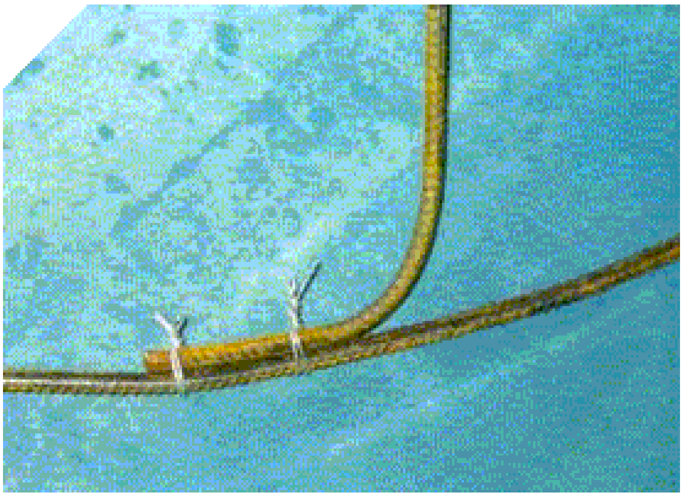


Figure 10. Showing arrangement for tying spoke to outer hoop.

- Place the next spoke 45^{o} around the perimeter hoop (these spacings were marked earlier) and tie it to the first spoke as shown in Figure 9. Continue in this way until the final spoke is tied to the first spoke and all eight spokes are in place.
- Put the two inner hoops in position and tie them in place (Figure 8). The small inner hoop that was formed earlier will be used when the access hatch lip is made later.
- The frame in now ready to have the chicken mesh or coffee tray mesh attached.

- Use chicken wire (0.5 inch mesh size) or coffee tray mesh (4mm mesh size) of 0.9m roll width. If chicken mesh is used, 9.6 metres length is required and two layers of chicken netting are applied. If coffee tray mesh is used then only one layer and 4.8m length are required.

- Eight pieces of chicken wire or 4 pieces of coffee tray mesh are cut to the dimensions shown in Figure 11. Two pieces can be cut from a 2.4m length of netting if cut as shown. A template can be drawn on the ground to aid cutting.
- Coffee mesh is easier to use, is firmer, and due to the small mesh size very little mortar is lost through the mesh during rendering. It is approximately twice the price per metre length and so the overall cost is similar as only one layer is required.

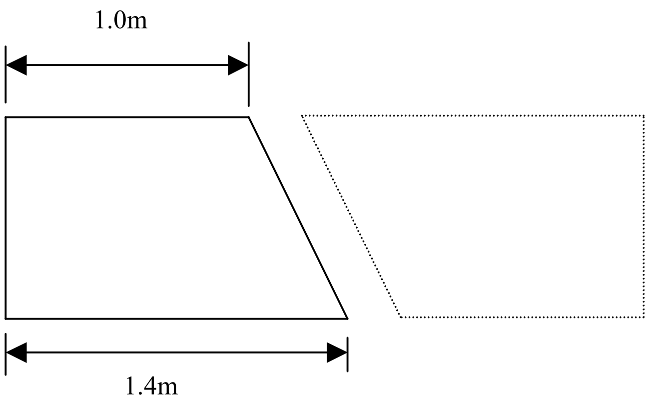


Figure 11. Cutting size for chicken mesh

- The pieces of netting are placed on the frame as shown in Figures 12 and 13 and the overlapping edges tied in place, pulling the wire as tight as is possible without distorting the mesh.

> Tip: for chicken mesh, a screwdriver can be used to pull the loose wires or end loops through holes in the mesh to tie the mesh in place. Use the rough edges of the netting to tie the folded edges into place. Use as little tie wire as possible at this point, as the netting will tied securely when the second layer is in place.

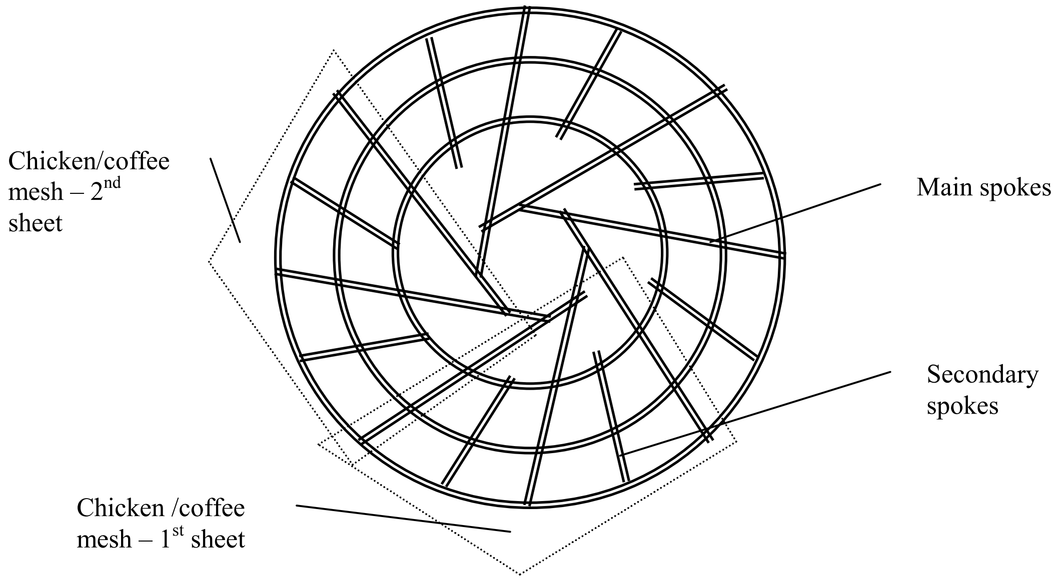


Figure 12. Pattern for application of chicken mesh

Figure 13. Applying the chicken mesh.

- ♦ When the first layer of chicken mesh is complete start the second layer one spoke (90^{o}) out of phase with the first and complete in the same manner.
- ♦ Carefully check that the mesh is as flat as possible and, if using chicken mesh, that both layers are close together. Tie the netting at regular intervals using the tie wire so that the netting is close to the steel bar. Bend all tie wires into the plane of the cover. Remember that we are trying to keep the cover as thin as possible and protruding wire ill have to be covered with mortar.
- ♦ The cover is now ready for the rendering (Figure 11).

Stage 2 - Rendering the cover

- ♦ It is important to use good quality materials and to maintain good standards of workmanship throughout the rendering process. The aim is to apply a layer of mortar to the chicken mesh that is as thin as possible. This, in practice, will vary between about 15mm and 25mm with an average thickness of about 20mm. The first coat is applied from the top and second coat applied from below.
- ♦ Put a plastic sheet on the ground so that render mix which falls through during rendering can be reused.
- ♦ Elevate the frame so that work can be carried out from above or below. Waist height is most suitable. The frame should be raised on 6 posts or boxes so that it is stable and can withstand the forces applied during rendering. A support should also be placed in the centre to prevent the centre sagging under the weight of the render (see Figure 14).

Figure 14 – the completed cover frame set on 6 supports and ready for rendering

♦ Render preparation: a mix of 1:3 (cement:sand) is used. A sharp sand should be used i.e. not a fine sand but sand with a moderately large grain size. There should be no silt or other contaminant in the sand. Ordinary Portland Cement (OPC) is used. The quantities should be carefully measured using a container – a bucket for example (do not measure using a shovel as this can be very inaccurate).
♦ The consistency of the render is very important. It should be dry enough not to fall through the netting while being plastic enough to be workable with a trowel. A mortar plasticiser will improve the workability of the render.
♦ Adding a plasticiser means that the water:cement ratio can be kept low while still keeping the render plastic. This ratio should be kept to approximately 0.4 by weight (i.e. 10 parts cement to 4 parts water by *weight*). Low water content not only gives a render which is easily applied to the mesh, but also gives improvements in strength and permeability of the cured render. In practice it is difficult to control the water:cement ratio because there is usually an unknown quantity of water in damp sand and plasticity is often achieved before the minimum measured ratio is met. The practical method involves experimentation to achieve the desired plasticity with minimum water content. The plasticiser should be used according to the manufacturers instructions.

Figure 15 – applying the top coat of render to the cover

♦ Keep mixes small because the render 'goes off' quickly. It may be wise to make a dry mix which is sufficient for the whole job and then add water to small amounts as required.
♦ Applying the render: this is fairly simple. Use a plasterers float and a small trowel. Put the float behind the mesh and work the mortar through the mesh onto the float as shown in Figure 12. Wipe the float away so that the mortar is slightly smoothed on the underside. Work small areas – take one 'panel' at a time and complete it. Some of the mortar will fall through onto the plastic sheet – this can be picked up immediately for reuse. Remember that the aim is to apply a very thin layer of mortar. The technique can be easily learned with a little practice.

♦ A basin of approximately 0.5m diameter is used to form the access hatch. The basin can later be left in situ and can act as the filter. These basins usually have sloping sides and so the basin can be inserted until it fits tightly.
♦ A lip is then built up around the basin to about 50mm deep and 50mm wide. The remaining steel hoop is built into the lip.

- Once the first layer of mortar has been applied the cover should be left for a day to allow the render to gain strength.

Figure 16 – showing the lip being built up around the basin – note the steel hoop in place

- After one day the underside can be rendered. Again use a 1:3 mix and keep the render quite thin, just covering the steel bars and wire mesh.
- The cover is then cured for 7 days. The tank should be wetted twice daily and covered with plastic sheeting to prevent evaporation of the curing water. It is essential that curing is carried out properly.
- A coat of 'nil' (pure cement water slurry) can be applied to top and bottom after two days of curing.

Figure 17 – Showing the underside of the cover after rendering is completed

Figure 18 – Curing the cover under plastic. The cover is wetted regularly during curing.

Putting the cover in place on the tank

- When the cylindrical tank body is being constructed, some thought should be given to the method of fixing the cover to the tank. If the cover is to be fitted to a thin walled ferrocement tank four (or more) tie wires should be left protruding from the tank wall and these are tied to the cover when it is in place. For brick, block or masonry walls, the cover can be laid on a bed of stiff mortar and then blended with the tank as shown in Figure 19.

- The cover can be lifted into place by four or six strong people. Special care should be taken not twist the cover or put any undue stress on it as this could cause it to crack.
- If the tank wall is quite high then a raised platform should be constructed (from earth or timber) to stand on.

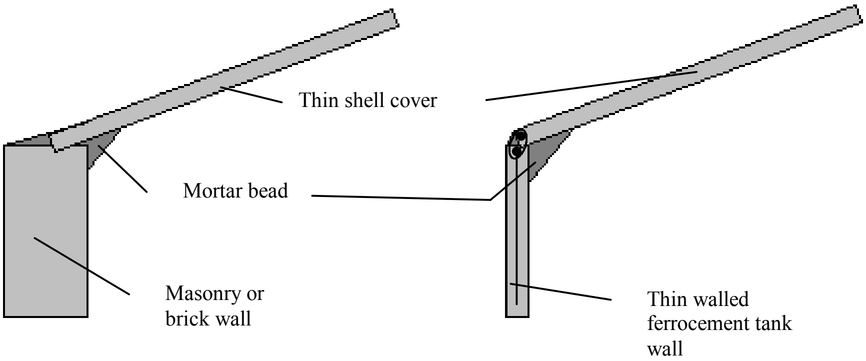


Figure 19 - Blending the cover with the tank wall.

Figure 20 – A completed cover fitted on a partially below ground tank in Uganda

Tank testing

Tests were carried out on the tank cover in the UK. The cover was uniformly loaded to 1000kg and there was minimal deflection. It was also point loaded to 160kg, again with minimal deflection and no visible sign of cracking or damage.

Figure 21 - Tank cover loaded to 1000kg

DTU Technical Release Series TR-RWH06

Ferro-Cement Jar

Instructions for manufacture

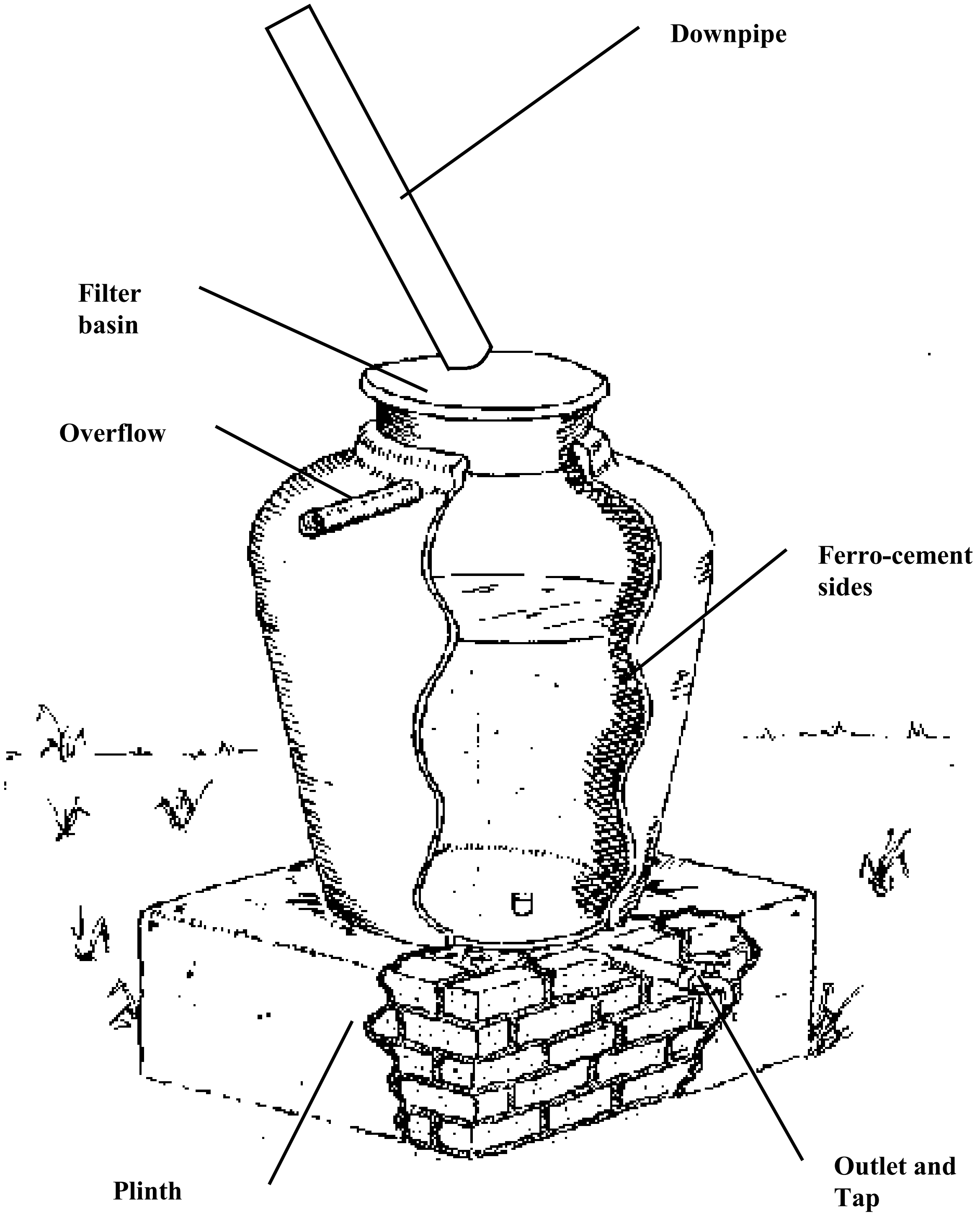
(Based on the construction of a Ferro-cement Jar at Kyera Farm, Mbarara, Uganda)

Prepared by Dai Rees and Vince Whitehead

Development Technology Unit
School of Engineering
University of Warwick
Coventry CV4 7AL
Tel: +44 (0)1203 522339
Fax: +44 (0)1203 418922
dgr@eng.warwick.ac.uk
www.eng.warwick.ac.uk/DTU/

September 2000

Contents:



Cutaway drawing of a ferrocement tank

1. Introduction

This manual gives guidelines for the manufacture of a 500 litre Ferro-cement Jar, which was based on a jar built at Kyera Farm, Mbarara, Uganda during June and July 2000.
The jar basically consists of a brick plinth, a Ferro-cement shell, and a filter basin.
This is a well proven technology that has been successful in Thailand, these were traditionally made from a rendered bamboo mould but now they use chicken mesh, 10 million jars were built in Thailand between 1985 and 1992.

2. Merits and drawbacks of the Ferro-cement Jar

Pros:

- Has the potential for small/large scale production by artisans.
- Very low maintenance.
- Repairs can be easily carried out.
- Low cost.
- Suitable for most ground conditions.
- Good protection against mosquitoes.

Cons:

- The level of skill required is quite high (based on recent experience in Kyenjojo, Uganda)

3. Ferro-cement Jar specifications

Table 1 gives the specifications for the jars main features:

Table 1 Ferro-cement Jar specification

Jar external diameter	0.84m (approx. at the widest point.)
Jar internal diameter	0.80m (approx. at the widest point)
Jar height	1.25m (approx.)
Jar capacity	Approximately 500 litres
Jar lining (inner surface)	Waterproof render
Wall thickness	20mm
Wall composition	Chicken mesh sandwiched between inner and outer coat of render
Water extraction	Gravity via tap
Plinth	Brickwork 0.8 x 0.8 x 0.5m
Top	PVC filter basin

N.B. See page 11 for Jar and plinth dimensions

4. Material and labour requirements

Table 2 Material and labour requirements for the jar.

	Units	Ring beam	Base	Infill	1st coat	2nd coat	Water extraction	Totals
Cement (OPC)	kg	10	12.5	6	20	20		68.5
Sand	kg	20	75	30	60	60		245
Aggregate <50mm	kg	40						40
Bricks (household size)	no		100					100
Rubble	kg			75				75
Chicken mesh 0.5" x 0.9m roll width	m					3.2		3.2
GI Pipe 1"	m						0.5	0.5
GI Elbow 1"	no						1	1
PVC Pipe 1.25"	m						0.5	0.5
Tap 0.5"	no						1	1
Reducer 1" - 0.5"	no						1	1
Basin	no						1	1
Labour (skilled)	days							2
Labour (unskilled)	days							4

5. Tools and equipment required

- Spade or shovel
- Hoe
- Spirit level (600mm)
- Bucket
- Trowels
- Plasterers float
- Tape measure (3m)
- Tin snips or wire cutters (for mesh)
- Pipe wrench

6. Jar Costing

Table 3 gives a breakdown of the materials used for the jar and their costs

Table 3 Jar costing

Item	Unit	No reqd	Unit cost	Total (UGS)	Total (US$)	Total (£)
Cement	kg	68.5	300	20550	13.70	9.26
Sand	kg	245	20	4900	3.27	2.21
Aggregate <50mm	kg	40	25	1000	0.67	0.45
Bricks	no	100	42	4200	2.80	1.89
Rubble	kg	75	0	0	0.00	0.00
Chicken mesh 0.5"	m	3.2	1667	5334.4	3.56	2.40
GI Pipe 1"	m	0.5	4200	2100	1.40	0.95
GI Elbow 1"	no	1	1500	1500	1.00	0.68
PVC Pipe 1.25"	m	0.5	1667	833.5	0.56	0.38
Tap 0.5"	no	1	5000	5000	3.33	2.25
Reducer 1" - 0.5"	no	1	1500	1500	1.00	0.68
Basin	no	1	1000	1000	0.67	0.45
Labour (skilled)	days	2	5000	10000	6.67	4.50
Labour (unskilled)	days	4	3000	12000	8.00	5.41
	Material costs			47918	31.95	21.58
	Total cost (incl. labour)			69918	46.61	31.49
	Cost per litre stoarge			96	0.06	0.04
	Cost per litre storage (incl. labour)			140	0.09	0.06

Notes

- Mould cost not included - cost of mould is approx. 6000UGS (US$3.38) and may last for up to 10 or 15 jars depending on care taken during manufacture
- Larger sizes of jar - say up to 1500 litres - can be achieved by experimenting with the mould size
- Sawdust can be obtained from local sawmills
- The volume is obtained by using a bucket of known volume and counting the appropriate number of buckets of sawdust
- Some transport costs included (i.e. for sand, aggregates and bricks)
- Cost of bucket slab not included

7. Site selection

It is important to select the right site for the jar so that it will remain a reliable source of water for years to come.

Some pointers for what constitutes a good site are given below:

- Good ground stability (i.e. not sandy soils).
- Jar should be close enough to the dwelling to avoid long lengths of guttering and downpipe (some suggest siting the jar mid way along the length of a building to reduce gutter size– this is fine if water from one side of the building only will be fed into the jar).
- Reasonably flat where possible – otherwise the ground will have to be levelled before marking out.
- Away from trees which may undermine the foundations and cause cracking.
- Away from areas where animals will wander – fence off if needed.
- Not so close to the dwelling that the foundations are undermined.
- Somewhere convenient for extracting water e.g. close to the kitchen area.
- It must be a suitable distance away from vehicle access as this may cause ground movement, fence off if necessary.

8. Manufacturing procedure for the jar

- Prepare a level piece of ground approximately 1.0m square.
- Mark out an inner and outer square with sides of 0.85m and 0.0.55m respectively as shown in Figure 1.

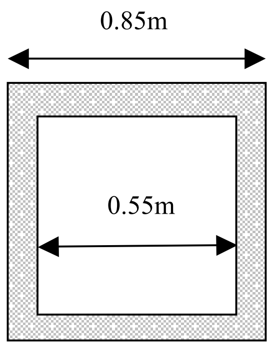


Figure 1 Inner and outer squares for the plinth foundations

- Carefully excavate between the two squares to a depth of 10cm. (NB if soil tends to be unstable excavate to 20 cm deep and fill with aggregate 10cm deep, alternatively make a larger deeper ring beam)
- Fill the excavation with a concrete mix of 4:2:1 (aggregate: sand: cement) and cover with damp grass/leaves and leave for two days.Start building the brickwork, with a plinth outer size of approximately 0.8m, on top of the concrete ring using a mortar mix of 5:1 (sand: cement) to a height of 0.55m as shown in Figure 1. Leave a gap in the top course of bricks for the outlet pipe.

- Fill the centre of the plinth with rubble/aggregate, compacting it well to prevent later settling.

Figure 1 Brickwork plinth

- Cut off a length of ∅3/4" GI pipe and fit on a ∅3/4" elbow at one end and a ∅3/4" to ∅1/2" reducer and a ∅1/2"tap at the other. Local plumbers can thread the GI pipes, alternatively a low cost method of threading ∅3/4" PVC pipe is explained in Appendix 1.
- Cut off a piece of pipe 25mm long pipe that has been threaded at one end and fit this to the elbow. This will give the jar a settling zone, but it can also be removed for washouts.
- Dig out a channel in the rubble and place the pipe in so that the tap and the elbow are vertical. N.B. make sure the top of the elbow is 3cm above the level of the rubble.
- Apply a 1cm layer of mortar to the top of the rubble and flush with the edges of the plinth, apply a layer of damp material and cover with plastic sheet, leave this to cure.
- Prepare a disc of mortar 0.6m diameter by 1cm deep on to the plinth base.
- Cut out a ring of chicken mesh to a diameter of 0.75m and turn up the edges 0.1m, so that it leaves a base diameter for the jar of 0.55m.
- While the disc of mortar is still damp lay the chicken mesh on to the top of this and then apply another layer 1cm deep to the mesh this should be level with the top of the elbow.

- Cut out and sew up a polypropylene or hessian sack to the dimensions shown in Appendix 2.

Figure 2 The filled polypropylene sack mould

- Fill the sack mould with sawdust (rice husk, coffee husks, sand or any other similar available material) and compact it as it is being filled. When full tie it up at the top and pat the sides to produce an even symmetrical shape then place this on the prepared base of Ferro-cement as shown in Figure 2. The capacity of the jar can be determined by filling the sack with buckets of known volume, simply count the number of buckets to until 500 litres is achieved.
- Place a medium sized basin centrally on top of the mould with a short length of PVC pipe for the overflow. Apply the first coat of mortar of 3:1 mix with a waterproof additive (quantity as per manufacturers instructions) to a thickness of 1cm as shown if Figure 3, (the thickness may vary as the irregularities of the mould shape is compensated for)

Figure 3 Overflow, filter basin and first coat application

- When the first coat has been applied cover with a damp cloth and a plastic sheet, leave for 24 hours, making sure the cloth is kept damp.
- Wrap around the jar a suitable length of chicken mesh to cover the whole area up to the filter basin pull the mesh tight and fix with short lengths of wire to secure in place as shown in Figure 4.
- Apply the second coat of mortar (make this layer smooth by rubbing over with a small wooden hand float) also apply mortar around the tap outlet to secure it to the base and jar side as shown in Figure 5.

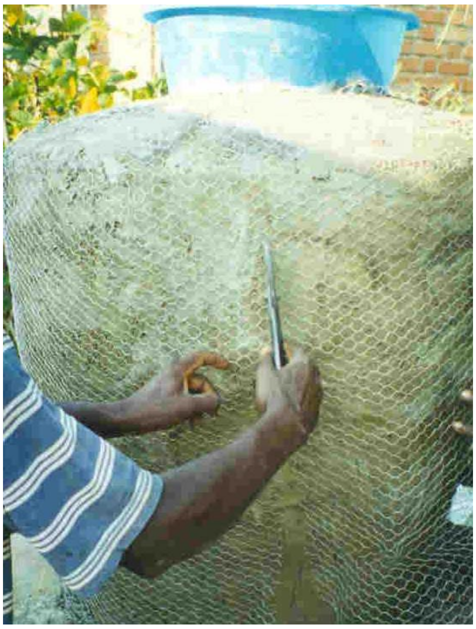


Figure 4 Applying the mesh on top of first coat

- Cover the shell again with a damp cloth and a plastic sheet, leave for 48 hours.
- When the shell is firm carefully remove the filter basin and untie the mould bag, and carefully take out the contents.
- Fill the bottom of the jar with enough water to come up to the level of the outlet pipe and replace the damp cloth with the plastic sheet and leave for 7 days to cure.
- Make a series of about twenty holes in the bottom of the basin using a hot nail or a drill (Ø6mm) for the rain to enter the jar. Fill the basin with aggregate (about 20mm) to about one third full.
- Cut out a section of clean cloth that will cover the basin and have sufficient to overhang down the sides, then tie around the sides of the basin with string or rubber inner tube strip to hold the cloth in place.

Figure 5 Detail of the outlet fixing

- Lower the basin on to the top of the jar.
- Fix the appropriate gutter to the roof and place the downpipe directly on to the cloth.
- Cut out a small piece of mosquito mesh and fit this on to the end of the overflow pipe with string/twine etc.

9. Care of the jar on completion

Once the jar is finished and has cured for 7 days, fill with 125mm (5”) of water each day so that the structure is gradually loaded rather than all at once.
It may be noticed that the jar will leak slightly somewhere around the sides, it is best if this is left for at least a week, as quite often the jar will eventually seal itself if the hole is small (based on masons personal experience in Uganda). However, if it shows no sign of sealing or the hole is quite visible, empty the jar and make the repair with a nil mix (i.e. purely a cement and water mix) from the inside of the jar.
It is important that there are no gaps around the filter and cover, or around the filter where light can enter as this will not only encourage the growth of algae but may be an entry point for mosquitoes both of which should be avoided. The water may have a cement taste at first so either rinse out well several times or use the jar for washing rather than cooking for the first few times.

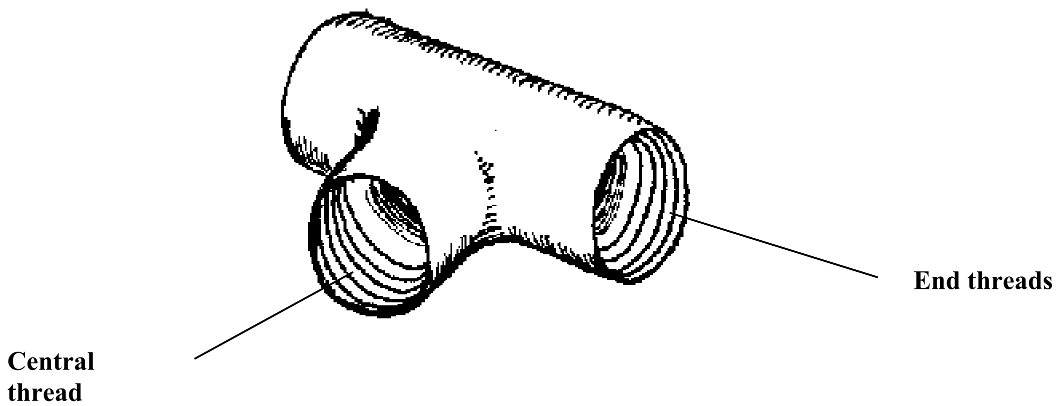
Appendix 1: Low cost threading of PVC pipes using a standard galvanised iron (GI) pipe fitting

There are many occasions were threads are required on PVC pipes so that other fittings can added to the pipe. This often involves the use of expensive threading equipment, which is not always available when needed and the charge for this service can become expensive when it is done repeatedly.
The method described here was tried out in Uganda after finding the problems mentioned above and was found to be a useful and successful solution that was very low cost. Though it requires some tools, a little bit of skill and some patience, once it has been made it will last for many threading operations and re-sharpening is simple to do.
The following example is for a ∅½” PVC pipe but the same procedure is carried out for other sizes:

Tools required:

One hacksaw blade (preferably with 24 teeth per inch)
A small (6”long) triangular file (the width across the faces should preferably be no more than about 3/16”or 4mm)
Pipe grips or a vice.
10” rough flat file.

The reliability of the threads for higher pressure applications has not been checked and care will be needed when trying this out.



1. Take a normal GI ∅½” Tee fitting as shown in Figure 1 and make three equally spaced saw cuts with the hacksaw blade in the central part of the Tee to just beyond the roots of the thread as shown in Figure 2.

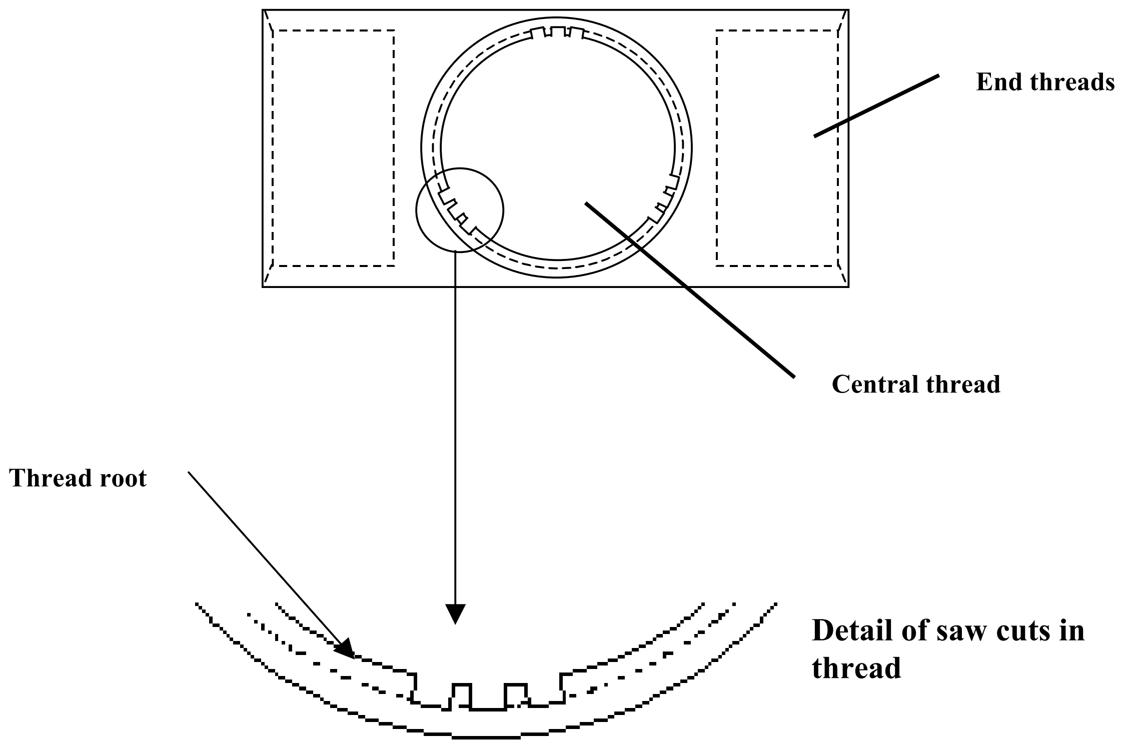


Figure 2 The GI Tee with the saw cuts equally spaced round the central thread

2. Make additional saw cuts as close as is practically possible to the first thread so that it is slightly wider than one of the faces of the triangular file, this is to ease the burden of filing.
3. Proceed to file each of the saw cuts so that the roots of the thread can no longer be seen.
4. File the left-hand side of the slot, as this will be the cutting edge, so that the profile is the same angle as the file i.e. 60° as shown in Figure 3.

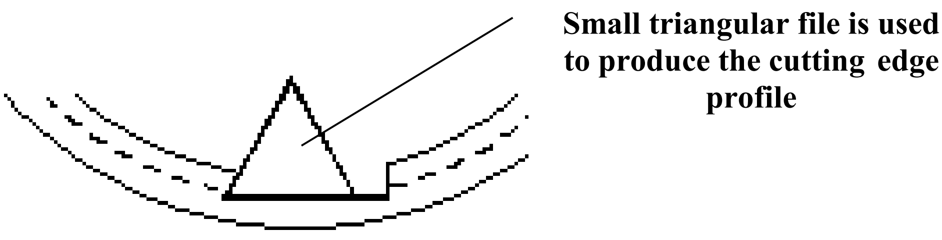


Figure 3 The GI Tee with detail of the cutting edge profile on thread

5. Using a rough file chamfer the end of the pipe to be threaded to the dimensions shown in Figure 4.

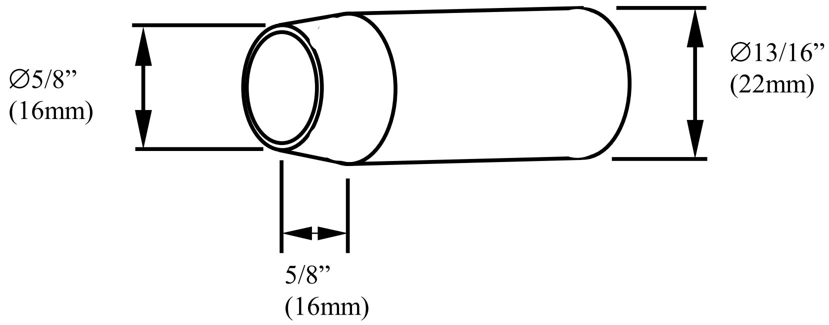


Figure 4 Chamfer dimensions for the ∅1/2” PVC pipe

6. Hold the pipe firmly in an upright position (using a vice or pipe grips) and apply a generous amount of grease or Vaseline to the thread cutter or to the pipe end. This will reduce the friction while thread cutting.
7. Place the thread cutter on top of the pipe and gently start to turn/thread it on to the prepared pipe, making sure that the top of the GI fitting is level as shown in Figure 5.

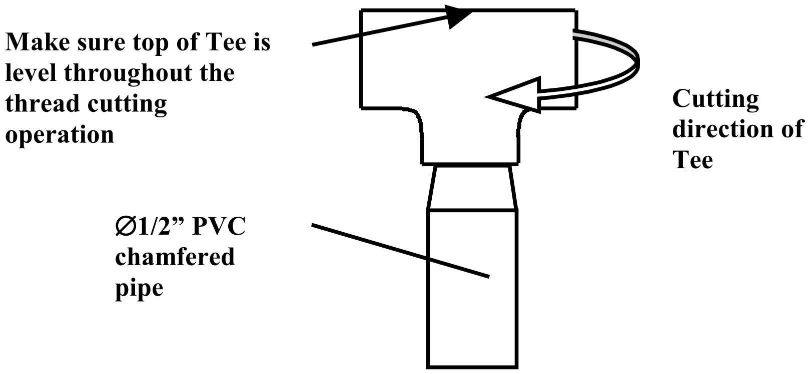


Figure 5 Starting the thread cutting

8. Turn several times (by inserting a screwdriver or steel rod through the Tee) then remove and clean out the thread of any plastic that has built up. For every revolution of the thread cutter turn back again half a revolution, this will break off the material being cut and avoid clogging of the cutting edge space.
9. Repeat the operation until a sufficient length of thread has been cut.

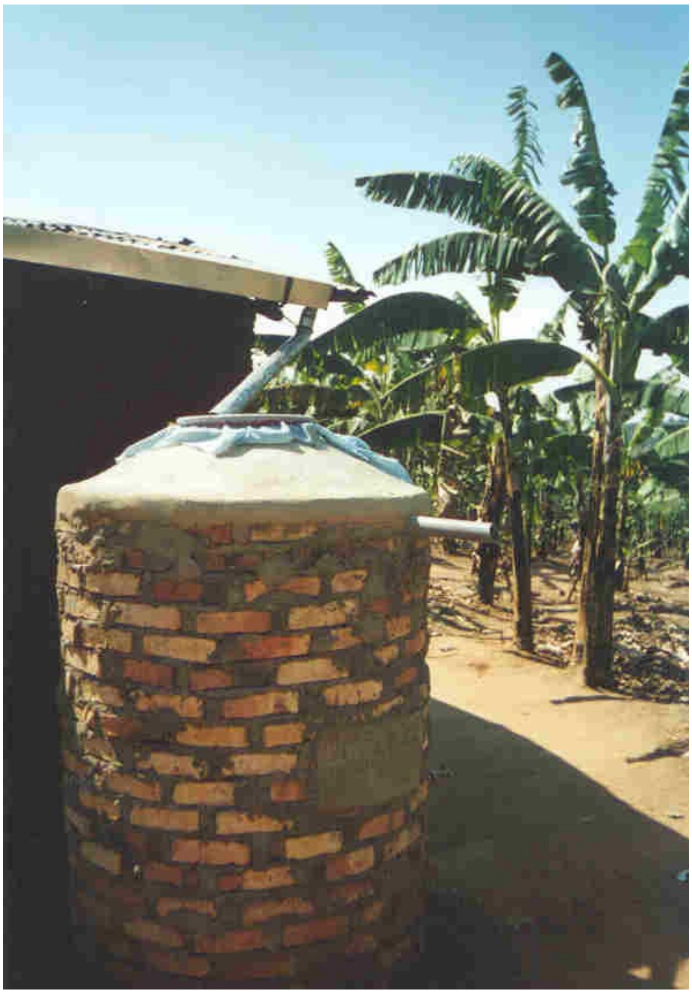
Re-sharpening the cutter is simply done by filing the cutting edge with the small triangular file until the blunt edge has been removed.
Polythene bags cut in to thin strips and wrapped round the thread is a good low cost substitute for PTFE tape.
Please note that it may take several attempts before a satisfactory thread has been made so practice on spare pieces of material until confidence and the quality is built up.

DTU Technical Release Series **TR-RWH07**

Brick Jars

Instructions for manufacture

(Based on the construction of a Brick Jar at Kyera Farm, Mbarara, Uganda)

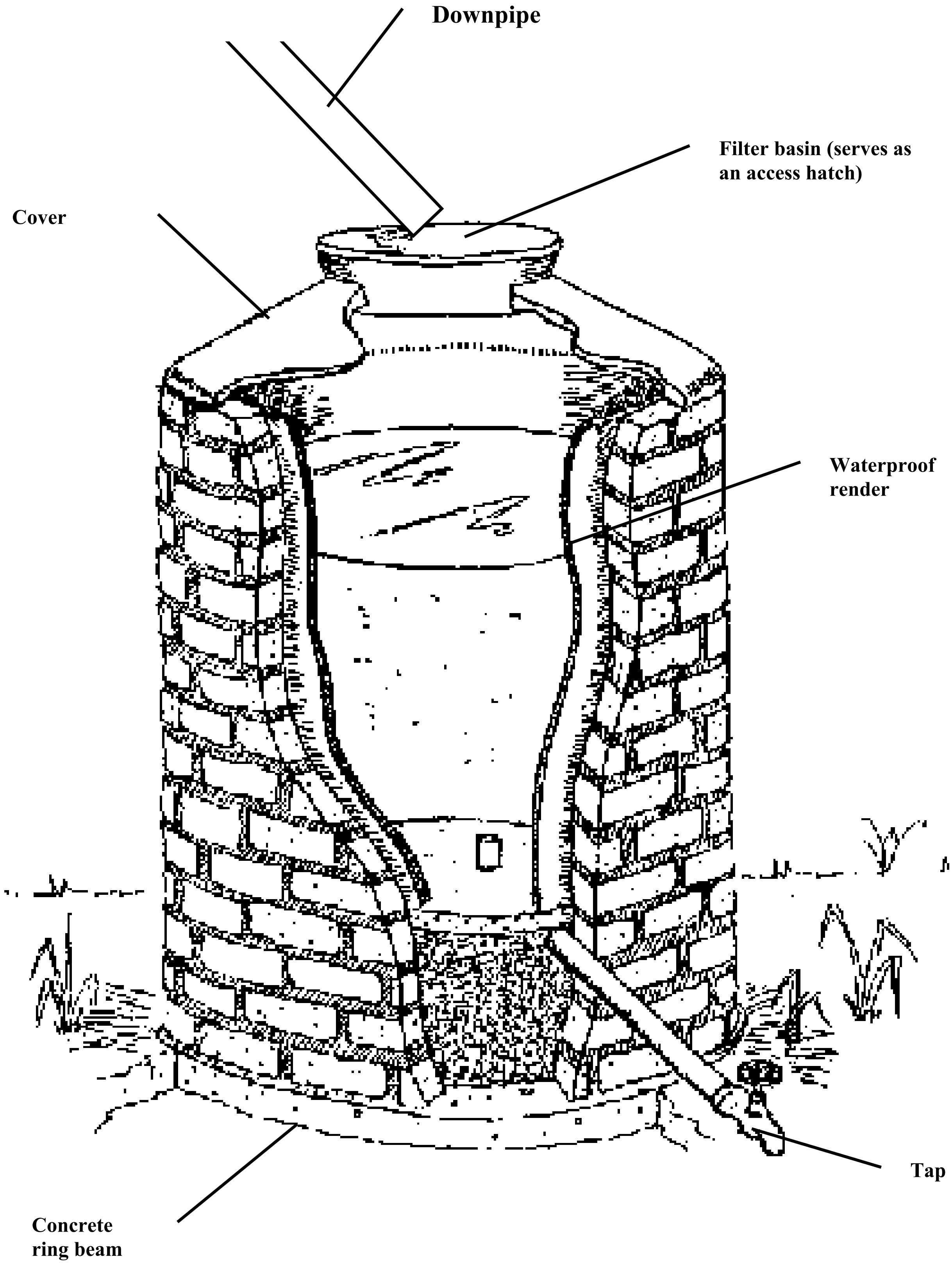


Prepared by Dai Rees and Vince Whitehead

Development Technology Unit
School of Engineering
University of Warwick
Coventry CV4 7AL
Tel: +44 (0)1203 522339
Fax: +44 (0)1203 418922
dgr@eng.warwick.ac.uk
www.eng.warwick.ac.uk/DTU/

September 2000

Contents:



Cutaway sketch of the Brick jar

Introduction

This manual gives guidelines for the manufacture of a 700ltr Brick Jar, which was based on a jar built at Kyera Farm, Mbarara, Uganda during June and July 2000.
The jar consists of a brick outer section, a waterproof internal render and a thin mortar cover with a filter basin.

1. Merits and drawbacks of the Brick Jar

Pros:

- Low manufacturing time.
- Very low maintenance.
- Repairs are easily carried out.
- Suitable for most ground conditions

Cons:
- Cost per litre storage is higher than the Plastic Tube tank

2. Jar specifications

Table 1 gives the specifications for the jars main features:

Table 1 Brick Jar specification

Jar internal diameter	1.0m
Jar external diameter	1.2m
Jar height	1.6m
Jar capacity	Approximately 750 litres
Jar lining	Waterproof render
Water extraction	Gravity via tap
Top	Mortar shell with PVC filter basin

3. Material and labour requirements

Table 2 Material and labour requirements for the jar

	Unit	Ring beam	Walls	Infill	Render	Water Extraction	Cover	Totals
Cement	kg	10	37.5	10	25		10	92.5
Sand	kg	20	225	40	75		30	390
Aggregate <50mm	kg	40						40
Bricks	no		300					300
Rubble	kg			75				75
GI Pipe 1"	m					0.5		0.5
GI Elbow 1"	no					1		1
PVC Pipe 1.25"	m					0.5		0.5
Tap 0.5"	no					1		1
Reducer 1" - 0.5"	no					1		1
Basin	no					1		1
Labour (skilled)	days							2
Labour (unskilled)	days							4

4. Tools and equipment required

- Spade or shovel
- Hoe
- Tape measure (3m)
- Spirit level (600mm)
- Bucket
- Trowels
- Pipe wrench

5. Jar costing

Table 3 Jar costing

Item	Unit	No reqd	Unit cost UGS	Total UGS	Total US$	Total £
Cement	kg	92.5	300	27750	18.50	12.50
Sand	kg	390	20	7800	5.20	3.51
Aggregate <50mm	kg	40	25	1000	0.67	0.45
Bricks	no	300	42	12600	8.40	5.68
Rubble	kg	75	0	0	0.00	0.00
GI Pipe 1"	m	0.5	4200	2100	1.40	0.95
GI Elbow 1"	no	1	1500	1500	1.00	0.68
PVC Pipe 1.25"	m	0.5	1667	833.5	0.56	0.38
Tap 0.5"	no	1	5000	5000	3.33	2.25
Reducer 1" - 0.5"	no	1	1500	1500	1.00	0.68
Basin	no	1	1000	1000	0.67	0.45
Labour (skilled)	days	2	5000	10000	6.67	4.50
Labour (unskilled)	days	4	3000	12000	8.00	5.41
	Material costs			61083.5	40.72	27.52
	Total cost (incl. labour)			83083.5	55.39	37.43
	Cost per litre storage			81	0.05	0.04
	Cost per litre storage (incl. labour)			111	0.07	0.05

Notes
1. Some transport costs included (i.e. for sand, aggregates and bricks)
2. Cost of bucket slab not included

6. Site selection

It is important to select the right site for the jar so that it will remain a reliable source of water for years to come.
Some pointers for what constitutes a good site are given below:

- Good ground stability (i.e. not sandy soils)
- Jar should be close enough to the dwelling to avoid long lengths of guttering and downpipe (some suggest siting the jar mid way along the length of a building to reduce gutter size– this is fine if water from one side of the building only will be fed into the jar)
- Reasonably flat where possible – otherwise the ground will have to be levelled before marking out
- Away from trees which may undermine the foundations and cause cracking.
- Away from areas where animals will wander – fence off if needed
- Away from areas where surface water will gather (i.e. depressions)
- Not so close to the dwelling that the foundations are undermined
- Somewhere convenient for extracting water e.g. close to the kitchen area.

- It must be a suitable distance away from vehicle access as this may cause ground movement, fence off if necessary.

7. Manufacturing procedure for the jar

7.1 Making the jar cover

This will take a week to cure so this needs to be made first.

- Prepare a level piece of ground of 1.1m diameter (same as jar outside diameter) on which to make the cover for the jar.
- Mound up and compact soil on the prepared ground to a height of about 25cm and make a well in the centre for the filter basin as shown in Figure 1.

Figure 1 Mould for brick jar cement cover

- Cover the mound with plastic sheeting and place a series of bricks around the mound at a diameter of 1.1m as shown in Figure 2.

Figure 2 Mould ready for the cement render

- Render the plastic sheeting with a mortar mix of 3:1 (if the sand is low quality use chicken mesh on sandwiched between two layers of render) cover with grass or leaves and keep damp until cured which will take about seven days.

7.2 The Brickwork section

- After finding a suitable location for the jar make sure the ground is level for 1.5m in diameter.
- Place a stake in the centre of the area and mark out an inner and outer circle using a piece of string and a nail. The radii of these are 45cm and 65cm respectively.
- Excavate between the two circles to a depth of 10cm. (NB if soil tends to be unstable excavate to 20 cm deep and fill with aggregate 10cm deep, alternatively make a larger deeper ring beam)

Figure 3 Checking the level of the concrete ring beam

- Fill the excavation with a concrete mix of 4:2:1 (aggregate: sand: cement), as shown in Figure 3, and cover with damp grass/leaves and leave for two days.
- Start building the brick jar on top of the concrete ring using a mortar mix of 5:1 (sand: cement).
- After 0.55m height of brickwork fill the centre of the jar with rubble/aggregate. (this can be neglected if a sump for the jerrican/bucket is made instead)
- Insert a ∅3/4" GI pipe through one of the brickwork joints for the tap outlet, on which is fitted a ∅3/4" elbow at one end and a ∅3/4" to ∅1/2" reducer and a ∅1/2"tap at the other. Alternatively use a PVC pipe which can be threaded by the method described in Appendix 1

- Render the base of the jar with a mortar mix of 4: 1 (sand:cement) to 2.5cm deep, ensuring that the top of the elbow is level with the base.
- Continue building the brickwork up to a height 1.55m above the ground, periodically checking that the diameter is constant all the way up. On the last course of bricks fit in a short length of ∅1 ½” PVC pipe (or nearest size available) for the over flow as shown in Figure 4 and cover the end with a piece of mosquito mesh.
- Once the mortar between the brickwork has set render the inside of the jar with mortar mix of 4:1 to a depth of 1cm. Scratch this coat with a nail for a good mechanical key whilst it is still damp and leave for one day.

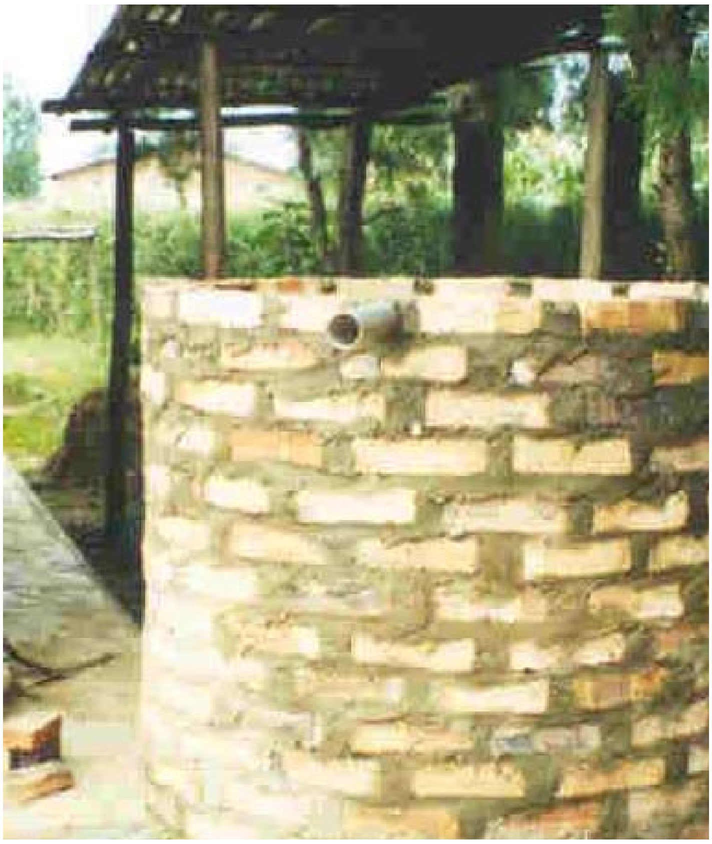


Figure 4 Full height of the brickwork clearly showing the position of the overflow

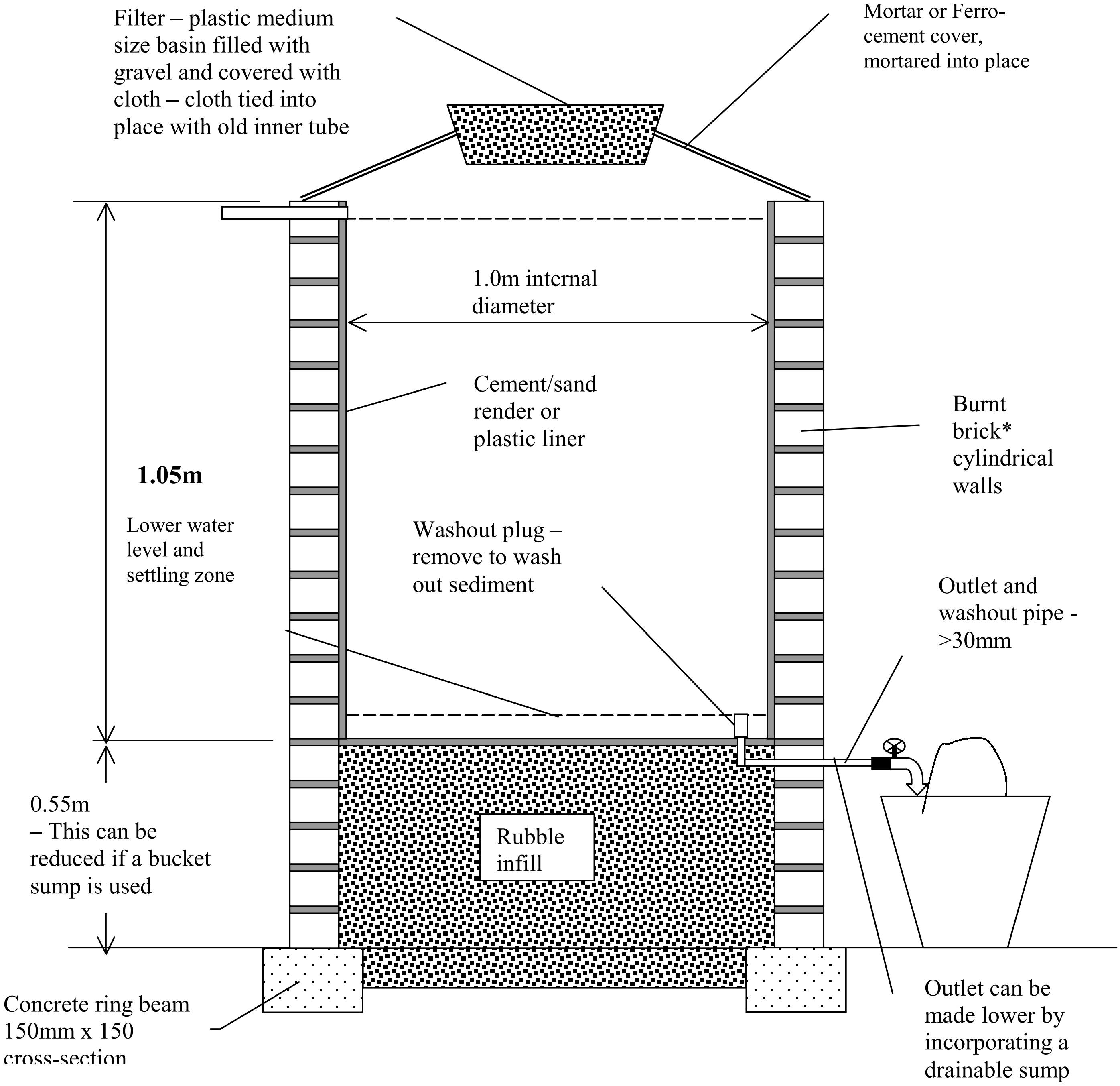
- Apply a mortar mix with a waterproof additive on to the walls and base (quantity as per manufacturers instructions) to a depth of 1cm. Cover and leave this for about twelve hours then pour in about 20 litres of water as this will help prevent cracking and shrinking (make sure the tap is turned off!), this should now be left for a further 24 hours.
- On the top course of bricks apply a layer of mortar and carefully lift on to this the shell cover, make a smooth radius around the base of the cover and the jar wall.
- Make a series of about twenty holes in the bottom of the basin using a hot nail or a drill (Ø6mm) for the rain to enter the jar. Fill the basin with aggregate (about 20mm) to about one third full.

- Cut out a square section of clean cloth that will cover the basin and have sufficient to overhang down the sides, then tie around the sides of the basin with string or rubber inner tube strip to hold the cloth in place.
- Lower the basin in to the jar.
- Fix the appropriate gutter to the roof and place the downpipe directly on to the cloth.

8. Care of the jar on completion

Once the jar is finished fill with 125mm of water each day so that the structure is gradually loaded rather than all at once.
It may be noticed that the jar will leak slightly somewhere around the sides, it is best if this is left for at least a week, as quite often the jar will eventually seal itself if the hole is small (this has come from masons personal experience in Uganda). However, if it shows no sign of sealing or the hole is quite visible, empty the contents and make the repair with a nil mix (i.e. purely a cement and water mix) from the inside of the jar.
It is important that there are no gaps around the filter and cover, or around the filter where light can enter as this will not only encourage the growth of algae but may be an entry point for mosquitoes both of which should be avoided.

9. Brick jar diagram



NB: for this tank the following brick details were:
* Burnt brick size – 225 x 100 x 75mm
1. Number of courses – 21*
2. Number of bricks per course - 14

Total number of bricks - 294

Diagram of Brick Jar construction

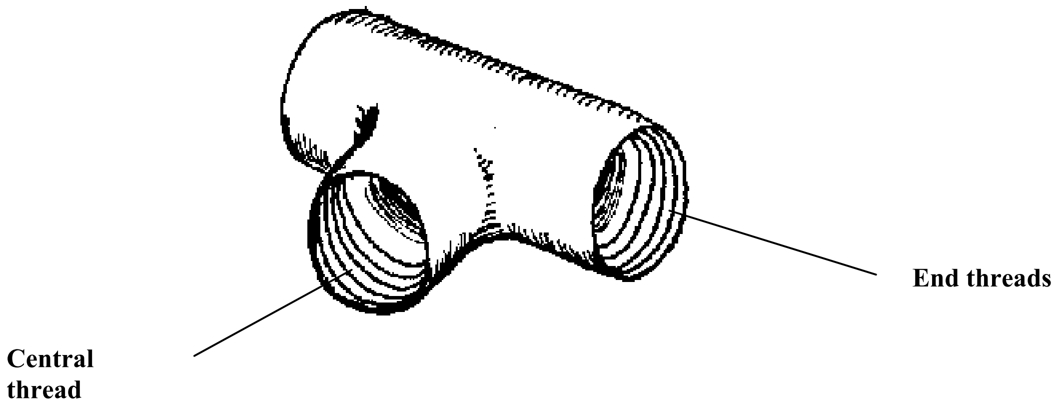
Appendix 1: Low cost threading of PVC pipes using a standard galvanised iron (GI) pipe fitting

There are many occasions were threads are required on PVC pipes so that other fittings can added to the pipe. This often involves the use of expensive threading equipment, which is not always available when needed and the charge for this service can become expensive when it is done repeatedly.
The method described here was tried out in Uganda after finding the problems mentioned above and was found to be a useful and successful solution that was very low cost. Though it requires some tools, a little bit of skill and some patience, once it has been made it will last for many threading operations and re-sharpening is simple to do.
The following example is for a ∅½” PVC pipe but the same procedure is carried out for other sizes:

Tools required:

One hacksaw blade (preferably with 24 teeth per inch)
A small (6”long) triangular file (the width across the faces should preferably be no more than about 3/16”or 4mm)
Pipe grips or a vice.
10” rough flat file.

The reliability of the threads for higher pressure applications has not been checked and care will be needed when trying this out.



10. Take a normal GI ∅½” Tee fitting as shown in Figure 1 and make three equally spaced saw cuts with the hacksaw blade in the central part of the Tee to just beyond the roots of the thread as shown in Figure 2.

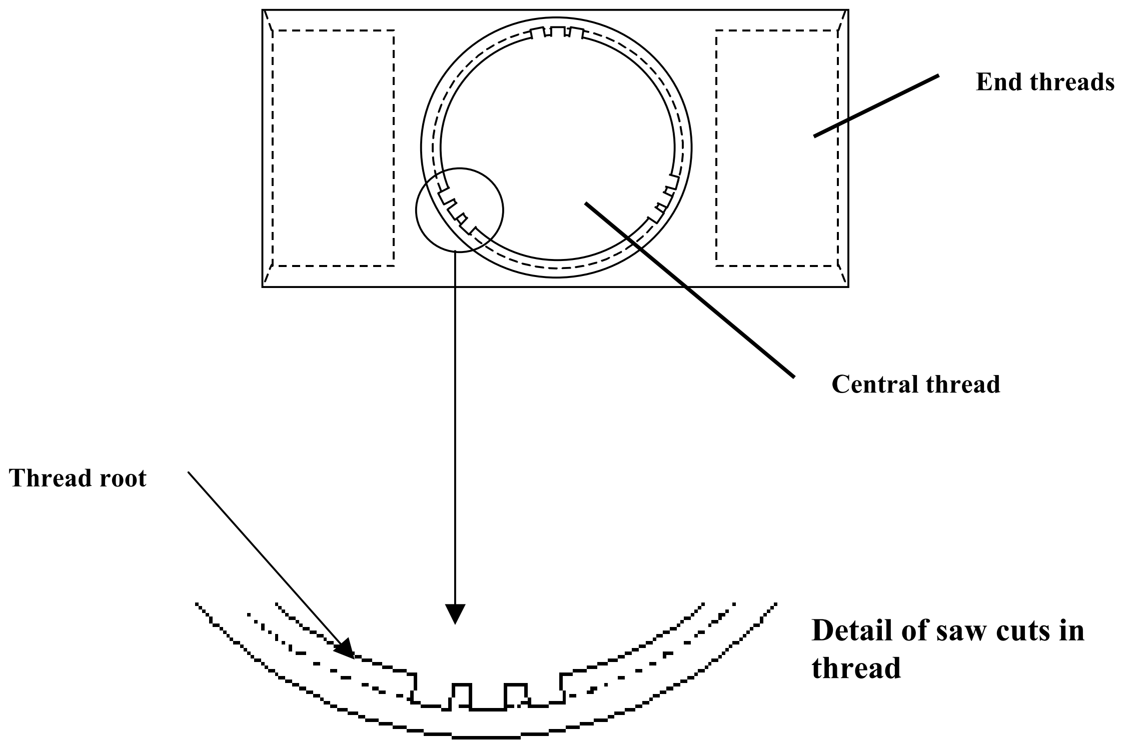


Figure 2 The GI Tee with the saw cuts equally spaced round the central thread

11. Make additional saw cuts as close as is practically possible to the first thread so that it is slightly wider than one of the faces of the triangular file, this is to ease the burden of filing.
12. Proceed to file each of the saw cuts so that the roots of the thread can no longer be seen.
13. File the left-hand side of the slot, as this will be the cutting edge, so that the profile is the same angle as the file i.e. 60° as shown in Figure 3.

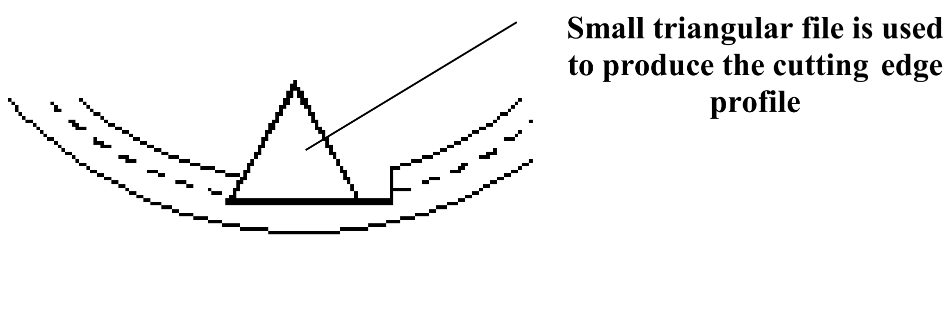


Figure 3 The GI Tee with detail of the cutting edge profile on thread

14. Using a rough file chamfer the end of the pipe to be threaded to the dimensions shown in Figure 4.

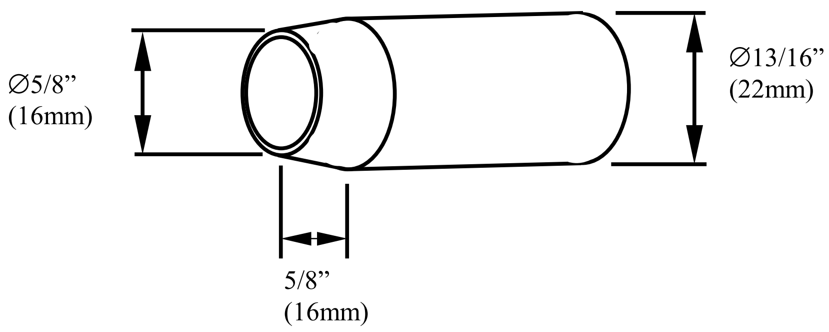


Figure 4 Chamfer dimensions for the ∅1/2” PVC pipe

15. Hold the pipe firmly in an upright position (using a vice or pipe grips) and apply a generous amount of grease or Vaseline to the thread cutter or to the pipe end. This will reduce the friction while thread cutting.
16. Place the thread cutter on top of the pipe and gently start to turn/thread it on to the prepared pipe, making sure that the top of the GI fitting is level as shown in Figure 5.

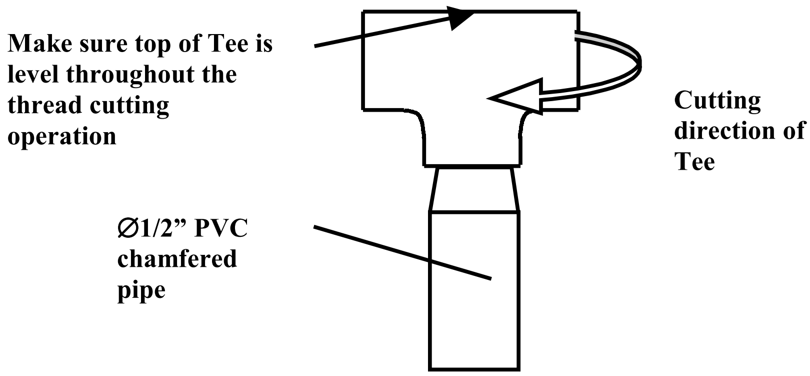


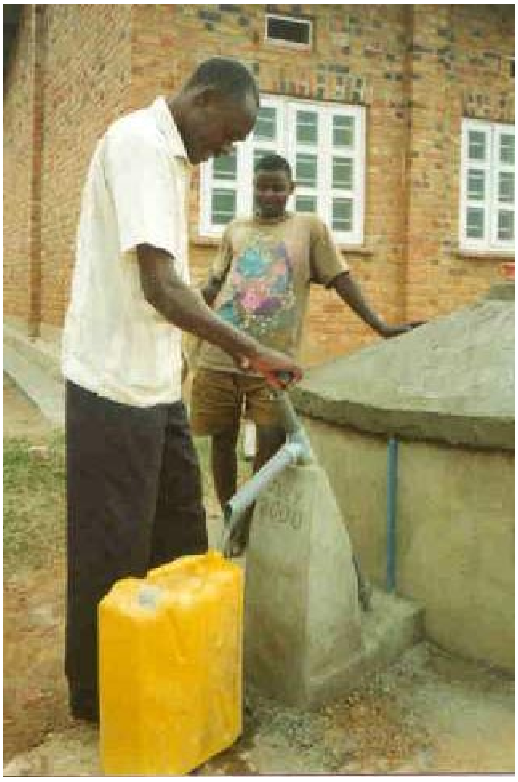
Figure 5 Starting the thread cutting

17. Turn several times (by inserting a screwdriver or steel rod through the Tee) then remove and clean out the thread of any plastic that has built up. For every revolution of the thread cutter turn back again half a revolution, this will break off the material being cut and avoid clogging of the cutting edge space.
18. Repeat the operation until a sufficient length of thread has been cut.

Re-sharpening the cutter is simply done by filing the cutting edge with the small triangular file until the blunt edge has been removed.
Polythene bags cut in to thin strips and wrapped round the thread is a good low cost substitute for PTFE tape.
Please note that it may take several attempts before a satisfactory thread has been made so practice on spare pieces of material until confidence and the quality is built up.

DTU Technical Release Series TR-RWH09

The Manufacture of Direct Action Handpumps for use with Domestic Rainwater Harvesting Tanks



Prepared by Vince Whitehead

Development Technology Unit
School of Engineering, University of Warwick
Coventry, CV4 7AL, UK
Tel: +44 (0)24 76523122
Fax (0) 24 76418922
Email: dtu@eng.warwick.ac.uk
Http://www.eng.warick.ac.uk/DTU/

September 2000

Contents

Introduction

Following the work of building some partially below ground tanks at Kyera Farm, Mbarara, Uganda for the collection of rainwater, there was a need to extract water from the tanks simply and effectively without incurring a large expense or relying on materials and spares being brought from overseas. During July and August 2000 a few designs and ideas were tried which caused sufficient interest from several NGO's to hold a training workshop at Kyera Farm. This manual is a direct result of that interest, it contains sufficient information for someone with access to a set of basic tools and a reasonable amount of practical skill to manufacture the handpumps without too much difficulty. Within this manual there are details of four types of direct action handpumps for use with domestic roof water harvesting (DRWH) tanks.

The main objective in the design of these hand pumps was to produce low cost handpumps, which can be manufactured and maintained with locally available materials and locally available skills.

It must be mentioned that these pumps are not intended for community use and are for household use only as community pumps require a much more robust design. All these pumps have been used at depths between 2.5 to 3m but may be used at beyond this subject to practical limitations. There may be situations where these pumps may be used for purposes other than with DRWH tanks, for example in irrigation, you are encouraged to apply them wherever there may be a use for them.

The following designs and parts list may be adapted or exchanged to suit the availability of materials or personal preferences, e.g. choice of foot valve or exchanging the handle from one pump to another.
I apologise for using both imperial and metric units in this manual, many items purchased in Uganda were found to use both units, please use the nearest equivalents wherever possible.

I would be grateful for any feed back you may have on any aspect of these pumps whether this is criticism or possible improvements.

Please forward any correspondence to:-

Development Technology Unit
School of Engineering
University of Warwick
Coventry, CV4 7AL, England.
Email:- dtu@eng.warwick.ac.uk

2 List of tools required for handpump manufacture

This list is a good guide of what tools are required for the handpumps but it may be found that other tools may make manufacture easier or more cost effective. It may depend on what tools are available.

1 Hacksaw and blade

2 Hammer

3 Pliers

4 Pipe wrench

5 Hand drill

6 Set of drills (2mm, 3mm, 4mm, 5mm, 6mm, 7mm, 8mm,)

7 Screwdriver

8 Set of files (150mm triangular, 250mm half-round, 250mm rough)

9 Scissors

10 Tape measure

11 Stove (or some adequate heat source)

12 Sandpaper

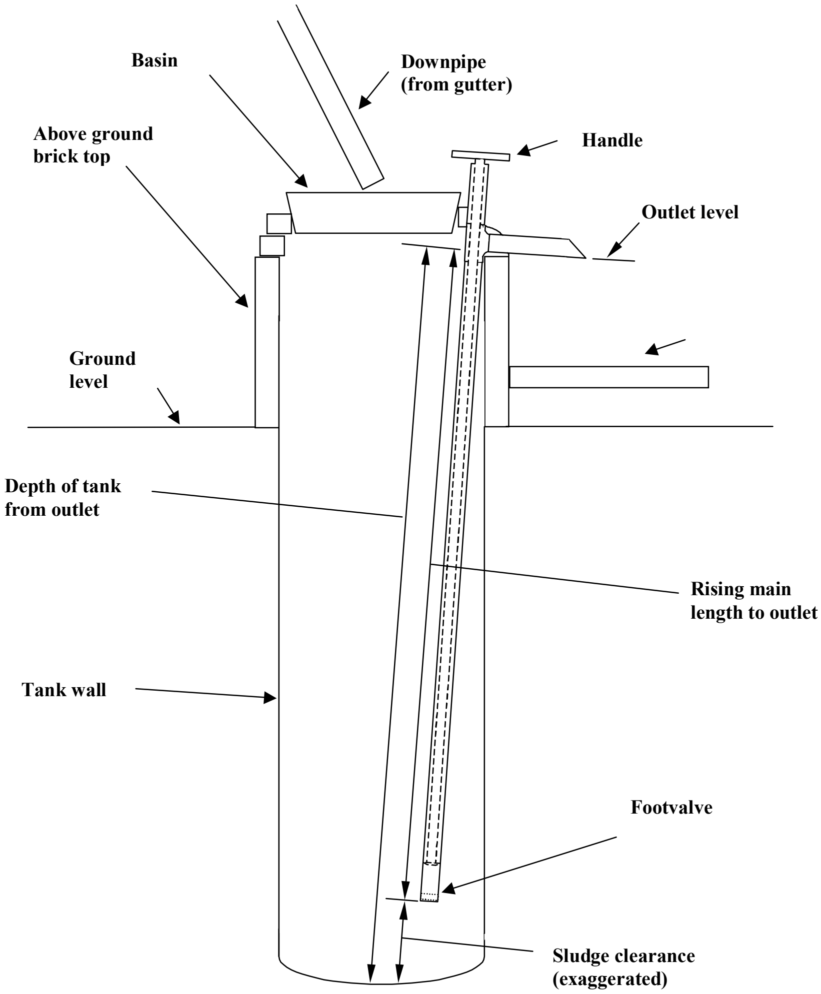


Figure 1 Principle components of domestic rainwater harvesting tank

A typical domestic rainwater harvest (DRWH) tank is shown in Figure 1 illustrating some of the major components and showing the handpump in position. It is important to note with the handpump that the footvalve must not come in to contact with the base of the tank and should ideally have sufficient clearance for any build up of sludge.

At the top of the tank the handpump protrudes through a small gap in the brickwork and is held rigidly with a small amount of sand and cement. This is placed around the Tee to restrain any movement from forces generated during operation. On larger tanks the handpump may protrude through a small hole in the cover, a mound of mortar is built up around the Tee to securely hold this in position. When maintenance is required the mortar can be carefully removed and the pump withdrawn.

Most containers used for collecting water in West Africa are plastic jerricans, so the distance from the handpump outlet to the stand should be about 25 to 50mm greater than the height of a 20 litre jerrican.

3 The DTU Handpump

The DTU Handpump is simple lift pump and uses a leather stirrup pump piston, which is available from most cycle shops. The principle of operation is as follows: As the handle is lifted the water above the leather washer is lifted with it, during this stage the footvalve is opened and the water fills the rising main below the leather piston. On the downstroke the footvalve is closed and the water in the lower section by passes the leather washer to the upper section, repeat operations transfers water to the outlet. It will be noticed during operation that water will pour from the outlet on both strokes, this is because the push rod displaces water within the rising main.

3.1 Manufacturing procedure.

- Measure depth of tank (from outlet level to bottom of tank) and subtract some small amount to ensure that the footvalve is not touching bottom of tank. This is to stop sediment from being drawn up in to the handpump (This may be around 250 - 300mm)

- Select a straight piece of Ø1 ½" PVC pipe with a smooth internal bore (This will cause less wear on the leather piston) and cut to required lengths as indicated in the following assembly drawing. Also cut the top tube and the outlet.

- Make sure the internal surface of the tee is clear of any obstructions, as this will interfere with inserting/removing the leather piston, file any obstructions until the leather piston passes through the tee.

- Make up the DTU or low cost valve (as shown in Appendix 1) and fix to the bottom of rising main.

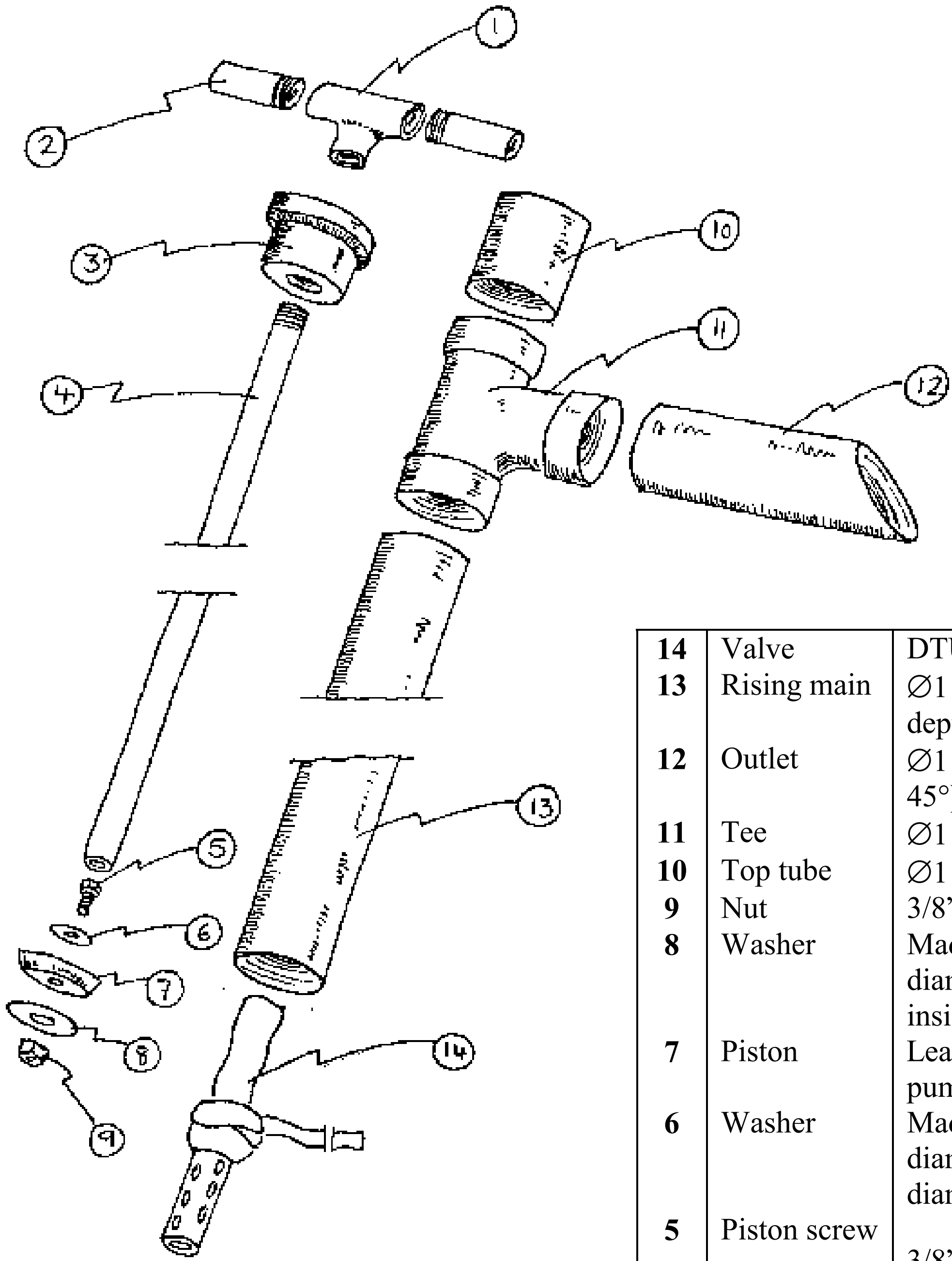
- Push top tube and tee on to rising main and measure or calculate the total distance from the top of the top tube to the top of the footvalve. From this subtract approximately 50mm and cut the pull rod to this measurement, this will ensure that the piston will not hit the footvalve at the bottom of the pull rod stroke.

- Thread the top end (details on threading PVC pipe is shown in Appendix 2) of the and pull rod and fit a Ø½ " PVC/GI Tee, also fit the two threaded handles and slide on the wooden support bush.

- Securely fix the 3/8" BSW (or an M8) screw in to bottom of pull rod. This can be done by heating the end of the pipe and pushing the PVC material in to the thread to stop the head of the screw from coming out of the pipe. A series of four saw cuts can be cut in to the end of the pipe to a depth equal to the saw blade (about 12mm) this will help when pushing the hot PVC material on top the thread.

- Place on the screw the washer, leather, second washer and the 3/8" BSW Nut on to the 3/8" BSW screw alternatively use a M8 screw and nut.
- Grease the leather washer (make sure that it will not taint or contaminate the water.
- Fit on the outlet tube on to the tee.
- Insert the pull rod into the rising main.
- Test operation of the handpump.
- If the handpump operates satisfactorily use PVC cement on the three joints of the PVC tee.

3.2 DTU Handpump assembly drawing



14	Valve	DTU type (see Appendix1)
13	Rising main	∅1 1/2” PVC (length to suit depth of tank)
12	Outlet	∅1 1/2” x 8” (200mm) (end at 45°)
11	Tee	∅1 1/2” PVC
10	Top tube	∅1 1/2” x 8” (200mm)
9	Nut	3/8” BSW or M8
8	Washer	Made from PVC pipe, outside diameter = 1 1/4” (30mm), inside diameter = 3/8” (8mm)
7	Piston	Leather washer from stirrup pump
6	Washer	Made from PVC pipe, outside diameter = 1” (25mm), inside diameter = 3/8” (8mm)
5	Piston screw	3/8” BSW x 3/4” (M8 x 20mm)
4	Pull rod	1/2” PVC pipe (length to suit rising main)
3	Pull rod bush	To suit pipe (see Appendix 3)
2	Handles	1/2” PVC pipe x 4” (100mm) (2 pieces)
1	Tee	1/2” PVC or GI

3.3 Cost example for 2.5m long DTU Hand pump

PART No.	NAME	UNIT	SIZE/ LENGTH	COST PER MATERIAL LENGTH (UGS)	COST/PUMP (UGS)	COST/PUMP US Dollar ($1= UGS1775)[1]
1	Tee	Inch	Ø½" GI (or PVC)	500	500	0.28
2	Handles	Inch	Ø½"PVC x 4" (100mm) 2 pieces	7,500 per 20ft	250	0.14
3	Pull Rod Bush	No	1	260	260	0.14
4	Pull Rod	Inch	Ø½"PVC x 80" (2000mm)	7,500 per 20ft	2,500	1.39
5	Screw	No	3/8" BSW X 1" or M8 x 25mm	500	500	0.28
6	Washer	No	3/8" (8mm)	100	100	0.06
7	Piston	No	Ø1 ½" leather	500	500	0.28
8	Washer	No	3/8" (8mm)	100	100	0.06
9	Nut	Inch	3/8" (or M8)	-	-	0.00
10	Top tube	Inch	Ø1 ½" PVC x 8" (200mm)	12,500 per 20ft	420	0.23
11	Tee	Inch	Ø1 ½ " PVC	2,500	2,500	1.39
12	Outlet	Inch	Ø1 ½" PVC x 8" (200mm)	12,500 per 20ft	420	0.23
13	Rising Main	Inch	Ø1 ½" PVC x 86" (2150mm)	12,500 per 20ft	4,480	2.50
14	Foot Valve (DTU)	No	1	675	675	0.38
					UGS13,205	**$7.36**

Material cost (UGS) = 13,205
(Labour Cost/day = 2,500)
Labour cost for 4 hrs = 1,250
Total cost = 14,455 ($8.14)

DTU VALVE COST:

¾" PVC pipe x 8" @ 10,000/20ft = 333
Inner tube x 4" = 90
2ft of inner tube strip = 250
Total cost = 673 ($0.38)

[1] Source: http://finance.yahoo.com/m5?a=1&s=USD&t=UGX (Sept 2000)

4 The Tamana Hand Pump

This slightly modified version of the Tamana handpump, which is used as suction pump, relies on a seal from the piston/valve and the bore of the PVC cylinder.

During operation and on the upstroke, the piston/valves lays flat on the PVC supports, this creates a lower pressure below the piston/valves which draws water in to the cylinder through the footvalve. As the push rod is depressed the piston/valves are slightly raised from their piston supports and water flows through the holes of these into the cylinder above the piston/valves. On each stroke water is discharged through the outlet, this is because on the downstroke the volume of the pull rod displaces water as with the previous handpump.

Labyrinth seals (a series of seals) can improve the time between seal replacements, this version uses only two as a demonstration but more could be added. A suitable length of ½ " PVC pipe is connected to the reducer which leads in to the DRWH tank and a floating valve is used for the intake (see Appendix 4 for details of the floating valve). The PVC pipe can either be threaded (as described in Appendix 2) and fitted with elbows to reach down in to the tank or it can be bent by heat application. The latter can be done by putting a wood stopper at one end of the pipe and pouring sufficient sand inside the pipe a few inches beyond where the bend is to be. Heat gently and evenly around the bend area and bend slowly until the desired angle is achieved. Cold water can be poured on to the pipe to set it quickly.

4.1 Manufacturing procedure

- Select a piece of Ø1 ½" PVC pipe (with a swaged end) with as smooth as bore as is possible and cut to 450mm long. Chamfer the swaged end and thread (using the method described in Appendix 2).

- Cut the top tube and outlet to length, to the dimensions shown in the following table).

- Cut three lengths of Ø3/4" PVC pipe and split each one lengthways. Their faces must be square with the bore.

- For the PVC supports: mark out two concentric diameters on a flattened piece of PVC pipe, the larger diameter is 1/8" smaller than bore of Ø1 ½" PVC pipe and the smaller diameter is the same as the outside diameter of the pull rod. Cut out the central hole out first, as this is easier. Then mark out 8 equally spaced holes and drill them 5mm diameter. Finally cut out the outside diameter and file smooth. N.B. The piston supports are made from PVC pipe by splitting, heating and flattening a suitable piece of pipe.

- Cut out two rubber piston/valves using the piston/valve cutter shown in Appendix 5.

- Assemble together as shown in following drawing, making sure that top and bottom piston stops are pushed up towards each other trapping the piston/valves in place, screw top & bottom piston stops in this position.

- Cut a 250mm piece of Ø3/4" PVC pipe for the handle, make a hole in the centre for the pull rod to pass through.

- Place a wood support bush on to pullrod and fix handle in place.

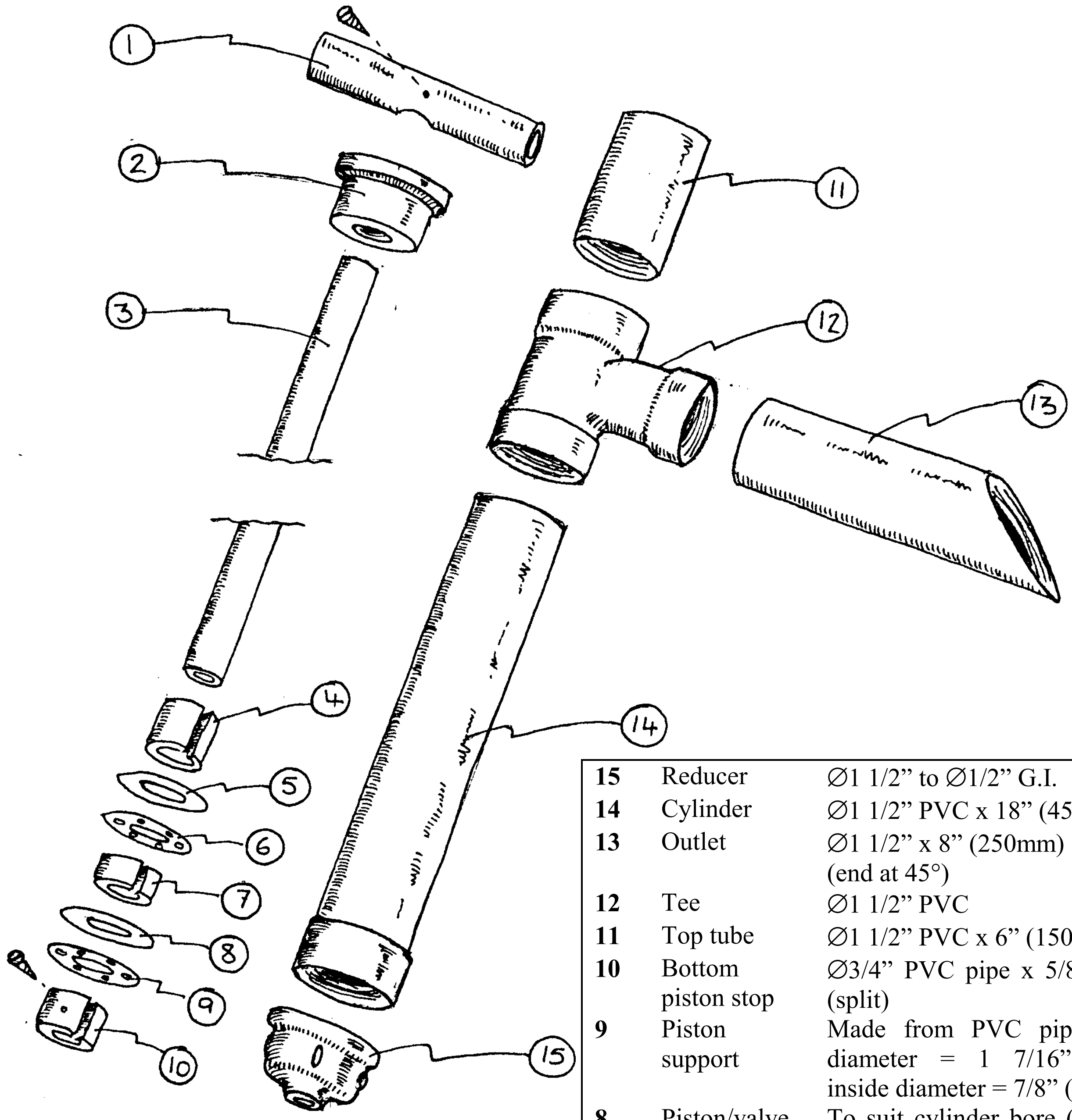
- Cut a small disc of PVC to 22mm diameter and cement this to the top of Ø½" PVC pullrod.

- Check that the piston assembly passes through the Ø1 ½" Tee, file any obstruction till this is achieved.

- Assemble and test.

- If pump works satisfactorily, cement the three joints around the Ø1½" Tee ensuring that pipes are in line with each other.

4.2 Tamana handpump assembly drawing



15	Reducer	∅1 1/2” to ∅1/2” G.I.
14	Cylinder	∅1 1/2” PVC x 18” (450mm)
13	Outlet	∅1 1/2” x 8” (250mm) (end at 45°)
12	Tee	∅1 1/2” PVC
11	Top tube	∅1 1/2” PVC x 6” (150mm)
10	Bottom piston stop	∅3/4” PVC pipe x 5/8” (16mm) (split)
9	Piston support	Made from PVC pipe, outside diameter = 1 7/16” (36mm), inside diameter = 7/8” (22mm)
8	Piston/valve	To suit cylinder bore (use piston cutter shown in Appendix 5)
7	Centre piston stop	∅3/4” PVC pipe x 5/8” (16mm) (split)
6	Piston support	Made from PVC pipe, outside diameter = 1 7/16” (36mm), inside diameter = 7/8” (22mm)
5	Piston/valve	To suit cylinder bore (use piston cutter as shown in Appendix 5)
4	Top piston stop	∅3/4” PVC pipe x 5/8” (16mm) (split)
3	Pull rod	∅1/2” PVC x 25” (625mm)
2	Pull rod bush	To suit pipe (see Appendix3)
1	Handle	∅3/4” PVC x 8 “ (250mm)

4.3 Cost example for 450mm long Tamana handpump

PART No.	NAME	UNIT	SIZE/ LENGTH	COST PER MATERIAL LENGTH (UGS)	COST/ PUMP (UGS)	COST/PUMP US Dollar ($1= UGS1775)[2]
1	Handle	Inch	Ø½"PVC x 8" (250mm)	10,000 per 20ft	335	0.19
2	Pull rod bush	No.	1	2000	260	0.14
3	Pull rod	Inch	**Ø½"PVC x 25" (625mm)**	**7500per 20ft.**	780	0.43
4	Top piston stop	Inch	Ø3/4"PVC x 5/8" (16mm)	10,000 per 20ft.	26	0.01
5	Piston valve		To suit cylinder		50	0.03
6	Piston support		1 7/16" (36mm) outside diameter, 7/8" (22mm) inside diameter	12500 per 20ft.	26	0.01
7	Centre piston stop	Inch	Ø3/4"PVC x 5/8" (16mm)	10,000 per 20ft	26	0.01
8	Piston/valve		To suit cylinder		50	0.03
9	Piston support	Metric	1 7/16" (36mm) outside diameter, 7/8" (22mm) inside diameter.	12500 per 20ft.	26	0.01
10	Bottom piston stop	Inch	Ø3/4"PVC x 5/8" (16mm)	10,000 per 20ft	26	0.01
11	Top tube	Inch	Ø1 ½" PVC x 8" (250mm)	12500 per 20ft.	420	0.23
12	Tee	Inch	Ø1 ½"	2500	2500	1.39
13	Outlet	Inch	Ø1 ½" PVC x 8" (250mm)	12500 per 20ft.	420	0.23
14	Cylinder	Inch	Ø1 ½" PVC x 18" (450mm)	12500 per 20ft.	940	0.52
15	Reducer	Inch	Ø1 ½" – Ø1/2"	2000	2000	1.11
					7885	**4.39**

Material Cost (UGS)	= **7,885**	**($4.39)**
(Labour Cost/day	= 2,500)	
Labour cost for 4hrs	= 1,250	
Total cost (UGS)	**= 9,135**	**($5.09)**

N.B. there will be additional cost to this as the Ø ½" PVC pipe to the tank and the floating valve have not been included.

[2] Source: http://finance.yahoo.com/m5?a=1&s=USD&t=UGX (Sept 2000)

5 The “Harold” handpump

This is another lift pump, though it does not rely on having a seal or a flexible membrane within the rising main. The piston as such is a moulded plastic cup which is slightly smaller than the bore of the rising main, this is shaped in such a way that it has greater resistance on the up stroke and water is lifted by the cup. A small amount of water will leak past the gap around the cup. During the upstroke the footvalve opens allowing water into the rising main. On the down stroke the foot valve closes, and the water within the rising main flows around and above the cup. Repeated operation lifts water to the outlet and very little is displaced on the return stroke because of the small volume of the pull rod.

Despite the gap around the moulded cup the pump discharges a surprising amount of water per stroke.

5.1 Manufacturing procedure

- Measure the depth of the tank from the outlet to the bottom of the tank and subtract some small amount to ensure that the footvalve is not touching bottom of tank. (This may be around 200 to 250mm)

- Select a straight section of Ø1 ½" PVC pipe and cut to the required length.

- Cut the top tube and outlet to the right length as shown in the following table.

- Push these sections together with the Tee, and fit a footvalve to the bottom of the rising main (see Appendix 1 for footvalve assembly)

- Measure the distance from the top of the top tube to the top of the foot valve, subtract 1" from this and cut a section of 3/8” or 8mm steel rod to this length.

- Slit one and with a hacksaw to a depth of 16mm and the other end to a depth of 30mm (if a hand vice is unavailable see Appendix 6).

- Cut an 8" long piece of Ø½” PVC pipe for the handle and drill a 3/8” or 8mm diameter hole at the midpoint. Insert the handle on the pull rod, so the handle is level with the bottom of the 30mm slit.

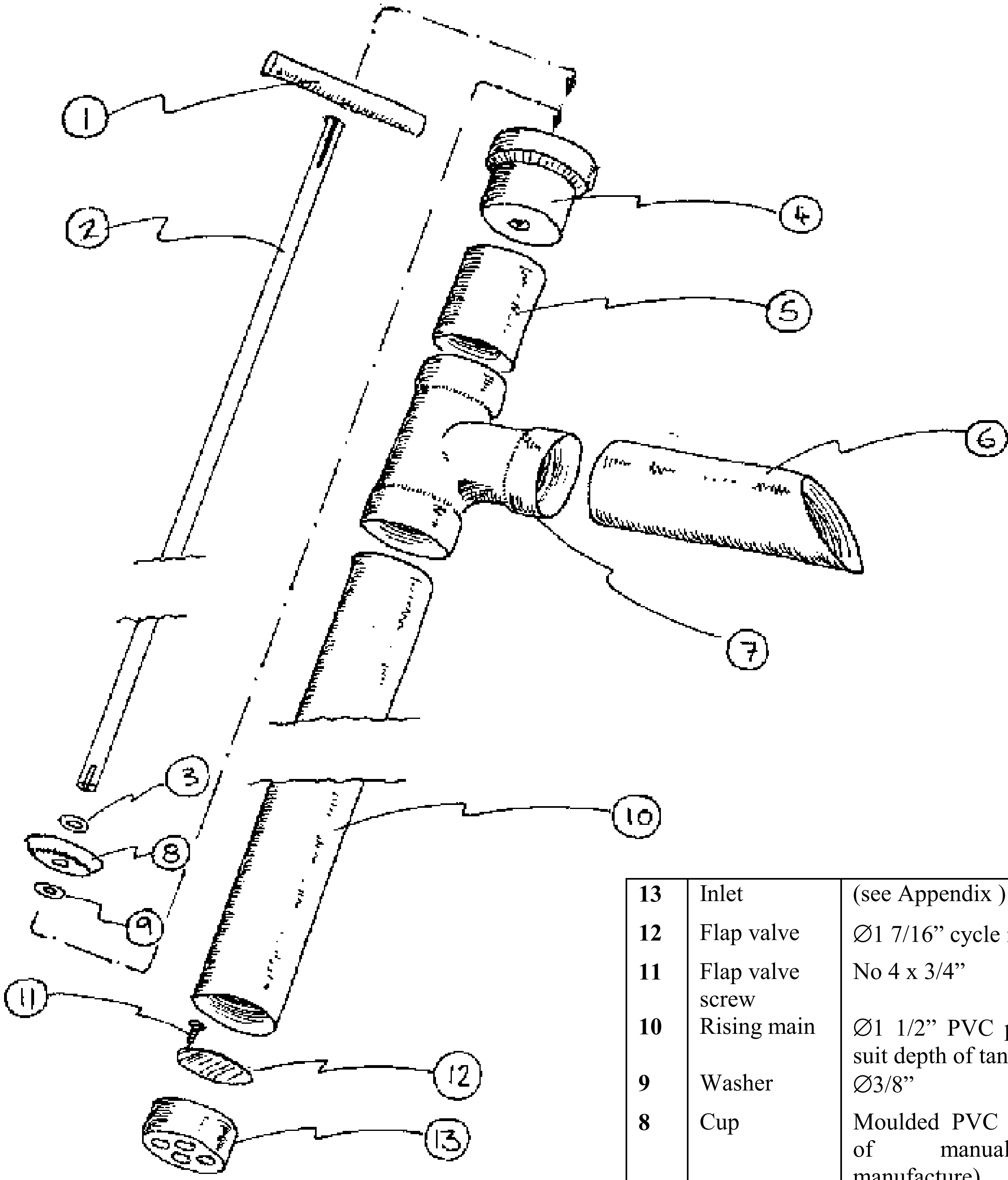
- Prize open the two halves of the rod with a screwdriver and gradually push and hammer back the steel around the handle until the handle is firmly held in place.

- Slide on the wood support bush towards the handle.

- At the other end insert a piece of copper wire through the slit and wind round several times, push the washer up to the wire, then fit on the moulded cup (see Appendix 7) and bottom washer. Wrap the remaining copper wire around the rod above the washer to retain it. Push a screwdriver blade into slot and gently prize open the two halves so that the assembly is held firm.

- Check that there is no obstruction in the 1 ½” Tee, making sure that the pull rod assembly will pass through the Tee, file clear if needed.
- Push the assembled pull rod down into the rising main and test operation.
- If the pump works okay cement the three joints around the Tee making sure the pipes are all in line with each other

5.2 Harold handpump assembly drawing.



13	Inlet	(see Appendix)
12	Flap valve	∅1 7/16" cycle inner tube
11	Flap valve screw	No 4 x 3/4"
10	Rising main	∅1 1/2" PVC pipe x (to suit depth of tank)
9	Washer	∅3/8"
8	Cup	Moulded PVC (see back of manual for manufacture)
7	Tee	∅1 1/2" PVC
6	Outlet	∅1 1/2" PVC pipe x 8" (end cut at 45°)
5	Top tube	∅1 1/2" PVC pipe x 8"
4	Pull rod bush	To suit pipe (see Appendix 3)
3	Washer	∅3/8"
2	Pull rod	∅3/8" steel x (to suit depth of rising main)
1	Handle	∅1/2" PVC x 8"

5.3 Cost example for 2.5m long Harold handpump

PART No.	NAME	UNIT	SIZE/ LENGTH	COST PER MATERIAL LENGTH (UGS)	COST/ PUMP (UGS)	COST/PUMP US Dollar ($1= UGS1775)[3]
1	Handle	Inch	Ø½" PVC pipe x 8" (200mm)	7500 per 20 ft	250	0.14
2	Pull rod	Inch	Ø3/8" or 8mm steel x 95" (2375mm)	2000 per 20 ft	792	0.44
3	Washer	No.	3/8" or 8mm	100	100	0.06
4	Pull rod bush	No.	1	260	260	0.14
5	Top tube	Inch	Ø1 ½" PVC pipe x 8'' (200mm)	12500 per 20 ft	420	0.23
6	Outlet	Inch	Ø1 ½" PVC pipe x 8'' (200mm)	12500 per 20 ft	420	0.23
7	Tee	No.	Ø1 ½" PVC	2500	2500	1.39
8	Molded cup	No.	1	12500 per 20 ft	26	0.01
9	Washer	No.	3/8" or 8mm	100	100	0.06
10	Rising main	Inch	Ø1 ½" PVC pipe x 89" (2225mm)	12500 per 20 ft	4635	2.58
11	Wood screw	No.	4 x ¾" (20mm)	1000/box	20	0.01
12	Valve	Inch	1 7/16" diameter or 38mm	20	20	0.01
13	Inlet	No.	1	260	260	0.14
					9,830	**5.46**

Material Cost (UGS) = 9,830 ($5.46)
(Labour Cost/day = 2,500)
Labour cost for 4hrs labour = 1,250

Total cost (UGS) = 11,503 ($6.16)

[3] Source: http://finance.yahoo.com/m5?a=1&s=USD&t=UGX (Sept 2000)

6 The Enhanced inertia handpump

The Enhanced inertia Pump is a lift pump, which again does not rely on a seal within the rising main but uses a central tube to lift the water, which overflows in to the rising main. On the upstroke the central foot valve is closed and the main footvalve is opened letting water in to the rising main. As the handle is depressed the central tube footvalve is opened letting water in to the central tube, meanwhile the main footvalve is closed. Repeat operations gradually brings water up the central tube, this then flows through the holes in the central tube in to the rising main and is eventually discharged at the outlet. A good commercial example is manufactured by made by the "New Zealand Pump Company"
This handpump seems to operate best when short strokes are used. The flow is steady on both strokes of operation again because of the high displacement from the central tube full of water on the downstroke.

6.1 Manufacturing procedure

- Measure the depth of the tank from the outlet to the bottom of the tank and subtract some small amount to ensure that the footvalve is not touching bottom of tank. (This may be around 8” - 10”)

- Select a straight piece of Ø1 ½" PVC pipe and cut to the required length.

- Cut to top tube and outlet from Ø1 ½" PVC pipe (see following table for dimensions)

- Make up the low cost valve or DTU valve and fit to bottom of Ø1 ½" PVC pipe. Make sure that it is a good tight fit, though once it is wet it will expand.
- Push together top tube, Tee and outlet and measure the distance to top of footvalve, from this subtract approx. 2" and cut a piece of Ø1 ¼" PVC pipe to this length. Use the swaged end for the top of the central tube as this seals the two tubes when the handpump is not being used.

- Cut a series of holes approximately 8mm diameter, which starts about 250mm below the top of central tube to around 200mm in length. The number of holes is not too important so long as there are sufficient holes for water to escape through and not too many that it weakens the tube.

- Cut a hole in top of central tube 7/8” or 22mm diameter for the handle.

- Cut the handle from a piece of Ø½" PVC x 200mm long and PVC cement it in to the hole in the central tube.

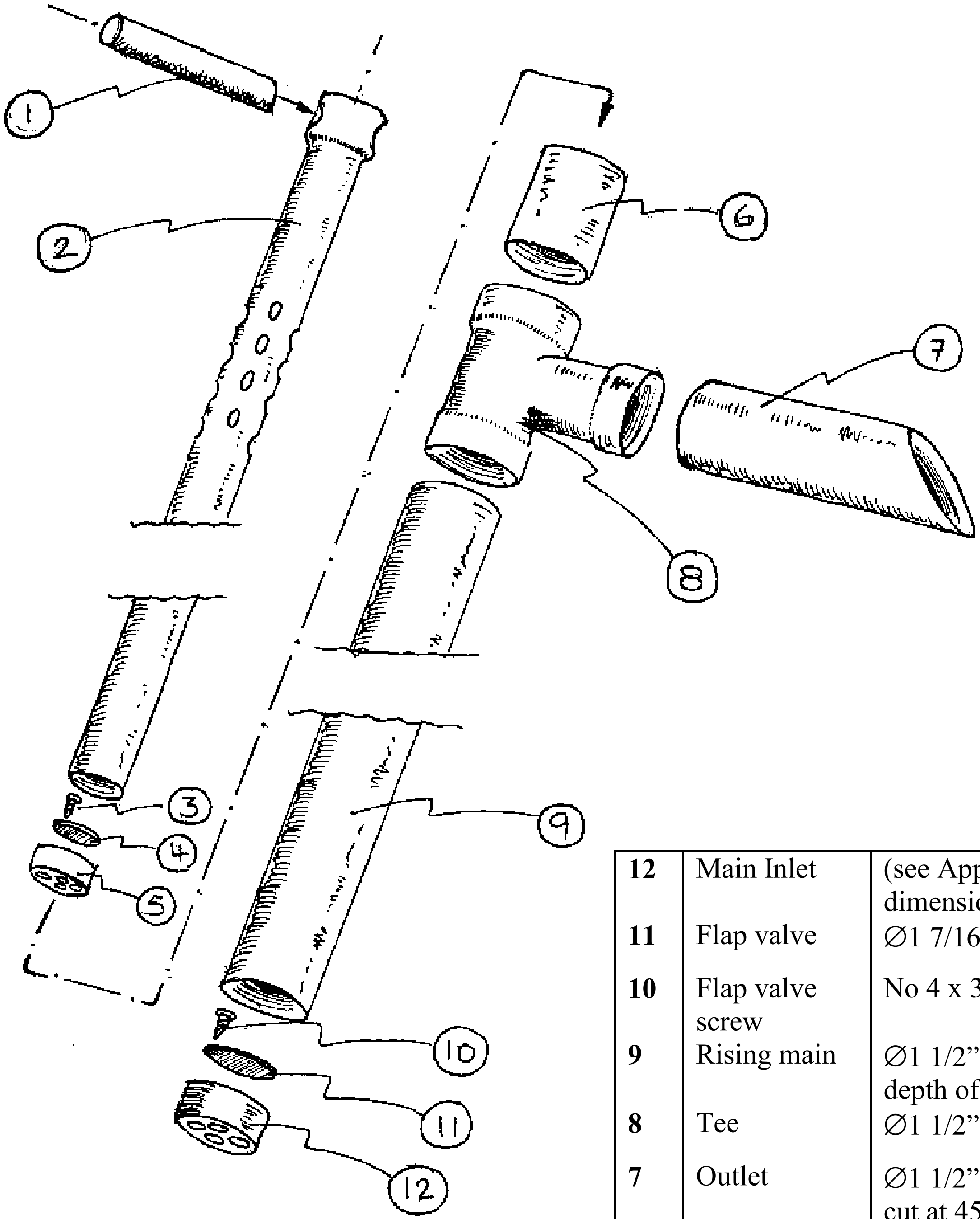
- Cut a PVC disc to fit in top of central tube and cement in place.

- Make up a foot valve and fit to the bottom of central tube. Note that the wood inlet needs to be a good tight fit inside the tube.

- Fit together and test.

- If test is okay, cement the three joints of the Tee ensuring that pipes and Tee are all in line with each other.

6.2 Enhanced inertia handpump assembly drawing



12	Main Inlet	(see Appendix 3 for dimensions)
11	Flap valve	∅1 7/16” cycle inner tube
10	Flap valve screw	No 4 x 3/4”
9	Rising main	∅1 1/2” PVC pipe x (to suit depth of tank)
8	Tee	∅1 1/2” PVC
7	Outlet	∅1 1/2” PVC pipe x 8” (end cut at 45°)
6	Top tube	∅1 1/2” PVC pipe x 8”
5	Central inlet	(see Appendix 3)
4	Flap valve	∅1 3/16” cycle inner tube
3	Flap valve screw	No 4 x 3/4”
2	Central tube	∅1 1/4” PVC pipe x (to suit rising main)
1	Handle	∅1/2” PVC x 8”

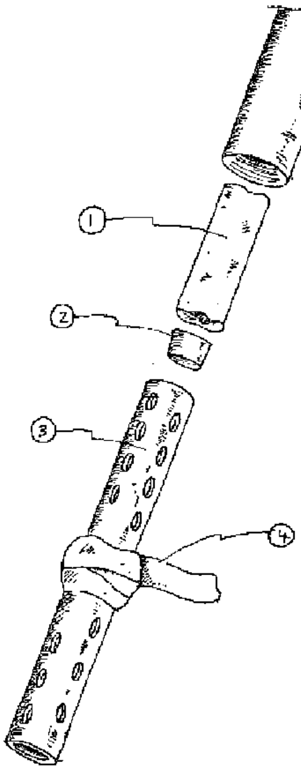
6.3 Cost example for 2.5m long Enhanced inertia handpump

PART No.	NAME	UNIT	SIZE/ LENGTH	COST PER MATERIAL LENGTH (UGS)	COST/ PUMP	COST/PUMP US Dollar ($1= UGS1775)[4]
1	Handle	Inch	Ø½" PVC pipe x 8" (200mm)	7500 per 20ft	250	0.14
2	Central tube	Inch	Ø1 ¼" PVC pipe x 83" (2075mm)	11000 per 20ft	3800	2.12
3	Wood screw	No.	4 x ¾" (20mm)	1000/box	20	0.01
4	Valve	Inch	1 3/16" diameter or 30mm		50	0.03
5	Inlet	No.	1	2000 per 20ft	260	0.14
6	Top tube	Inch	Ø1 ½" PVC pipe x 8'' (200mm)	12500 per 20ft	420	0.23
7	Outlet	Inch	Ø1 ½" PVC pipe x 8'' (200mm)	12500 per 20ft	420	0.23
8	Tee	No.	Ø1 ½" PVC	2500	2500	1.39
9	Rising main	Inch	1 ½" PVC pipe x 86'' (2150mm)	12500 per 20ft	4480	2.50
10	Wood screw	No.	4 x ¾" (20mm)	1000/box	20	0.01
11	Valve	Metric	1 7/16" diameter or 38mm		50	0.03
12	Inlet	No	1	2000	260	0.14
					12530	**6.98**

Material Cost	= 12,530	($6.98)
(Labour Cost/day	= 2,500)	
Labour cost for 4hrs.	= 1,250	
Total cost	**= 13,780**	**($7.68)**

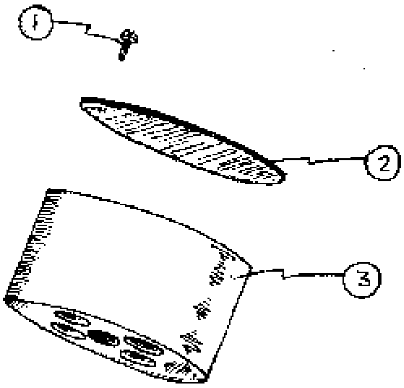
[4] Source: http://finance.yahoo.com/m5?a=1&s=USD&t=UGX (Sept 2000)

Appendix 1Footvalves



The DTU valve is a simple one way valve, which has only four key components. The manufacturing procedure is as follows:

- Cut off a length of Ø ¾ ” PVC pipe 8” long
- Drill a number of Ø 3/8” holes at the top and bottom sections, leaving a gap in the middle of about 2 ½”
- Make up a wood plug and wedge this in at the top of the PVC pipe.
- Slide a 4” length of cycle inner tube over the PVC pipe and start wrapping the rubber strip round the centre of the pipe and trapping the bottom of the inner tube sleeve.
- Make sure that when wrapping the rubber strip round that it is very tight. Wrap sufficiently round so that it is a good tight fit inside the bore of the rising main.
- Continue to wrap the remainder of the rubber strip round the pipe to seal the valve and rising main together.



The low cost valve needs little explanation, as it is self-explanatory. The important point is that the inlet needs to be a good tight fit inside the bore of the rising main as the forces generated during operation could dislodge it.
It may be preferable to make the inlets on a wood lathe but if there is no access to one then careful selection of a suitable piece of wood and some filing will suffice. Dimensional details of the inlets are given in Appendix 3.

1	Screw
2	Rubber inner tube disc
3	Wood inlet

Appendix 2 How to cut a thread in PVC pipe

There are many occasions where threads are required on PVC pipes so that other fittings can be fitted to the pipe. This often involves the use of expensive equipment, which is not always available when needed and the charge for this service can be expensive.

The method described here was tried out in Uganda after finding the above mentioned problems and was found to be a useful and successful solution that was very low cost. Though it requires some tools, a little bit of skill and some patience, once it has been made it will last for many threading operations and re-sharpening is simple to do.
The following example is for a Ø1/2" PVC pipe, but the same procedure is used for other sizes.

Tools required:

One hacksaw blade (preferably 24 teeth per inch)
A small 6" (150mm) long triangular file (the width across the face should preferably be no more than 3/16" or 4mm)
Pipe grips or a vice.
10" (250mm) flat file

The reliability of the threads for higher-pressure applications has not been checked and care will be needed if this is tried.

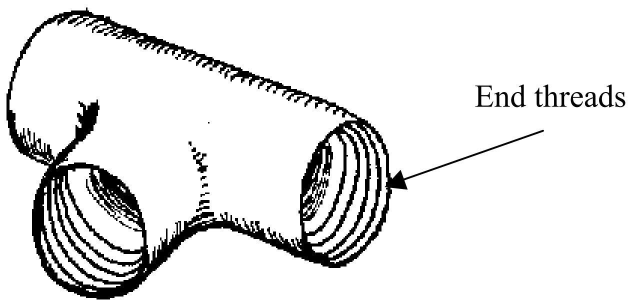


Figure 4 Galvanised Iron tee

Take a standard GI Ø1/2" tee as shown in Figure 4 and make three equally spaced saw cuts

- Take a normal GI Ø½" Tee fitting as shown in Figure 4 and make three equally spaced saw cuts with the hacksaw blade in the central part of the Tee to just beyond the roots of the thread as shown in Figure 5.

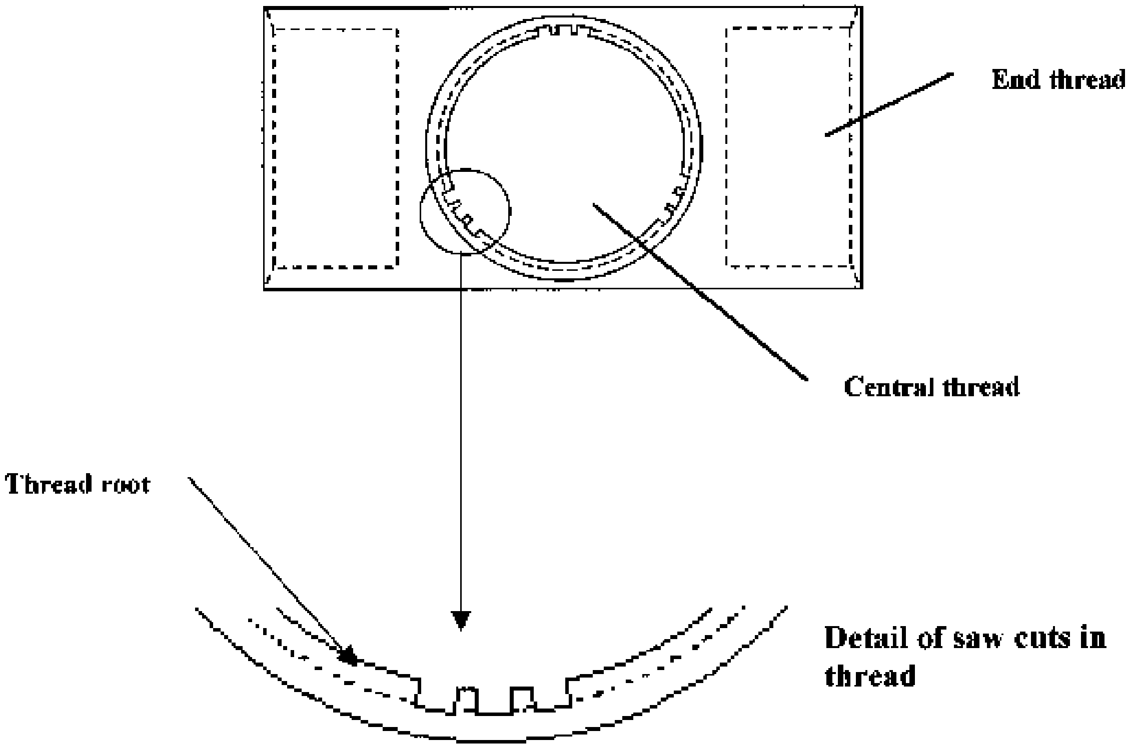


Figure 5 The GI Tee with the saw cuts equally spaced round the central thread

- Make additional saw cuts as close as is practically possible to the first thread so that it is slightly wider than one of the faces of the triangular file, this is to ease the burden of filing.
- Proceed to file each of the saw cuts so that the roots of the thread can no longer be seen.
- File the left-hand side of the slot, as this will be the cutting edge, so that the profile is the same angle as the file i.e. 60° as shown in Figure 6.

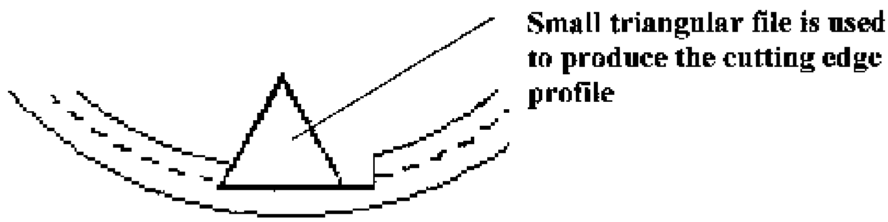


Figure 6 The GI Tee with detail of the cutting edge profile on thread

- Using a rough file chamfer the end of the pipe to be threaded to the dimensions shown in Figure 7.

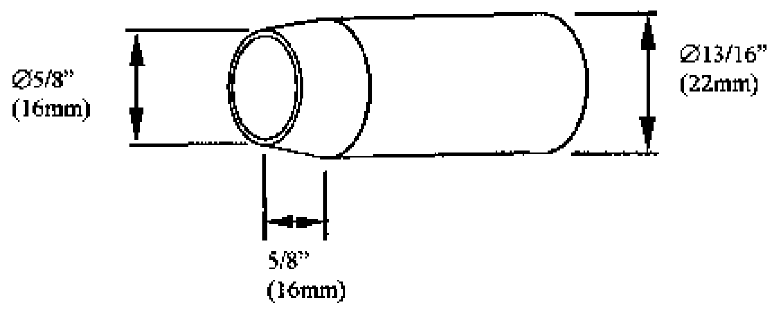


Figure 7 Chamfer dimensions for the Ø1/2" PVC pipe

- Hold the pipe firmly in an upright position (using a vice or pipe grips) and apply a generous amount of grease or Vaseline to the thread cutter or to the pipe end. This will reduce the friction while thread cutting.
- Place the thread cutter on top of the pipe and gently start to turn/thread it on to the prepared pipe, making sure that the top of the GI fitting is level as shown in Figure 8.

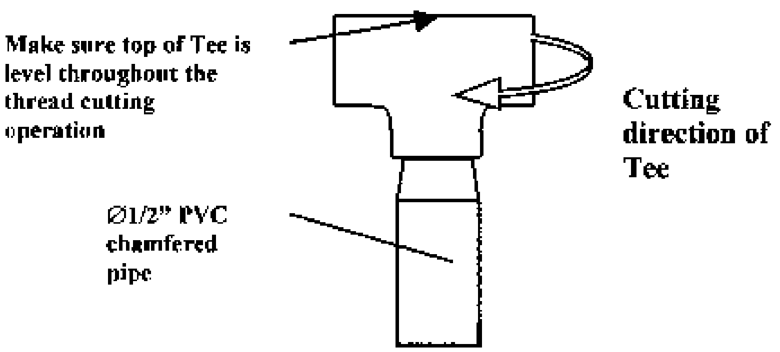


Figure 8 Starting the thread cutting

- Turn several times (by inserting a screwdriver or steel rod through the Tee) then remove and clean out the thread of any plastic that has built up. For every revolution of the thread cutter turn back again half a revolution, this will break off the material being cut and avoid clogging of the cutting edge space.
- Repeat the operation until a sufficient length of thread (no more than the length of thread inside the Tee) has been cut.

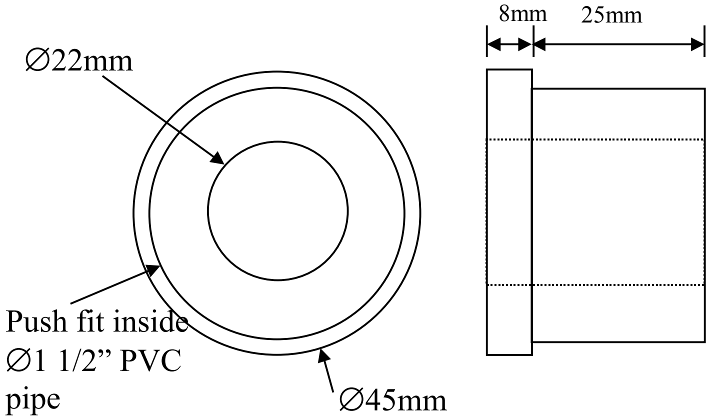
Re-sharpening the cutter is simply done by filing the cutting edge with the small triangular file until the blunt edge has been removed.
Polythene bags cut in to thin strips and wrapped round the thread is a good low cost substitute for PTFE tape.

Please note that it may take several attempts before a satisfactory thread has been made so practice on spare pieces of material until confidence and the quality is built up.

Appendix 3 Pull rod bushes and inlet details

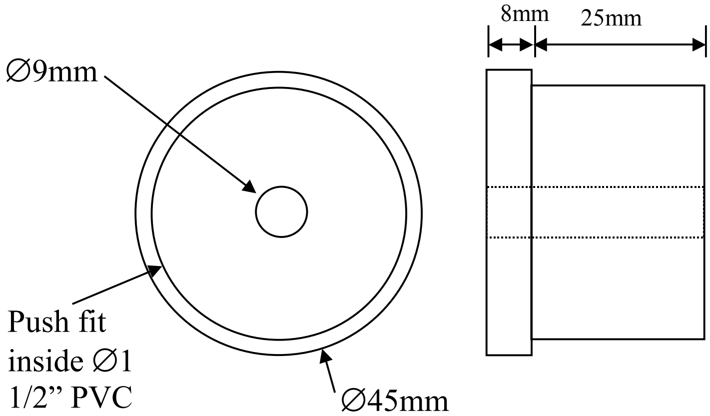
Pull rod bush for DTU and Tamana handpumps

Material: Hardwood



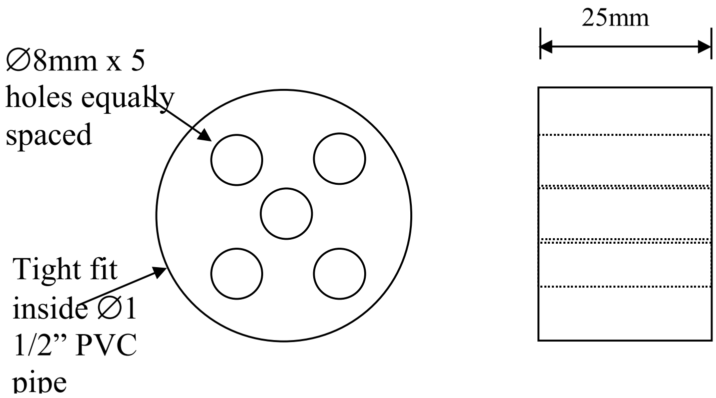
Pull rod bush for Harold handpump

Material: Hardwood

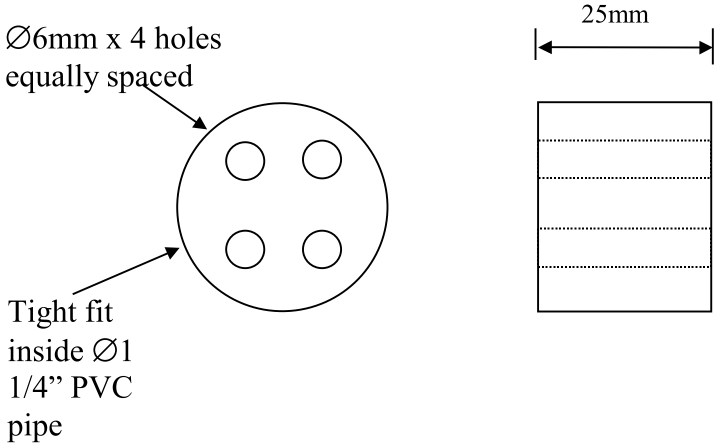


Inlet for ∅1 1/2” PVC pipe, Harold and Enhanced inertia handpumps

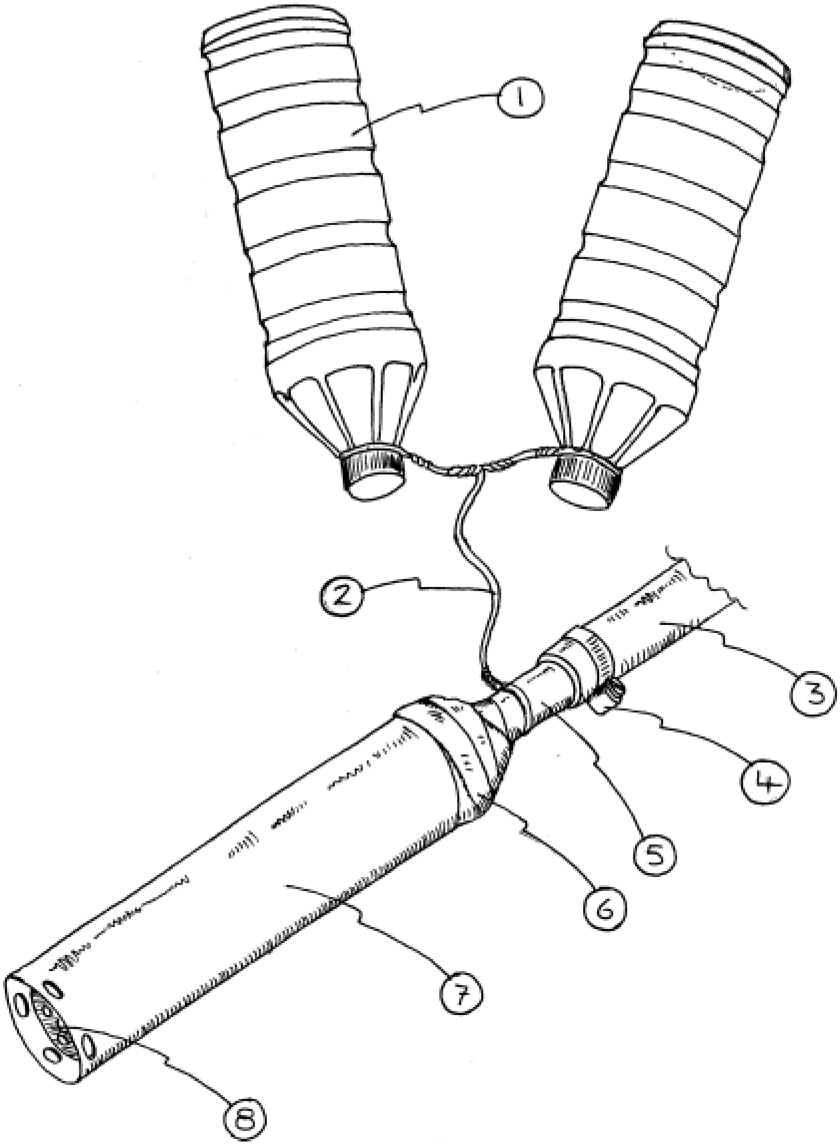
Material: Hardwood



Inlet for ∅1 1/4” PVC pipe, Enhanced inertia handpump

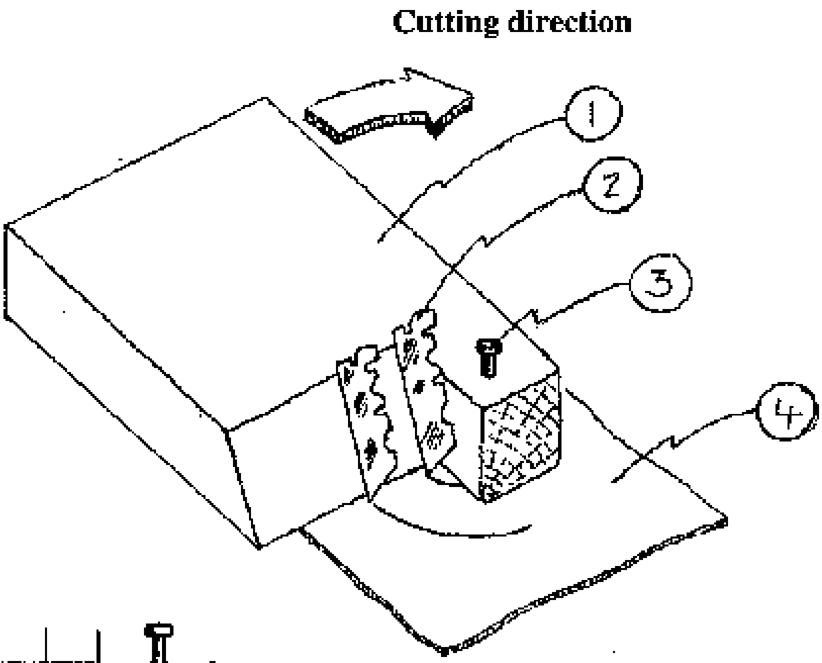


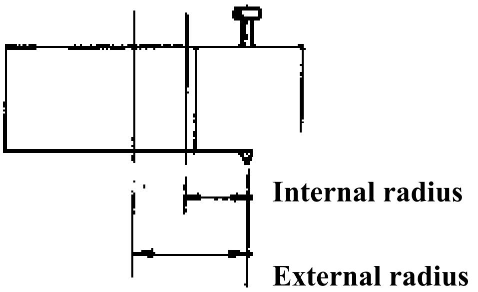
Appendix 4 The floating valve



1	Plastic bottles
2	Insulated copper wire
3	Flexible hose
4	Hose clip
5	Ø1/2" PVC pipe (approx. 150mm long)
6	Rubber inner-tube strip
7	Ø1 1/2" PVC pipe (approx. 200mm long)
8	Wood inlet valve (see appendix 1)

Appendix 5 The piston/valve seal cutter





1	Wood block
2	Razor blade sections
3	Pivot (nail)
4	Rubber being cut

The Tamana seal cutter is made from a suitable piece of wood with dimensions of 50mm x 12mm x 75mm.
The angle of the cut out is at 45° cut sufficiently far back so that the pivot pin and razor blades are all in line with each other.
A razor blade is broken in half and one end of each is broken again to give the acute cutting edge as shown.
After hammering a nail in for the pivot mark off as accurately as possible the internal and external radii taken from the outside diameter of the pull rod and the bore of the rising main respectively on to the wood. The razor blades are then pushed in at these marks making sure the bottom of the blades are at the same height as the pivot point.

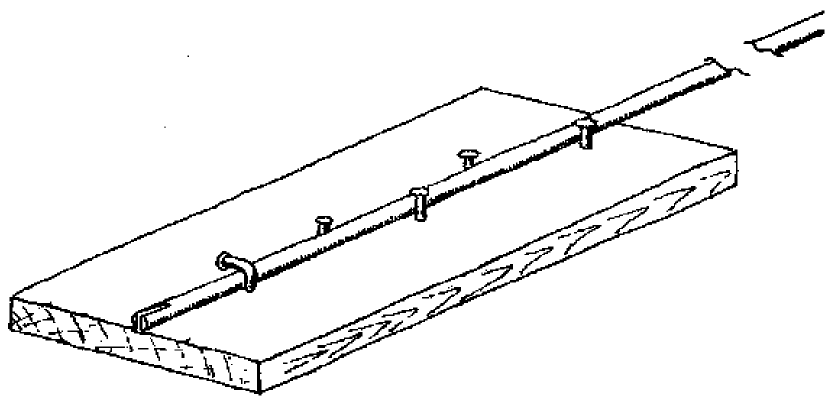
Cut out a small square of rubber inner tube slightly larger than the finished diameter and put the pivot in the centre of the square. Hold this with one finger while rotating the block in a clockwise direction as shown until the disc is cut through.

Appendix 6 Fixture for slitting Harold pump pull rod

The difficulty in obtaining a low cost vice has led to the development of this simple method of work holding. It uses parts and tools that are easily available and will hold a rod firmly enough to saw a slot in the ends.

It uses a series of nails that are staggered down the length of the rod as shown in the figure below. These are hammered into place on alternate sides whilst it is laid flat on a piece of wood. This traps in to position. The nail nearest the end to be sawn is bent over the bar so that it stops the rod from lifting upwards and sideways while sawing, and is turned sideways when removing the pull rod after sawing.

When starting the cut keep the saw at an angle of around 25 degrees and keep it at this angle until the right length of the saw cut is achieved. Then twist the rod round 180 degrees and saw to the same length. Once this is done gradually lift the saw up while sawing until it is 90 degrees to the pull rod. After sawing the rod can be removed by rotating the bent nail out of the way and the bar can be slid out.



Appendix 7 How to make the Harold pump moulded cup

Using the diagrams shown on the following page the procedures are:-

1 Cut a length of Ø1 ½" PVC pipe to 50mm long
2 Cut lengthways.
3 Heat gently and open at split
4 Heat again to produce a flat strip by pressing on a flat surface.
5 While not and pliable, use the wooden form to press into a short length of Ø1 ½" PVC pipe and hold till set (approx. 1 minute).
6 Mark off the centre of the cup and drill to suit the pull rod diameter (8mm).
7 Cut off cup so that target diameter is about 2mm less than inside diameter of 1 ½" PVC pipe.
8 Remove sharp edges with sandpaper.

How to make a wooden form for moulded cup.

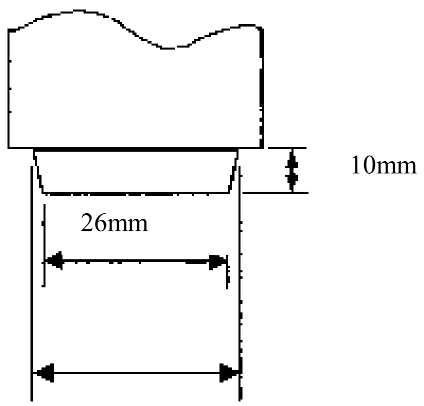
Cut a length of Ø1 ½" PVC pipe off at about 25mm long.

Using the inside diameter and mark off with a pen onto the end of a suitable piece of wood.

Mark off a second diameter of Ø26mm (this will give an angle of approximately 12°)

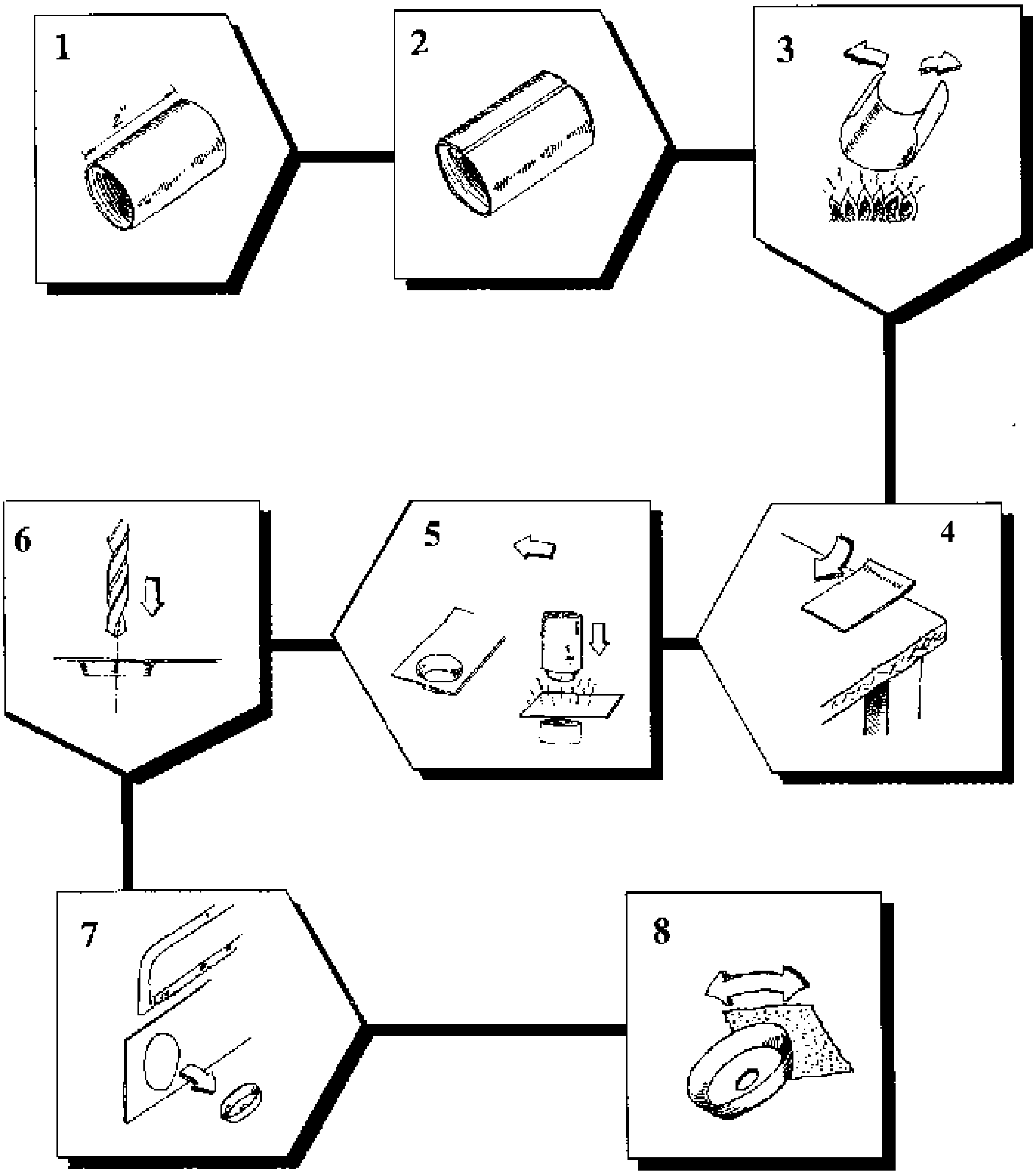
Mark off length of form to 12mm long.

File the taper to shape (or alternatively use a wood lathe)



To suit inside diameter of
pipe minus 2 x wall thickness

Sketches of manufacturing the moulded cup



TR-RWH 10 RAIN-JAR HANDBOOK

PRODUCTION AND USE OF MORTAR RAINWATER JARS

This Handbook was written with special reference to Uganda, whose climate is favourable to roofwater harvesting. It is based on 2-years immediate research in Uganda and several years Ugandan experience of jar production (however general Ugandan DRWH practice is based on other, and more expensive, forms of water storage). Unreinforced mortar jars are sometimes called ‘Thai Jars’ as they have been extensively used in Khon Kaen Province of Thailand since 1985 and are also common in Cambodia and Vietnam. In these countries, concrete rather than wooden moulds (as described below) are preferred: however trials of three type of mould when making 8 jars in Mbarara in 2005 indicated that wooden moulds are easier to use.

Costs are given in Ugandan shillings – USh.2000 = €1.0; USh.3000 = £1.0

Acknowledgment We acknowledge with thanks the financial support (in 2006) and encouragement of WaterAid Uganda that enabled the completion of mortar rain-jar development and testing, and the writing of this Handbook.




Contents

1 Why collect roofwater

Rainwater running off a roof is a very convenient source of household water, because you don't have to walk far to fetch it. If roofwater harvesting is done carefully, the quality of the water is similar to that of water fetched from boreholes or protected springs.

Roofwater harvesting needs

- A water store such as a jar, tank or underground cistern.
- A suitable roof (normally of mabati or tile – collection from thatch is rarely worthwhile)
- A guttering system to intercept the run-off and lead it to the water store

Normally roofwater harvesting is only considered for houses that already have a suitable roof, in which case, the water store is the main expense. This Handbook is about how to make a 1600 litre mortar jar (with little or no metal reinforcement). Such rain-jars provide one of the cheapest and most flexible forms of water storage – costs can be as low as USh.50 per litre of storage capacity (= €25 per cubic meter). The technology of manufacture is fairly simple, does not require an electricity supply and is suitable for a micro-enterprise in a village.

2 How much roof do you need; how much water can you harvest?

Most of Uganda gets a good rainfall (1000 to 1500 mm a year) spread over two rainy seasons. In all but the far North it is uncommon to have more than 8 weeks with no rain. In the 'Cattle Corridor' rainfall is less – 800 to 1000 mm a year – but adequate for harvesting. Only in NE Uganda is rainfall so low (under 800 mm a year) that roofwater harvesting is not rewarding. Appendix A lists Ugandan Districts and their rainfall status.

Roofing is a bigger problem, since although over 60% of houses have some 'hard' roofing, there are parts of the country where the fraction is much lower (whereas other parts have over 90% hard roofing). However for many reasons, metal roofing is steadily growing in popularity and the fraction of roofs suitably for roofwater harvesting will probably rise to over 80% within ten years. In extreme cases it is worth

Of the rain that falls on weven a hard roof, a fraction is lost as splash or evaporation. The rest can be led to a water store, but sometimes that store is already full so the new water is wasted as 'overflow' A small store overflows more than a big store. Using the water jars described in this handbook, you may assume that 15% of the rainfall on the roof does not reach the jar and another 25% overflows the jar, leaving 60% of the rainfall to be drawn from the store.

Table of expected yields from a roofwater harvesting system

The **bold** figure is thousands of litres per year; the figure in brackets **( )** is yield in litres per day for 8 months a year (during the remaining 4 months the yield is only half this figure)

District Rainfall (see Appendix A)	High	Medium	Low	Suggested number of jars
Large roof (100 m^2)	**78** (250)	**66** (220)	**54** (180)	4 - 6
Medium roof (60 m^2)	**47** (150)	**40** (130)	**32** (100)	3 - 4
Small roof both sides (40 m^2)	**31** (100)	**26** (85)	**22** (70)	2 - 3
Small roof one side (20 m^2)	**15** (50)	**13** (45)	**11** (35)	1 - 2

The table above is based upon a particular way of managing the stored water. It would be possible to draw 40% more water in the wet months if none were saved for the dry months.

3 Where to place rain-jars round a house

Most Ugandan houses are rectangular and longer (along the roof ridge) than they are wide. For this shape of building, the best place to put rain-jars would be near the centre of each long wall – at the front and the back. This arrangement is shown in Figure 1 and will work with small gutters (for example 2" wide ones). However many householders would not want to place a jar in front of their house but prefer it at the end of the house as in Figure 2 -

which will require bigger (say 3”) guttering. With jars it is **not** normally sensible to lead the water from both front roof and back roof to the same jar since the long piping will be ugly and too expensive for the benefit obtainable.

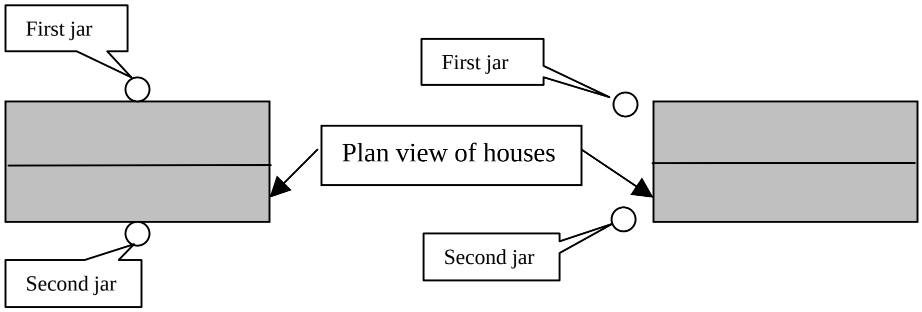


Figure 1 **Figure 2**

If the roof is single-sided – as is the case with many trading centre shops – the jars can be set side by side.

The same arrangement usually applies where more than 2 jars are used, as shown in Figure 3.

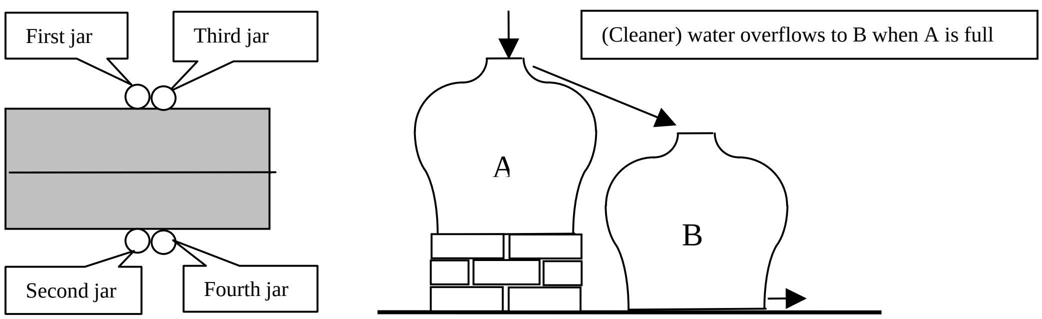


Figure 3

4 *Why mortar jars rather than other forms of water storage*

For storing very large quantities of water the ground itself is the cheapest store: thus wells draw from a reservoir of groundwater that is replenished during each rainy season. Artificial ponds (‘valley dams’) are another way of storing hundreds of thousands of litres of water. However neither the ground nor a pond is suitable for cleanly storing roofwater, for which we always install a jar, tank or cistern.

The main options for a roofwater store are as follows

Pottery jar	30 litres	v small & difficult to keep water clean	USh.100 per litre
Oil drum	180 L	if open-topped then water gets polluted	USh.170 per litre
Plastic drum	1000-12000 L	clean and light but expensive	USh.250 per litre
GI Tank	2000-10000 L	shortened life due to rusting	USh.70 per litre
Ferrocement tank	6000 L	built on site	USh.80 per litre
Partly below ground	6000-12000 L	needs a handpump and good ground	USh.65 per litre
Mortar Rain-Jar	**1500 L**	**built in workshop**	**USh.50 per litre**

5 *Making a jar – preparing for the two stages of production*

Each jar has a base and a shell (body). Both of these are made of un-reinforced ‘mortar’ (sand + cement) which once formed into shape needs to be cured for 7 days. So the main stages of production are (A) making the base disc, (B) making the shell and (C) curing.

Before starting work we need to establish a suitable working space (workshop) that has enough shaded space, tools for making mortar, somewhere to keep tools and materials secure at night and the special equipment needed for jar manufacture.

Space requirement Each jar needs about 5 square meters of floor-space. It needs to be both made and cured under shade from the sun. However production includes applying a layer of mud and then letting it dry, which may be difficult on wet days. A completely indoor workshop is perhaps too expensive and cannot use the sun to dry the mud quickly, however it offers the best security against damage to newly plastered jars and against theft of water. An outdoor workshop with shade matting cannot be used during rain. The ideal would be to build each jar on a trolley that could be moved in and out of the workshop door.

The number of jars in the workshop at any one time depends on how quickly they are sold once they are ready for sale. Jars are not safe to be moved until they have been cured for 7 days. So a workshop making 4 jars per week needs to be *at least* 20 square meters of shaded, hard and level space with 2.4 meters headroom, plus 'outside' space for jars awaiting collection. The workshop door must allow entry of the special jar-lifting cart, which is approx 2 meters high and 2.2 meters wide.

Tools A minimum tool-kit is
- Wheelbarrow for fetching and holding mud
- Spade
- Trowel
- 'Floats' (wooden for main plastering, metal for finishing)
- Level
- Container for measuring out sand and cement
- Blankets or grass to completely cover each jar during curing
- Polythene (such as DPC) for putting on the floor under the base disc and for covering jar during curing

Special equipment
- Two sets of wooden moulds (or more if production is to exceed 4 jars per week). For details of the moulds see Appendix B. There are other sorts of moulds such as filled-bag moulds and concrete ones. Each has disadvantages and we recommend wooden moulds.
- Jar-transporting hand-cart (both for delivery and to move jars from workshop to sales area). For details of the cart see Appendix C.
- Frame to allow each jar to be entered by a man/woman (in order to remove the wooden moulds in it 1.5 to 2 days after plastering.) without their touching the still-weak top of the jar.

Materials

The materials involved in constructing jars are cement, sand, mud, water and steel (rod and pipe). The cement is standard 'ordinary portland' cement (experiments substituting pozzolanic cement for Portland cement have not been done, so we recommend you **don't** use pozzolanic cement). To the cement, which should be bought from a reliable source, a waterproofing agent ('leak-seal') should be added and well mixed. The usual mix is 1kg of waterproofing agent to 50 kg cement.

Sand is a subject of much discussion in Uganda. 'Lake sand' is claimed to be the best for concrete and mortar, but it is often expensive to transport to the production site and we try to use local sand wherever possible. A local sand is 'suitable' for making mortar if
- It contains less than 10% 'fines' (as measured by the bottle test described below)
- Not more than 65%, and not less than 15%, of the sand is so small it will pass through a 0.4mm sieve (see the sieve test described below).
- It does not feel sticky when wet
- There is no organic material in it and it does not smell of vegetation

If the available local sand is too fine to satisfy this specification, it may be mixed 50:50 with a coarser sand, or with stone dust, brought from elsewhere.

Mud is required (1 – 2 wheelbarrows-full) for covering the wooden moulds as a release agent. This should be sticky enough to hold onto the wood, but not so sticky (so much clay) that it dries very hard. The producer may need to try out different sandy soils to make the mud from. See section 15 about possible problems relating to mud.

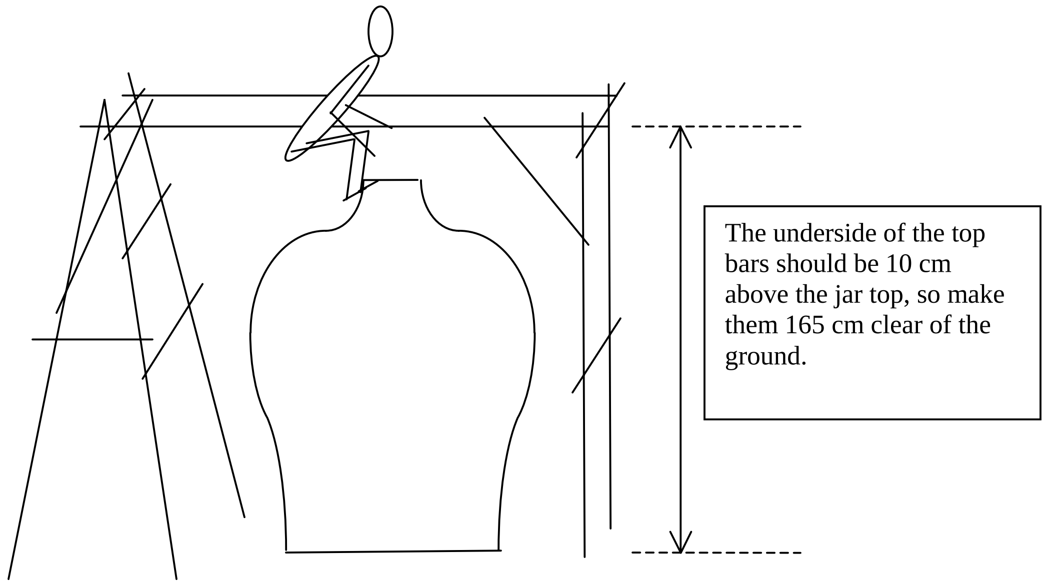


Figure 4 Frame of nailed poles for access into top of jar

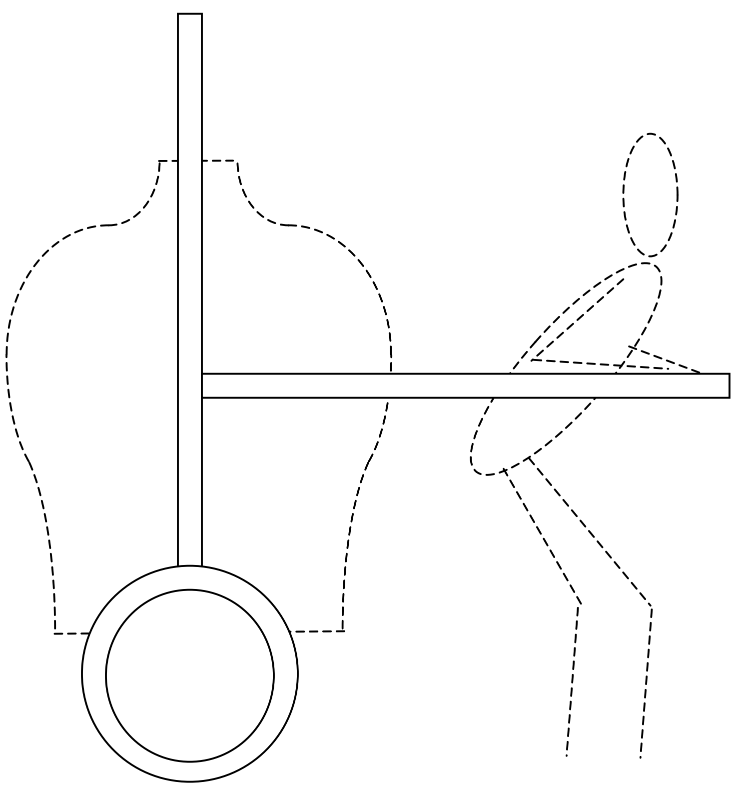


Fig 5 Special handcart (or donkey cart) for lifting, transporting and placing jars

Two Tests For Selecting Mortaring Sand

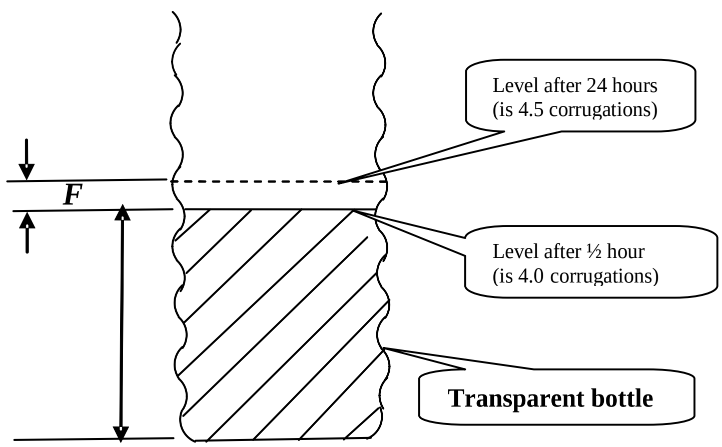
Bottle Test for seeing what fraction of a soil is 'fines'

Take the 'sand' you would like to use and break up any lumps with a mortar and pestle (or with the back of a spoon in a saucepan). Pass the soil through a very coarse kitchen sieve or through coffee-tray mesh. With what passes through the sieve, 1/3 fill a transparent bottle such as a ribbed plastic spring-water bottle and add clean water to nearly fill the bottle. (If you use chlorinated tap water, let it stand in an open pan for some hours to get rid of the chlorine.) Let the soil soak for 10 minutes, then shake it hard for ½ minute. At the end of shaking, immediately put the bottle on a flat surface in good light and start timing.

After ½ minute all the sand will have fallen to the bottom and will form a layer whose height *H* you should measure. (Sometimes you can use the corrugations of the bottle itself to measure height).

After 24 hours the 'fines', namely the silt and clay, will have fallen down to form a layer on top of the sand. The height of this extra layer we can call *F*.

For good mortar work *F* should not be more than 1/10 of *H.*



Sieve test to check the grading of the sand

You will need two sieves, a coarse one (with 1.2 to 2 mm holes) and a fine one (such as a household sieve for tea) with 0.4mm holes.

First sieve some soil through the coarse sieve and weigh out 100 grams (on a Post Office scale?).

Now sieve this 100 grams through the 0.4 mm fine sieve and weigh what goes through. Not more than 65% (or at worst 70%) should go through.

6 Stage A – Making the base disc

The base disc is a simple component of mortar (3:1 sand:cement), but it must be made before the tank shell and properly cured (= kept covered and moist). It is essentially a mortar disc (115cm in diameter x 25mm thick) in which other components are embedded. These components are

- a water outlet pipe (essential)
- a washout pipe (usually omitted)
- lifting handles (optional)
- a strip of steel mesh (optional)

The disc is cast onto a flattened surface covered with polythene and inside a metal strip 25mm thick bent into a circle and joined by adhesive tape to maintain its inside diameter as 115 cm. The mortar must be tamped so that it completely fills the space inside the strip – the mix being made **just** moist enough to do this reliably but still rather dry and stiff. After 2 hours, the metal strip can be removed (unwrapped). The top of the disc, for 5cm in from its edge, is made rougher by heavily pitting it with a trowel or spike or by cutting. In addition a 7 cm strip of (3-4mm) mesh may be buried in the part of the disc where the shell will sit on it, leaving about 4 cm protruding. The disc is next covered and kept moist for at least 6 (and preferably 24) hours to cure.

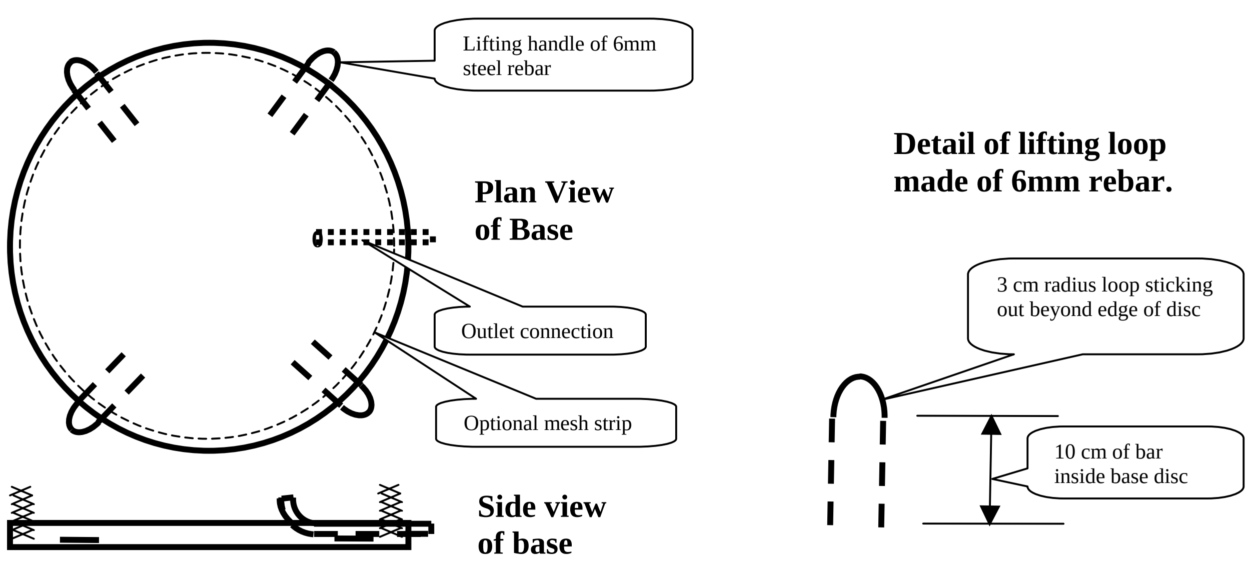
The outlet connection is set into the base disc as soon as the mortar there has set enough to cut a slot for it. The outlet (3/4" metal pipe + elbow or pipe and bent-up length of hose) is placed in the slot and mortared over. The inside end of the outlet (elbow or hose) should stand at least 2 cm above the surface of the base disc. This is so that outflowing dirty water is not drawn from the very bottom of the jar. The entry to this outlet should be plugged with wood or cloth to prevent mortar falling into it during later stages of construction.

The reason why a washout pipe is usually omitted is that an inlet filter should be used that prevents much debris ever entering the jar so that it doesn't need annual cleaning. Moreover the base disc is not thick enough to allow a washout pipe of useful diameter (over 25 mm bore). Cleaning a jar itself introduces further likelihood of contamination so is best avoided, however these jars are small enough that some accumulated mud can be removed without a person actually entering the jar.

Lifting handles can be incorporated in the base as an alternative to lifting the jar with a chain wrapped round its circumference below its widest point. (That is the method employed when the jar is lifted onto the Delivery Handcart described in Appendix C).

The base is now ready for moulds to be placed onto it as the first stage in shell construction.

Figure 5 The base of the jar



7 *Stage B – Making the jar shell*

The shell is made in 4 or 5 steps once the base has been cast and allowed to set (for at least 3 hours). These steps are

a) Assemble wooden moulds on the base and cover them with mud to roughly form shape of jar.

b) Let the mud dry then plaster over it with mortar.

c) Cover and cure for at least 48 hours, then remove the wooden moulds and mud and add 'nil' layer.

d) Cover and cure for 2 more days then (optionally test with water.

e) Cure for 3 more days.

Step	a)	b)	c)	d)	e)
Activity	Mud onto moulds	Plaster with mortar	Remove moulds, add 'nil'	Test with water (optional)	Completed, ready for delivery
		Cure under moist cover (see box titled 'Curing' below)			
Days	1	2	4	6	9

Notice that each jar occupies workshop space for 7-8 days and the wooden moulds are in use for 3-4 days. Thus each set of moulds can be used to produce, at most, 2 jars per week.

The steps above are now described in more detail.

a) Mould assembly and mud application on Day 1

The 5 layers of mould boxes (A to E) and the 4 layers of planks (F to I) are assembled on the base, using mud to fix them in position and to space one above each other. A spirit level can be used to ensure each layer is level. A basin (at least 45 cm diameter), weighted with stones, is placed at the top. A wooden bar may be used to support it, as shown below.

Figure 7 Joint between shell wall and base

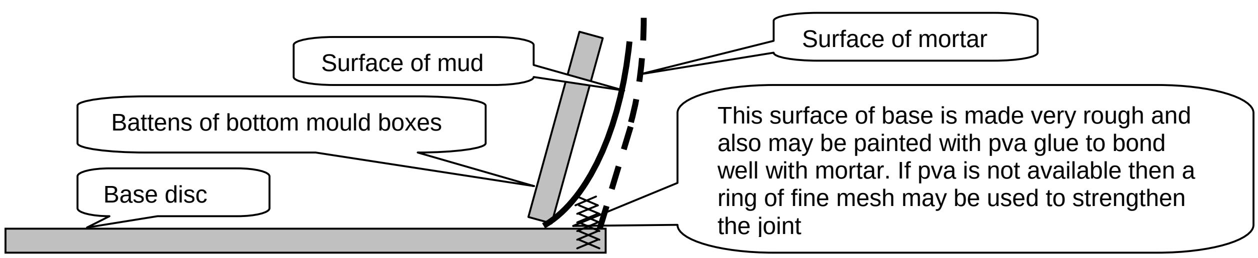
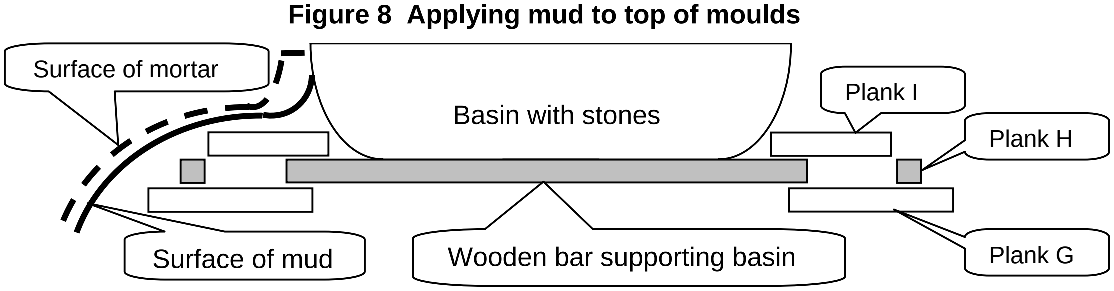


Figure 8 Applying mud to top of moulds



The moulds are then covered with a fairly thin layer of mud shaped to make the desired shape of the jar. The mud should be moist enough to stick to the battens on the outside of the moulds when thrown at them in the manner of a plasterer. The shaping can be with a wooden or metal plasterer's float. The edge 5 cm of the base disc and the protruding mesh (if used) should be kept clear of mud.

Curing

Unlike mud bricks or lime, cement needs to be kept as dry as possible during mixing and as wet as possible during the next week (the 'curing' period). As we are applying mortar by plastering, the water content of the mix has to be suitable for the plaster to stick to a vertical surface. However after a couple of hours, when the mortar has 'gone off' (hardened a little) we want to put it into a moist atmosphere to stop it losing water by evaporation. Experiments have shown that well-cured cement is much stronger than cement left out in the sun.

In practice we do three things:
- shade the jar from the sun
- put some water inside the jar and close its top opening
- cover the jar with wet grass or wet cloth and covering that with polythene sheet to keep the moisture in.

Curing for a long time means the workshop must be large enough to hold many jars. In practice 7 days curing should be enough. When the jar is transported to site it may experience bigger forces than after it has been placed in its final position. For this reason it is unwise to move it after only 3 or 4 days.

Because it must be applied and removed many times, it is worth making a polythene cover which can be slipped over the top of a jar and then tied together round the bottom of the jar. Under the polythene can be wet grass or wet blankets. Grass is probably easier to moisten and then keep moist. Blankets should be sewn together to make a jacket that can easily be put on and taken off a jar. They should be dried out after use on each jar to stop them rotting. Production workers are always reluctant to replace the exterior grass/blankets once they are removed, so it is recommended that only the material covering the top aperture is removed to allow access for demoulding on Day 4.

b) Let mud dry, then plaster over it with mortar on Day 2

When the mud is semi-dry, with a wet float fill over any large cracks that have formed and to give it an even surface. (If the cracks are more than 5 mm wide, the mud has shrunk a lot and therefore contains too much clay: this will cause problems during demoulding [step c)] – a sandier mud should be used for the next jar.)

The whole jar shell should be mortared, over the mud, to a uniform thickness of 10-12 mm with 3:1 well-mixed sand-cement. The cement contains 2% leak-sealing compound. The water content of the mortar should be that which allows good dense plastering – if it is too wet the final mortar will be weaker, if it is too dry the plastering will be rough and contain voids. On completion of applying the mortar, it should be re-trowelled with a well-moistened metal float to give a dense smooth surface. Later, see step c), the inside of the jar below the widest point will be given another 2 mm of 'nil' (cement + water slurry).

The top collar of the jar should be formed using the basin as a mould and (for best appearance) a scraper resting on the basin to ensure the collar is uniform thickness (say 25 mm), circular. The top should be flat or sloping slightly down to the outside.

The joint between shell wall and base is an important one and is shown in the figure above. The base must be very rough (pitted or grooved), clean and wetted where the wall meets it. If available (not in Uganda), pva glue, neat or diluted 1:1 with water, should be applied to the base and allowed to go sticky before applying the mortar. Where pva is not available and there is any uncertainty about the quality of the joint between base and wall, a strip of mesh will have been set in the base with 4 cm protruding. Bend this mesh outwards to allow mortar to be placed behind it, then bent it back upright, press it into the mortar and cover it with further mortar.

Some water should be put in the jar as soon as the mortar has hardened to ensure the atmosphere inside the jar is moist and the mud does not dry out further (thereby getting too hard and also perhaps stealing moisture from the mortar it touches).

c) Cover and cure, then remove both wooden moulds & mud, add 'nil' layer on Day 4

Cure for 48 hours (see Section 8 below). Now carefully remove basin and moulds without cracking the shell. This takes some skill and it may be prudent to start with mould removal on day 5 and change to day 4 once skill has been built up. To remove planks or boxes the mud holding them in place must be loosened with a knife or stick. Only the planks can be taken out from above. All the boxes are loosened and passed out by a person *inside* the jar. However to enter and leave the jar no force must be put on the jar's collar: instead the person enters and leaves using the framework shown in Figure 4.

After removing the moulds they should be inspected for any damage (like loose battens) and repaired if necessary.

All loose mud is scraped or brushed from the inside of the jar and taken out. The jar inside is then washed and the plug blocking the outlet pipe is removed. Do not leave much mud in the shallow pool at the bottom of the jar.

A slurry is made of cement and water using about 10 kg of cement. This is brushed onto the *inside* of the jar from its widest point downwards to the base to form a waterproofing layer 1 to 2 mm thick.

d) Cover and cure for 2 more days then (optionally) test with water on Day 6

Cure for 48 more hours. Now a strength test can be applied by filling the jar with water. As it is 4 days since the mortar was applied, the strength of the jar should have reached 50% of its final value. The different test options are described in Section 10 of this Handbook. The outlet hosepipe should be tied up the side of the jar to a wooden stick marked with heights (10cm, 20cm etc) so that the level of water in the jar can be monitored. If water is available it certainly a good idea to at least half-fill the jar (say up to 65 cm deep). this has two purposes:

- to make sure the jar has no serious defects low down (damp patches may form on the surface but these will disappear after 24 hours as the leak-seal compound reacts with the seeping water). If however there is leakage from a point defect, fine sand can be sprinkled onto the inside water surface immediately above the defect so that the sand is drawn into the leak to block it.
- to help it cure completely.

e) Continue curing until Day 9

Day 9 is 7 days after the mortar was made. Curing well (moistly) for 7 days should have developed most of the jar's strength so that it is ready for transport to site.

8 *Inlet, outlet and overflow arrangements*

The purpose of an **inlet filter** is to let in water but to prevent organic debris *or light* from entering the jar. The inlet to a jar is usually a basin that fits snugly into the top collar (e.g. the same basin that was used for forming the collar, whose diameter of 45-50cm was big enough to allow entry of the demoulder to the jar). Basins are slightly tapered and this taper improves the fit. The bottom of the basin should be pierced with many holes (for example using a heated nail) to let the water flow through it. The basin is next 2/3 filled with gravel sufficiently large that it does not slip through the holes. Finally the basin is covered with a cloth that allows water to flow through it. If the cloth is given an elasticated edge, it can easily be removed, washed and replaced. A stick can be used to raise the centre of the cloth so that some of the debris (twigs, leaves) that it catches will slide off it. However if the stick is too long (and the cloth-cone too steep-sided) some water will be lost too.

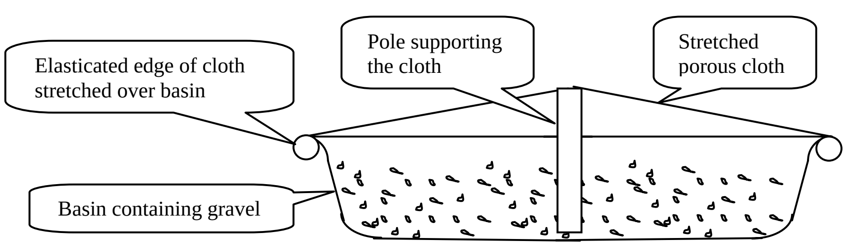


Figure 9 Basin used as inlet filter

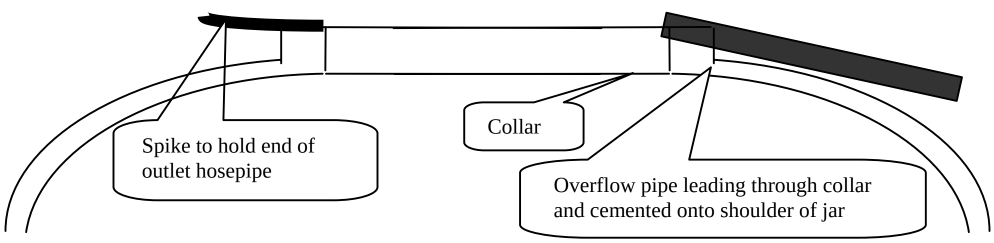
If the bowl is directly under the end of a gutter, or (even better) under the ends of two gutters discharging towards each other) then no down-pipe may be needed. If the water overshoots the basin during intense rainfall, a chain can be hung from gutter-end to basin to guide the run-off flow.

Some systems, especially those near dusty roads, have an arrangement called ‘first-lush diversion’ whereby water falling from the gutter can be temporarily diverted so that it misses the basin. This diversion is put into place during a dry season so that when the first rains come, the very dirty ‘first flush’ runoff doesn’t go into the jar. After 10 minutes of good rain, the runoff should become cleaner and it is time to remove the diverter.

The combination of a cloth-and-gravel inlet filter and the diversion of the silt-laden ‘first flush’ will ensure that the jar very rarely needs cleaning – perhaps once in 3 years. Frequent cleaning is not a good practice for rainjars as it introduces contamination and removes any slime layer on the walls that are digesting bacteria.

An **overflow** is needed so that when the jar is completely full any new roof-runoff water has somewhere to go. The simplest arrangement is to allow water to emerge around the rim of the basin and run down the outside of the jar. However it is often better to incorporate a 20-25mm bore pipe in the collar of the jar that leads out as far as the shoulder of the jar, so that overflow water can be directed in a chosen direction or taken to a second jar as shown in Figure 3 (in Section 3 above). Overflow water is generally a little cleaner than the water already in the jar, so the overflow jar should be used mainly as a source of drinking/cooking water.

An **Outlet** is obviously needed. We have shown the outlet pipe embedded in the base. This is because if the outlet pipe were just held by the thin jar wall, it would not be secure and big forces applied to it could break the jar. Where the pipe emerges from the base, at the bottom of the jar wall, it can be a simple metal tube onto which a hosepipe is fitted (and tied by rubber strap). The other end of the hosepipe should be hooked over a spike coming from the collar of the jar. Such a hosepipe outlet is cheap, easy to use to fill containers and can be easily replaced if broken or stolen.



However a hosepipe gives no protection against water being stolen or the tank contents being lost due to careless or malicious dropping of the pipe. So some users will prefer to have a tap which in turn requires a thread to be formed on the end of the tube emerging from the jar base.

Whether using a tap or a hosepipe, when the jar is nearly empty the water is available only at the level of the jar’s base. (The last 2-3 cm of water cannot be withdrawn – it is likely to be dirty with sediment). Thus to fill a jerrycan placed on the ground, the jar base must be on a plinth at least 30-40 cm high. Alternately the plinth can be lower and a pit dug next to it to accommodate the jerrycan. Such a pit may need a gravel base and a drain.

9 Testing the completed jar

A good jar is one of the right thickness (about 13mm at the bottom and 10mm at the top), good to look at, having no leaks and of course not breaking when filled with water.

We can roughly test the average thickness by noting how much cement we use. More than 1.6 bags and the jar is too thick; less than 1.2 bags and it is too thin.

The jar is made of mortar – a material that gets stronger over time. Mortar reaches almost its full strength 28 days after it was mixed, but at 7 days it has only developed 70% strength and at 4 days only 50%. So if we can half fill it at 4 days without it bursting, it is likely we could safely fill it completely at 28 days. Actually we want the jar to be more than ‘just’ strong enough so filling it completely on the 4th day shows it has a ‘safety factor’ of about 2. – a good value.

It is possible to apply an ‘over-pressure’ to the jar at 7 days or even 28 days after mixing the mortar. This is an interesting test to apply during research but too difficult to do during production, because it needs the inlet filter to be replaced by a special strong and watertight seal. Indeed all testing with water is a bit difficult because it requires a lot of water (75 jerrycans) to be available, to be put unto the jar and taken out again. The likely water tests are therefore

a) half-fill every jar at 4 days (with 750 litres of water) to confirm safety factor >1,
b) completely fill every jar at 4 days (with 1500 litres of water) to confirm safety factor >2,
c) completely fill at 4 days every 10th jar that is made to check that the production procedures are sound,
d) completely fill at 7 days every jar whose purchaser pays to have it so tested (say USh.5000/-)

Although passing test a) does not absolutely guarantee that a jar is sound, it requires less water than the other loading tests and it enables any low-down cracks to be identified. It is very difficult to effectively repair cracks, so a jar found to have vertical cracks over 50mm long should be abandoned.

Test b) is the most severe.

Passing tests d) shows a safety factor of at least 1.5.

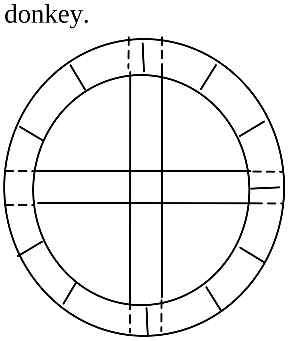
The overall requirement for test-water is reduced if water can be transferred from jar to jar. Half the water in a jar can be transferred to another jar by gravity (through a hosepipe connected from outlet to outlet), but the rest of the water will need raised by hand-pump or by lifting jerrycans.

When a jar is first filled with water its outside surface will become moist and remain so for a day or more until the waterproofing additive develops its power to seal off seepage.

10 Delivering and placing the completed jar

The jar, once made and purchased, needs to be delivered. It is heavy (say 270 kg) and not easy to grasp. The easiest way is to suspend it from two hooks by chains sleeved with hosepipe. One chain is fitted round the jar at about 40 cm above the ground: the chain is joined by a bolt with washers and a nut. Two more chains are bolted to this ring-chain to form loops that will connect to carrying poles or to a special transporter cart. The cart, sketched in Figure 5, has hooks about 1.8 meters off the ground which can be lowered to 1.55 meters when the cart is tilted back. So the jar chains are attached to these hooks in their low (tilted back) position and then – with the pull of several people – the cart is pulled upright, so lifting the jar 20-25 cm. The jar is then tied with ropes to the frame of the cart (at all points of contact a car tire is used to buffer the mortar jar from the steel frame) and the cart is pulled or pushed by 3 people or a donkey.

A jar can be carried on a pick-up truck (or even, in flat country, on a motorcycle trailer. However in both cases it is difficult to lift the heavy jar on and off the vehicle. Therefore for distances up to 6 km in a low-wage country, delivery by handcart is likely to be the best option, even though it needs a team of three people or one person and a donkey.



On arrival at site, where a plinth of 40 cm height has already been built, the jar must be lowered onto the plinth. This will require temporary ramps to be placed for the cart wheels to run up – for example planks supported by two bricks at one end. The arrangement is shown in Figure C2 of Appendix C. Once positioned above the plinth, the jar is lowered onto it. The plinth should be solid, given a level mortar top and then be surfaced with 1 cm of wet mortar just before the jar is placed on it. A suitable plinth would be built on a rammed foundation 15 cm below ground level and take the form of a circular wall with one or two interlocked diameter walls – as shown in the plan alongside.

11 Costing the jar production

The costs below are in Ugandan shillings in 2006 at which time the exchange rate to one Euro was about USh2000/-

There are three main costs associated with jar production (up to the point a jar is ready for delivery) namely:

- Materials and components
- Labour
- Use ('rental') of capital equipment.

Materials and components

Each jar requires		
Cement (opc) 65-75kg		USh.24000/-
Sand (assumed local sand 'balanced' by up to ½ 'Lake sand' brought from afar) 200 kg		USh. 2000/-
'Leak-seal' waterproofing agent, 1 kg		USh. 1500/-
Basin and outlet pipe/hose		USh. 6000/-
[optionally 3m x 8cm mesh		USh. 1000/-]
	Total materials	*USh.34000/-*
Labour		
Assuming a production rate of 4 jars per week from a team comprising 1 mason and 1 labourer		
Labour cost per jar is USh.(60000+20000) / 4		*USh.20000/-*
Use of capital equipment		
Wooden moulds costing USh.150000 and assumed to last for 30 jars		USh. 5000/-
Premises – 6 m^2-wk per jar @ USh.700/m^2/month		USh. 1000/-
Hand tools and frame - USh.100000 divided over 6 months production		USh. 1000/-
Water for testing, assume some recovery so 300 L per jar @ USh.5000 / m^3		USh. 1500/-
	Total rentables per jar	*USh. 8500/-*
On site cost		
A plinth of height 400mm and diameter 1100mm needs to be built at the site for each jar and the jar mortared to it after delivery		*USh. 6000/-*
Total without delivery		***USh.68,500/-***
Delivery by handcart over 3km (cart rental & unskilled labour)		*USh. 8000/-*
So the cost per delivered jar (with no profit to the producer) would be		***USh.76,000/-***

The investment required to set up jar production at a rate of 4 per week is		
Moulds (2 off) x USh.150000		USh.300000/-
Tooling		USh.100000/-
Handcart		USh.350000/-
Work in progress (4 jars not delivered)		USh.270000/-
	Total investment excluding premises	**USh.1,020,000/-**

12 "On-site' and 'factory' production of jars compared

There are two ways of making mortar jars for use at a household. Under 'on-site' construction the jar is made in its final position alongside the house; under 'factory' construction the jar is made in a workshop and then carried to its final site. With large above-ground tanks and with all underground tanks, there is no choice – they have to be made 'on site'. With the mortar jars described in this Handbook, which weigh about 270 kg, there is the possibility of carrying them to site and hence the option of making them in a small workshop 'factory'.

The advantages of ‘factory’ production are

- a higher level of production quality control can be maintained, hence allowing thinner jar walls and the use of less cement,
- suitable sand can be found and used,
- A permanent shading structure can be created and used rather than having to be created anew for each jar,
- the bulky moulds (occupying 1.5 m^3) do not need to be carried to the customer’s house,
- the jar can be made and displayed prior to purchase – the householder does not have the uncertainty about timing or quality that comes with having to commission a mason to visit their home to construct jars,
- if each (or every say 5^{th}) jar is to be water-tested, which requires 1500 litres of water, this water can be held at the workshop and used for testing many jars,
- the jar-builder does not have to visit the site three times (for moulding, for plastering after the mud has dried and for demoulding) or stay on site for 3 days.

The advantages of ‘on-site’ production are

- the jar does not have to be transported or lifted onto its plinth,
- some materials and perhaps a little labour can be provided by the householder,
- the location of any jar subsidised by some aid programme can be guaranteed.

13 What can go wrong in jar manufacture

Making jars takes some skill. The first jar made by a new producer is likely to have faults and will probably need to be discarded.

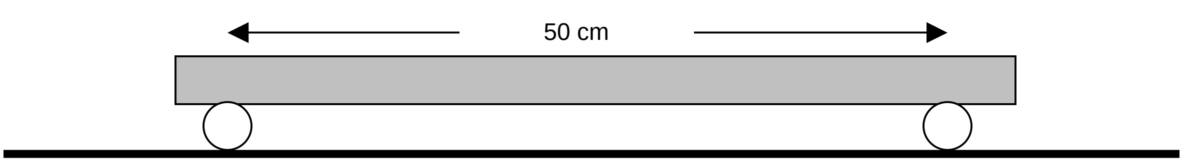
A badly made jar may fail either by leaking or by actually bursting when filled with water. It may also look mis-shapen or ugly. Here is a list of what could go wrong:

- the cement and/or sand is of poor quality
- the jar develops cracks when the moulds are being taken out
- the jar leaks where the shell joins the base
- the jar shell is too thin in places
- the jar was not cured properly
- the jar looks irregular.

(a) Poor cement and/or sand

Cement quality is often a problem in Uganda. The cement is of variable quality before it reaches the market and often deteriorates due to bad storage thereafter. Occasionally cement is adulterated with ash etc. There have been several recent building collapses in Kampala that may be due to poor cement. It is not practical to thoroughly test cement but a test for really poor cement is as follows:

Mix sand:cement in proportions 3:1 and cast three mortar bars (inside a wooden mould) each 5cm x 5cm x 60cm. Make sure the mortar is not too wet and tamp it well into the corners of the mould. Cure it well – 7 days under moist conditions. Now put it on two round poles resting on flat ground and placed 5 cm from each end of a bar as shown. Put a load of 10 kg onto the centre of each bar. If all bars break the cement is quite unsuitable; if only 1 bar breaks it is barely suitable (the jars should be made thicker than normal). If all the bars survive the “10 kg test” the cement-sand combination is acceptable.



In the **materials** paragraph of section 5 of this Handbook, there is advice on selecting sand.

(b) The jar develops cracks when the moulds are being taken out

When a jar contains water, its walls are subject to tension so that any crack will tend to open up slightly: cracks cannot carry any tension. Vertical cracks more than 5cm long make the jar much weaker, and it may fail by bursting when filled. If the crack is 10 cm long and is found anywhere except near the top of the jar, the jar will almost certainly fail. The main cause of cracks is too much force being applied when taking out the moulds. This may be due to wrong practice (esp. to not scraping away enough mud before taking out each mould box) or due to the mud being too strong when dry (containing too much clay). So inspect the jar for cracks before and after taking out the moulds to see if demoulding has produced new cracks. If it has, try to be more careful next time, or wait longer (more than the two days listed in Section 7 of this Handbook), or make the mud more sandy.

(c) The jar leaks where the shell joins the base

The shell is made about 2 days after the base, so the joint between the two is a joint between 'new' and 'old' mortar. This is not ideal but it cannot be avoided. So to make a good joint we want the part of the base under the shell (wall) to be very rough – pitted with a chisel or grooved, completely cleaned/washed of mud and at least 4cm thick. If there is significant leakage under the bottom of the wall, for the next jar pay more attention to roughening and cleaning. If PVA adhesive is available, paint the edge of the base with a 1:1 mix of that before mortaring the shell wall onto it. Alternatively introduce mesh into the joint, as described in Section 6 as an option.

(d) The jar shell is too thin in places

It is almost impossible to measure the wall thickness of a jar (unless it breaks) so the main check on thickness has to be during plastering. Remember the lower part of the jar (below its widest diameter) needs to be about 2 mm thicker than the upper part. Using too little cement (under 1.25 bag) in the 3:1 mortar is an indication the walls are *on average* too thin. However if a jar fails it will probably be where it is thinnest. So if a jar does fail during water testing, examine the pieces to see whether any are thinner than 12mm (lower part) or 10mm (upper part).

(e) The jar was not cured properly

It is hard to persuade masons to take moist curing seriously, as many believe that cement is like mud and has to dry out. There is no test you can apply to test whether curing was good, so you cannot ever be sure poor strength is due to inadequate curing. Thus good practice has to be enforced during production and it is especially important that the jar is not exposed to the sun during its curing period, that 'wet covers' are replaced after demoulding and that as soon as possible some water is put inside the jar.

(f) The wall-base joint leaks after the jar is delivered although it did not leak earlier when tested.

The cause of this is inadequate support from the plinth. The plinth must meet the base disc all the way round and also not be made of such soft material that the disc can sink perceptibly under the weight of the water in it (1.5 tonnes). Inadequate foundations for the plinth may also allow the jar to gradually lean over after some months of use.

(g) The jar looks irregular

The good shape of the jar mainly depends on the skill of the plasterer when applying the mud and therefore should improve with experience. Jars produced in Thailand over the last 20 years look very smart and 'machine-made' although they use only hand-plastering. It would be possible to make a shaped piece of plywood to place across the whole top of the jar to check its symmetry after mudding, but this is not really necessary. However such a guide could be more useful in making sure the top collar (round the basin) has a level top, is of uniform thickness and looks a clean circle.

The surface of the jar can be made smoother by painting it with a thin layer of weak 'nil' (= cement plus water) or with whitewash/lime.

14 Instructions for using rain-jars

[These instructions need to be passed on from the jar-maker to the jar user by word of mouth or by some sort of document.]

How much water should you draw from each jar?

A rainjar accepts water on wet days so that you can use it later on dry days. However you need to ‘manage’ the stored water to get the greatest benefit from having the jar. The method of management is to decide how much water to draw from the jar each day – an amount that may vary from one season to another.

Sometimes a jar overflows because there isn’t room in it to accept more rainfall: overflow water is wasted. If you draw more water from the jar each day, that creates more space for new water to enter and so reduces overflow. Up to a certain point, the more water you draw in the wet season, the more water you get.

However if you draw too much water in the wet months there won’t be much left at the beginning of the next dry season.

We recommend the following way of balancing wet and dry season use, by varying what you try to draw from the jar according to how much water is still left in it.

The jar is marked with three bands:

Band 1 (coloured red) is 0-600 litres. If the water level is in this band, you haven’t much left and we recommend you draw ***not more than 2 jerricans a day*** from each rainjar. If there is no more rainfall, the water will last for 15 days. To make it last a month without rainfall, reduce the demand to ***not more than 1 jerrican a day***.

Band 2 (coloured yellow) is 600-1000 litres. If the water level is in this band, we recommend you draw ***not more than 3*** jerricans a day from each rainjar.

Band 3 (coloured green) is 1000-1500 litres. If the water level is in this band, we recommend you draw ***not more than 5*** jerricans a day from each rainjar.

How can you ensure high water quality?

Properly used, roofwater is safe to drink and cook with. For this to be so you should:

Clean out your gutters twice a year – more often if your roof is under trees.

Make sure the filter bowl is always in position, that it is full of gravel or very coarse sand and its cloth cover is in position. Wash that cover whenever it gets clogged with leaves or mud and replace it when it gets torn or decayed. This inlet filter stops vegetation, lizards, rats and insects from entering the jar.

The first rains of a new wet season will wash much dust off the roof, making the run-off water very cloudy. So for two days after such ‘new rains’ boil the water from the jar before drinking it. Some people also divert the roofwater away from entering the jar during the first half hour of such new rains. If you live by a dusty road it is also worth sweeping down your roof at the end of the dry season (but not leaving the sweepings in the gutter!)

It should NOT be necessary to regularly clean out your jar: such cleaning may do more harm than good. In any case do not let anyone enter the jar – if you want to clean out a jar, do so from above using a cup on the end of a stick.

Mosquitoes

Although adult mosquitoes and their eggs can easily get into a rainjar, new mosquitoes will not breed in it provided organic matter like leaves is not allowed to enter and the water in the jar is kept DARK. (Light enables microscopic plants to grow and indirectly to feed the mosquito larvae.)

16 General conclusions

1500 litre mortar jars can be made in a simple workshop at a “workshop-door” cost of around USh.70,000, including testing with water on Day 4. After adding the cost of small gutters, a plinth, transport to site and some producer profit, the price might rise to USh.100,000. This is cheaper per litre than other roofwater storage technologies available in Uganda, and such capacity can be installed in easy steps up to say 6000 litres (USh.350,000/-).

However successful jar production requires skills and an attention to detail unlikely to be obtained from a casual employee or self-help householder. Therefore the technology is only suitable for serial production by an artisan who carries the consequences of poor quality control.

Capital of about USh.1 million is needed to set up each workshop.

APPENDICES

Appendix A Classification of Ugandan districts by rainfall

The table below (for all Ugandan Districts created by end 2004 but not those created since) categorises each District according to its annual rainfall (averaged over the whole District). In some large Districts there is such variation *within* the district that some parts may be wetter and other parts drier than as shown in the table.

The four rainfall categories are
High H (over 1200mm); **Medium** M (1000-1200mm); **Low** L (800-1000mm) and
Very low U (under 800mm and therefore unsuitable for roofwater harvesting).

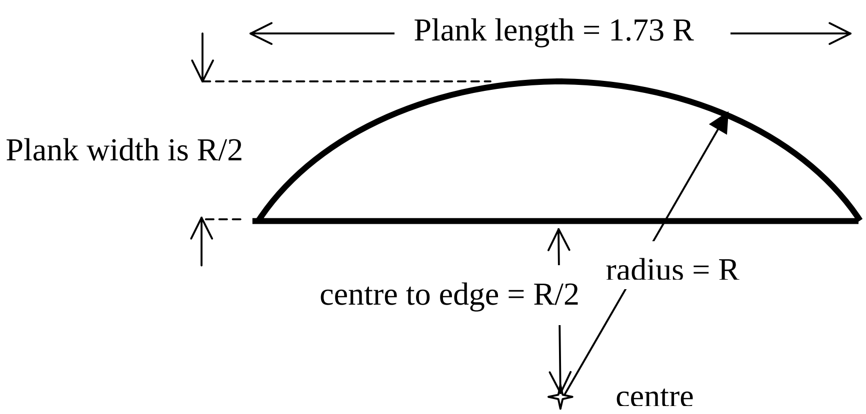
District	R	District	R	District	R	District	R
NW Region		*E Region*		*Central Region*		*SW Region*	
Adjumani	M	Bugiri	H	Kalangala	H	Bushenyi	H
Arua	H	Busia	H	Kampala	H	Kabale	M
Moyo	H	Iganga	H	Kayunga	M	Kanungu	M
Yumbe	H	Jinja	H	Kiboga	M	Kisoro	H
N Region		Kaberamaido	H	Luweero	M	Mbarara	L
Apac	H	Kamuli	M	Masaka	M	Ntungamo	L
Gulu	H	Kapchorwa	M	Mpigi	H	Rukungiri	M
Lira	M	Kumi	H	Mubende	M	*W Region*	
Kitgum	M	Maguye	H	Mukono	H	Bundibugyo	H
Pader	H	Mbale	M	Nakasongola	L	Hoima	M
NE Region		Pallisa	M	Rakai	M	Kaberole	M
Amuria	M	Sironko	M	Sembabule	L	Kamwenge	L
Katakwi	L	Soroti	M	Wakiso	H	Kasese	M
Kotido	U	Tororo	H			Kibale	M
Moroto	U					Kyenjojo	M
Nakapiripirit	U					Masindi	M

Appendix B Instructions for making a wooden mould-set (for a 1500 litre Rainjar)

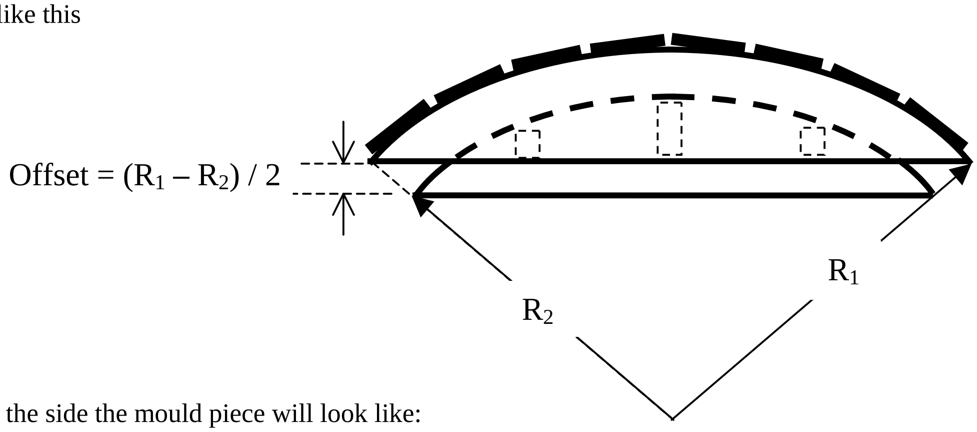
A mould set consists of 5 layers of open wooden boxes (A to E) and 4 layers of plain planks. Each mould box consists of

- A top plate
- A bottom plate
- Spacers to keep the two plates the right distance apart
- Battens connected the curved edges of the two plates together.

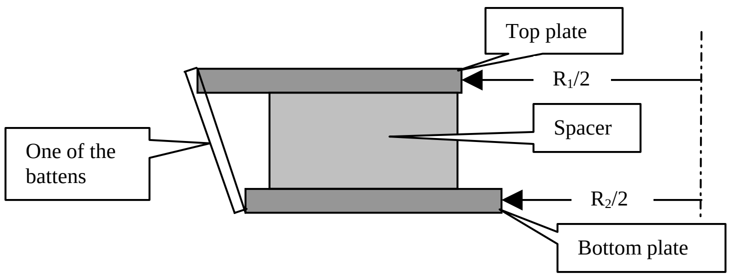
A typical full plate has the shape below. Its curved edge makes exactly 1/3 of a circle.



A typical mould piece uses two plates with different radii (R1 and R2) but the same centre and from *above* might look like this



From the side the mould piece will look like:



The height *H* of the mould piece is the measurement from top face to bottom face,
Make *H* = 230mm.

The plates are made of wood 20-25mm thick, so make the spacers 190mm high.

The battens should also be made 240 mm long.

Table of dimensions for Mould pieces

Layer (200mm)	Plate	Radius, mm	Width, mm	Offset, mm	Length, mm
A (bottom box)	Bottom (a)	480	240	12	830
	Top (b)	505	253		874
B	Bottom (b)	505	253	25	874
	Top (c)	555	278		960
C	Bottom (c)	555	278	28	960
	Top (d)	612	306		1059
D	Bottom (d)	612	306	14	1059
	Top (e)	640	320		1107
E (top box)	Bottom (e)	640	320	-85	1107
	Top (f)	470	235		813
F (first plank)		430	126		608
G		380	111		537
H		320	95		455
I		250	73		354
Extra planks D-E		*640*	*320*		*1107*

Wood required

We need two qualities of wood: fairly good and well-dried (for the top and bottom plates of the boxes and for the top planks) and rough unplanned wood for the outside battens.

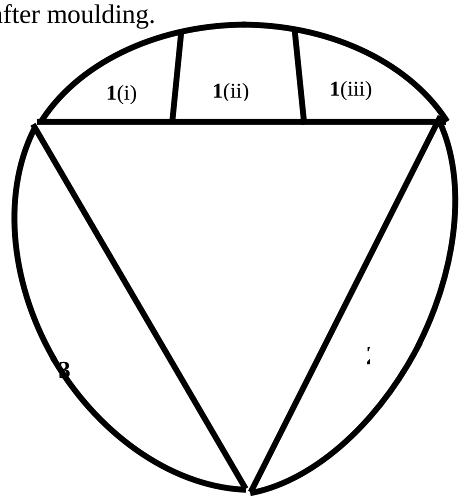
Good quality wood (the individual plates can be cut so that they overlap a little along the plank and hence there is a little saving on total length) comprises:

- Planks F, G, H and I need 6” wide wood of total length about 7.5 meters
- Plates (a), (b) and (f) need 10” wide wood of total length about 10 meters
- Plates (c), (d) and (e) need 12” wide wood of total length about 16 meters
- Plates (e) need an extra strip attached to their inside edge to make them wide enough.

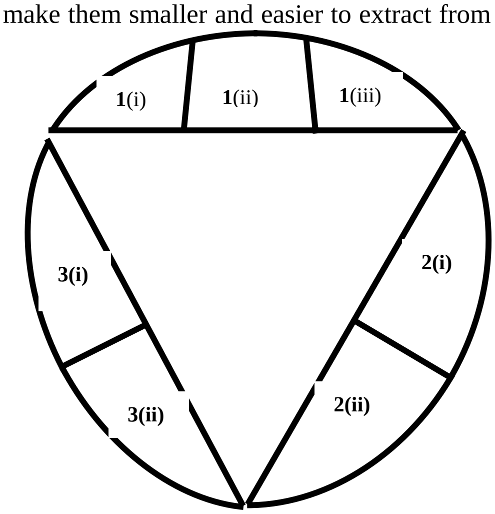
Such good wood costs USh.6000 to USh.12000 per plank of length 3 to 3.5 m.

Production

Each box layer is divided into several mould pieces. First three ‘full-size’ pieces are made for each layer (A to E), then one of them is cut into three parts as shown below **1**(i), **1**(ii) and **1**(iii), so that it is easy to extract them first [starting with **1**(ii)] when taking the moulds out, after the mortar has cured for 2 days. For the top two layers (D and E) it is a good idea to also cut pieces **2** and **3** into two parts, to make them smaller and easier to extract from the jar after moulding.



Lower layers
A, B and C, 5 pieces



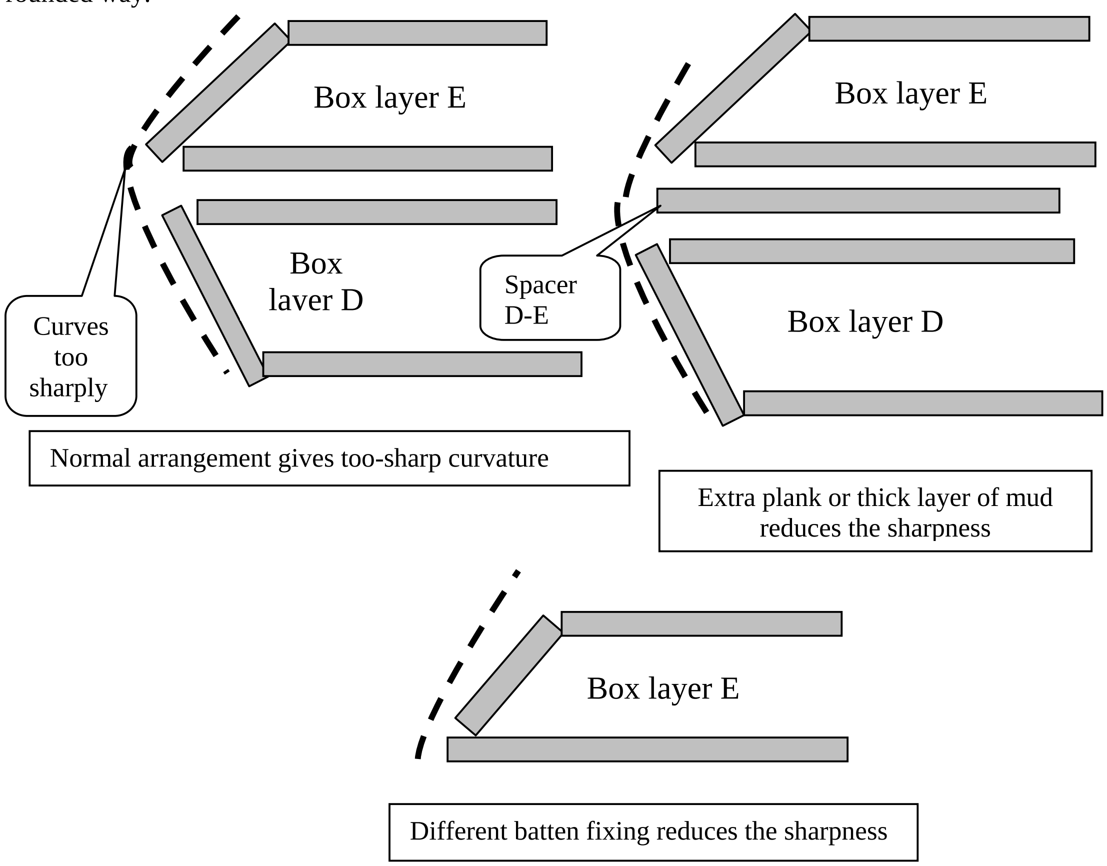
Upper Layers,
D and E, 7 pieces

The actual shaping of the planks can be done quickly and cheaply with a panga by a skilled woodworker, can be cut with a saw or can be machined in a mechanised carpentry shop. It is helpful to first mark out the shape of each piece onto thin (1/8") plywood and cut out a set of patterns. Only one pattern is needed for each of the plates (a) to (e) and planks F to I. These easily transportable patterns are used to mark out the 1" planks.

After cutting (and perhaps transporting), the 1" thick plates and spacers need to be nailed together in the right relative positions (with correct offsets) and then the battens are nailed onto the outsides. 2" (=50 mm) nails should be used.

Making the jar without too-sharp corners

Because layers D and E meet at a sharp angle, the finished jar will also have such a sharp angle, which does not look good. This can be avoided by adding an extra plank layer (called *D-E* in the table).Other alternatives are to use a lot of mud between layers D and E, or to shape the edge battens on boxes D and E so that they meet in a more gently rounded way.



Normal arrangement gives too-sharp curvature

Extra plank or thick layer of mud reduces the sharpness

Different batten fixing reduces the sharpness

Top pieces

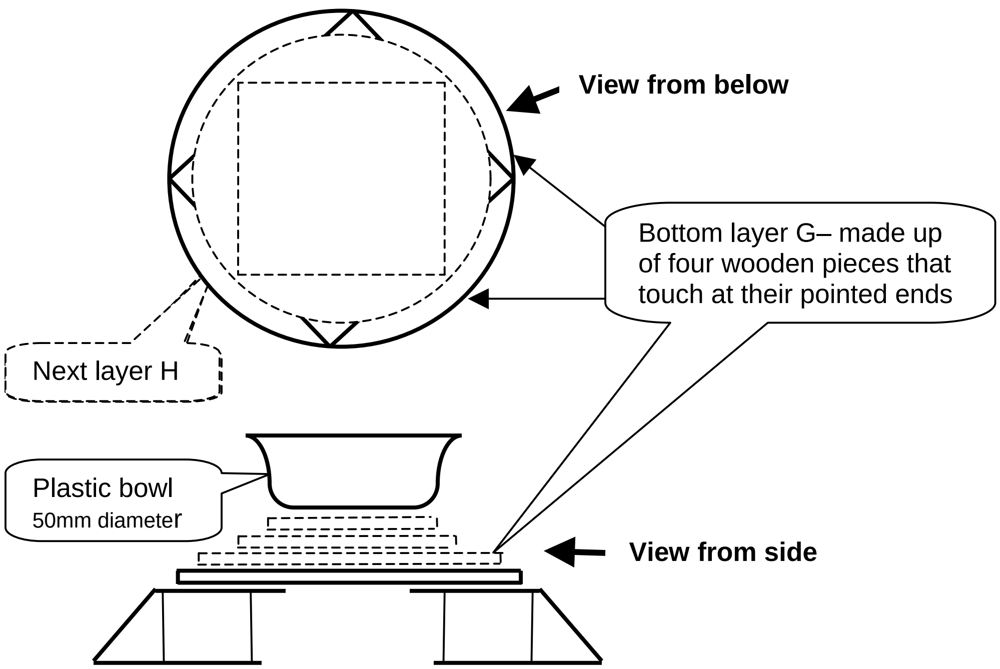
To complete the top of the tank there needs to be several layers of thinner wood. This is because the neck of the tank decreases in size fairly rapidly. The top pieces are not hollow boxes but planks (with no battens attached to their rims) laid directly onto each other.

There should be four layers (F, G, H and I) each of thickness 25mm (separated with a layer of mud), with four pieces per layer. The layers will be placed on top of each other so that they overlap. A plastic bowl is placed on the top layer to act as mould for the neck of the final jar. Removing these planks should be quite easy, as they will slide out and can then be extracted through the top of the partly-cured mortar jar.

The drawing below shows two of four layers on top of each other (as viewed from below). The radii will decrease from each layer to the next. Suggested radii for 4 layers (x 25mm) are given in the table above.

The plastic bowl is not just a part of the mould as it will later be the inflow screen. Its diameter needs to be enough (at least 45mm) for a man to climb through. The bowl edge should be rounded (as shown) and not folded over.

Assembly of top pieces (F to I) and bowl on top of boxes E



Appendix C Instructions for making a jar-delivery cart

Jars are 140 cm wide and 140 cm high and weigh over 250 kg: they are therefore very difficult to lift onto a motor vehicle for transport from workshop to house. Therefore a handcart (which can also be a donkey-cart) has been developed specifically to lift jars, transport them and lower them into place. Such a cart can also be used to move jars round the workshop.

The main parts of such a cart are

2 wheels,

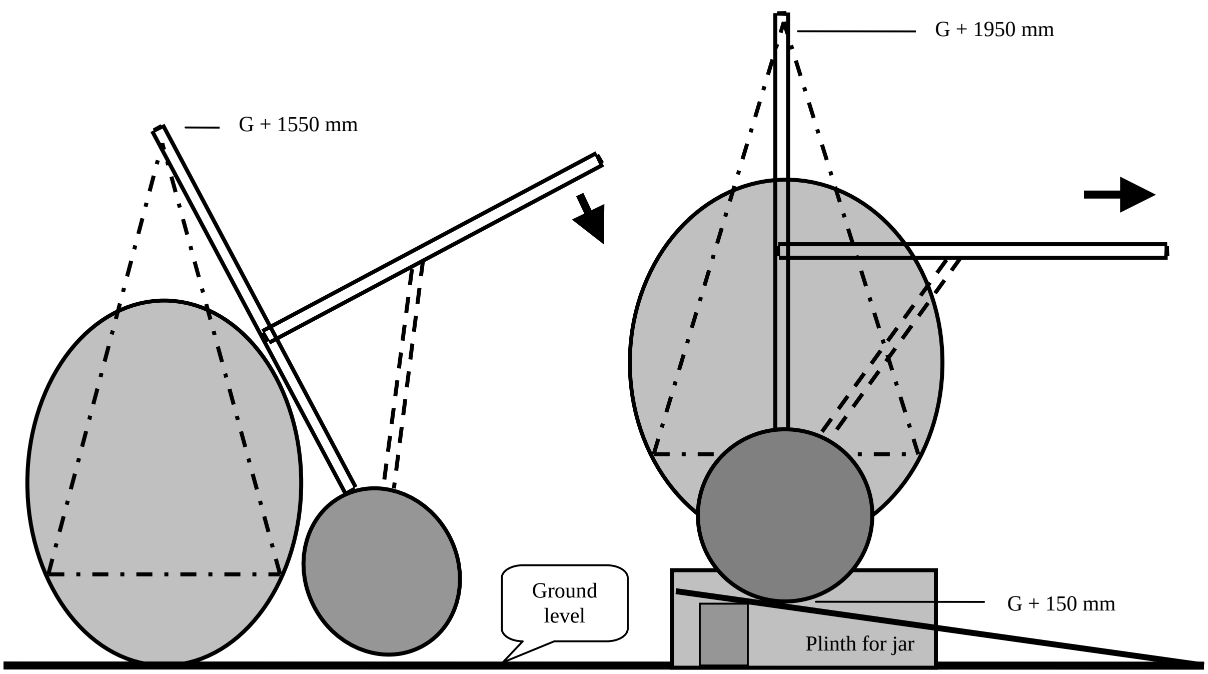
2 wheel bearings but *no* axle crossing from side to side,

A frame with hooks from which a cart may be suspended by chains,

One or two forward shafts (one shaft for 2 donkeys, two shafts for one donkey)

A suitable cart that uses motor-vehicle inflated wheels is described in Technical Release TR56 “Water tank delivery cart for two donkeys” that can be downloaded from web site www.eng.warwick.ac.uk/dtu/pubs However the drawings in this TR56 are in fact for a one-donkey cart (two shafts). Moreover the TR shows a winch above the cart frame for lifting the jar. Most users will omit this feature and instead use tilting the cart frame as their way of lifting the jar off the ground.

Figure C1 Tilted cart, jar on ground Figure C2 Upright cart on ramp, jar 400mm above ground



In Figure C1, the tilted cart has just been attached to the jar by chains (15-20mm steel chain, sheathed in plastic piping and joined together by bolts and nuts).

Before transport the cart must be pulled upright, so that the shafts are horizontal and the jar is clear of the ground. The jar is then secured to the frame by ropes. A cushion such as a car tyre is placed between jar and frame to protect the jar from damage.

During transport it is pulled (and pushed from behind) with the shafts horizontal and hence the load balanced.

After transport, on arrival at the site, the cart is run up onto two planks as shown in Figure C2 so that it is high enough to clear the plinth built to receive it. The cart can now be tipped a little to lower the jar onto the plinth.

The cart described in TR56 is built of heavy hollow, square-section, rolled steel (RHS) 50 x 50 x 3 mm. This thick section is only available in Kampala. One alternative is to make such a section by welding together pairs of angle iron (each 50 x 50 x 3 mm), but this is expensive. Another alternative is to use a thin but widely available section (50 x 50 x 1.2 mm) and add extra members to the frame such as that shown dashed in Figure C1. The bottom ends of the two frame uprights (where the wheels bearings are to be welded on) can be stiffened by forcing a short length of 1½“ water pipe up inside them.

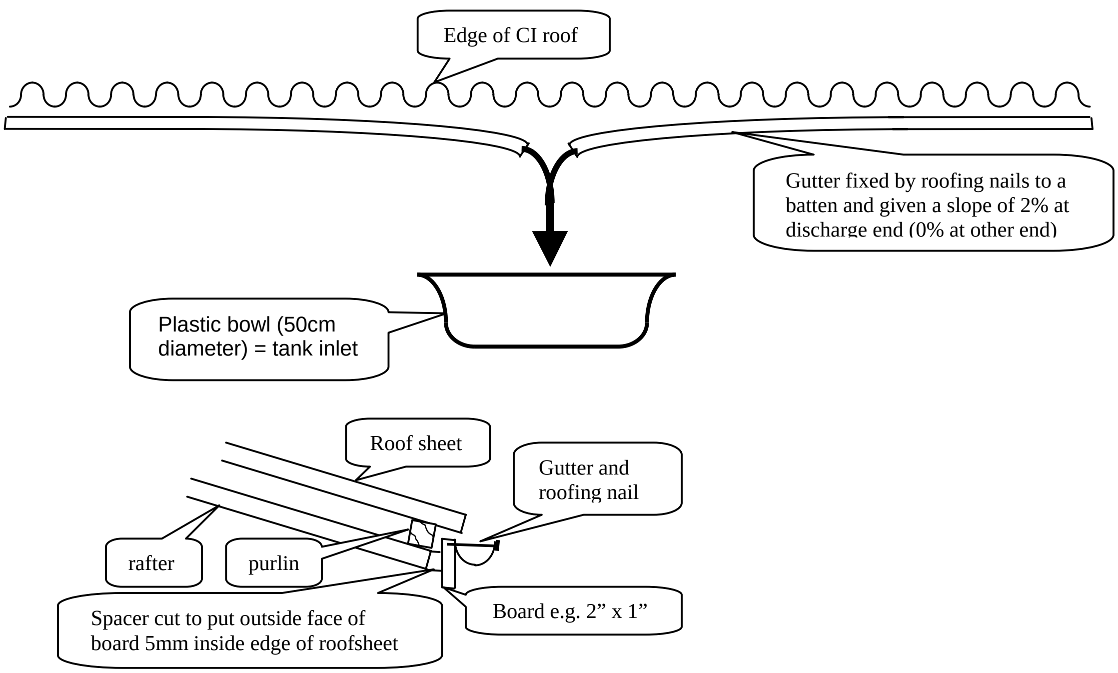
Having made the steel frame, choose suitable scrap car wheels (rim + tyre); no good tread is required. A 14” rim with 4 bolt holes is suitable. (If a pipe-in-pipe bearing is used – see below – then the hole in the wheel centre should be large enough to pass over a 2” water pipe.

Either find a scrap front-wheel car bearing that fits the wheel (usually available in Kampala) and weld it onto the frame (with or without the brake disc still attached)

Or make a pipe-in-pipe bearing, which is a 2” water pipe rotating round a 1½” water pipe. The space between the pipes is filled with grease and with a spiral of 1mm thick plastic cut from a pvc pipe. The outer pipe attaches to the wheel. The inner pipe is welded to the frame. Further details are given in TR28.

Appendix D Guttering

There are many demonstrations in Uganda of guttering that is ineffective or unnecessarily expensive. Gutter slopes are often wrong, large guttering is commonly used where small gutters would suffice, joints leak etc. The PVC guttering widely used in other countries is not yet available in Uganda. With jars, long gutters are not required and it is not necessary to lead water from one side of a house to the other. Ideally each jar should be fed from its own 8–15 m^2 of roofing via its own guttering. If the jar is fed from both sides by short (e.g. 3m) length of guttering, then these gutters can be as small as 50mm width – for example a medium-gauge 2" pvc pipe sawn in half. The 'non-discharge' end of the gutter needs to be blocked with wood or by using heat to fuse the two sides of the gutter together.



If the jar can be placed right under the gutters, no further connection is needed. However the jar is 140 cm diameter and usually needs to be placed a little further out than the roof edge. In this case a short length of sloping larger gutter (say 4") needs to be placed perpendicular to the roof edge to bring the runoff out to the tank inlet bowl.

Appendix E Labelling demonstration and normal jars

If information is painted onto jars by a signwriter, then the jar 'explains' itself. The following 4 boxes show possible patterns of information.

Yellow box

This Rain-jar holds 75 jerrycans (1500 litres) of water.
It was made on *[Date]* using 1.1/4 bag (70kg) cement and no steel reinforcement.
With one rain-jar + 6 m of gutter you can harvest 60 litres of water a day in the wet months. With 4 rain-jars + 16 m of gutter you can harvest 100 L a day in wet months, 60 L per day in dry months.

Green box	**Orange box**	**Red box**
1500 litres	**1000 litres**	**600 litres**
When the water level is in this band, draw up to 5 jerrycans per day.	When the water level is in this band, draw not more than 3 jerrycans per day.	When the water level is in this band, draw not more than 2 jerrycans per day.

The green band is at the top of the jar, the orange band in the middle and the red band at the bottom.

DTU

Ram pump programme

Computerised ram pump calculators
A short user guide

TECHNICAL
10
RELEASE

DTU Ram Pump Programme

Computerised calculator programmes

Hardware requirements

The minimum hardware requirement for these programmes is an IBM compatible PC with an 8088 processor. If you have the minimum requirement, only the DOS programme DOSPUMP will run. Although all three programmes should work on an IBM compatible machine with an 80286 processor running Windows 3, we recommend that the Windows programmes are used on an IBM compatible machine with at least an 80386 SX processor and four megabytes of RAM that is using Microsoft Windows 3.1 (or later versions).

Contents

1. Calculator Installation

There are three programmes on the 720k 3.5" disk inside the cover of this book. Two of them must be installed on a hard disk and run under Microsoft Windows®. The third programme runs under DOS and can be run from the floppy disk drive or installed onto a hard disk.

The two Windows programmes are:

WINPUMP
PUMPDATA

The DOS programme is:

DOSPUMP

You will not need to use both the Windows and the DOS programmes because they do the same thing. If you have a version of Microsoft Windows® on your computer, do not bother to install DOSPUMP as well as the other programmes.

1.1 Installing the Windows ram pump programmes

To install the ram pump programmes that run under Microsoft Windows® insert the disk in the floppy disk drive, then type the letter identifying the drive, followed by a colon (:).

For example:

A:

Then press the [Return] key.

The [Return] key is also called the [Enter] key and often has an arrow on it, like this ↵.

Type **PWIN** followed by the letter identifying the hard disk drive being used, a colon (:) and a backslash (\), and the directory name **DTUPROGS**, which is where the programmes will be installed.

For example:

PWIN C:\DTUPROGS

Then press the [Return] key.

When the programmes have been installed, a message appears advising you that installation is complete and telling you to press any key to continue.

Press the space bar.

The Windows programmes have now been copied to your hard disk, but Microsoft Windows® does not know this. The next stage of installation tells Microsoft Windows® that they are there and installs them as Icons at the Microsoft Windows® Programme Manager screen.

To do this, type the letter identifying the hard disk drive where the programmes have been copied, followed by a colon (:).

For example:

C:

Then press the [Return] key.

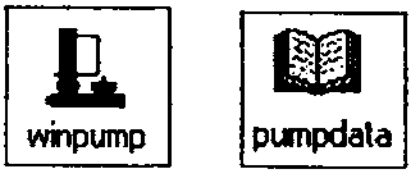
Type **CD\DTUPROGS** and press [Return] to change to that directory on your PC.

Type **SETUP** and press the [Return] key.

Microsoft Windows® starts and the DTU programme installation window appears. The DTU programme installation window has two buttons:- OK and EXIT.

Use the mouse to click on the OK button to start the installation.

The programmes are installed in a few seconds with progress being shown in the bar above the control buttons. A programme group called DTU programmes is created under Microsoft Windows' Programme Manager, and the DTU Windows programmes are installed there with these icons.



1.1.1 To run the Windows DTU calculator programmes — WINPUMP and PUMPDATA

Start Microsoft Windows® and click the mouse on the icon for the programme that you want. Click twice quickly (using the left mouse button) to run the chosen programme.

1.2 Installing the DOS DTU ram pump calculator programme — DOSPUMP

To install the ram pump calculator that runs under DOS, insert the disk in the floppy disk drive, then type the letter identifying the drive, followed by a colon (:).

For example:

A:

Then press the [Return] key.

Type **PDOS C:\DTUPROGS**

When the programmes have been installed, a message appears advising you that installation is complete and telling you to press any key to continue.

Press the space bar.

If you do not have a hard disk, the programme can be run from the floppy disk.

1.2.1 To run the DOS DTU calculator programme — DOSPUMP

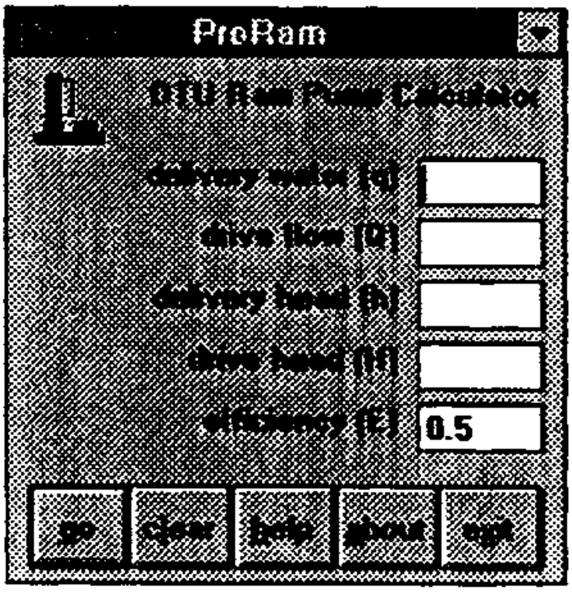
There are two ways to run this programme — from the hard disk or the floppy disk.

Running from the <u>hard disk</u>	Running from the <u>floppy disk</u>
Type the letter identifying the hard disk drive where the programmes have been copied, followed by a colon (:). For example: C: Then press the [Return] key. Type **CD\DTUPROGS** and press [Return] to change to that directory on your PC. Type **DOSPUMP** and press [Return] to start the programme.	Put the floppy disk into the drive, then type the letter identifying the floppy disk drive, followed by a colon (:). For example: A: Then press the [Return] key. Type **DOSPUMP** and press [Return] to start the programme.

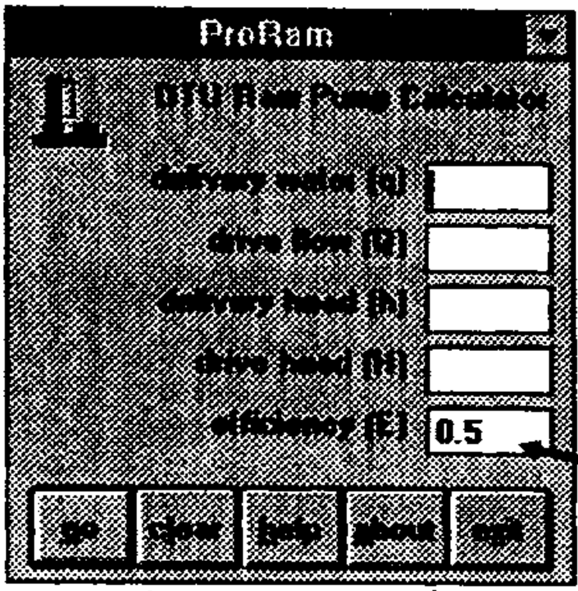
2. Using the DTU programme WINPUMP

When you click on WINPUMP the ProRam window opens. This window is used to enter variables to work out the equation: $E\ (Eta) = \frac{q\ h}{Q\ H}$.

This is the ProRam window:



Winpump is used by filling in any four of the variables for a ram pump system, (Q,q,H,h, and E) and clicking on the GO button. The missing variable is then provided for you.



Use this button to "minimise Winpump, but leave it running in the background behind other programmes.

When you have made an entry in one box, press the Tab key to move to the next box (or use the mouse to click on it.

You can change the value given for "efficiency" if you know that your pump has a higher or lower value. The value is expressed as a fraction of 1, so a 75% pump efficiency would be typed in as 0.75.

The buttons each have one letter underlined. Each button makes something happen, and it can be made to do this by either typing the underlined letter, or clicking on it with the mouse.

If you are using more than one pump, always remember to divide the drive flow between them before calculating.

When you have made one calculation and wish to make another, click on the "clear" button. All the values will disappear, except for 0.5, which is the default pump efficiency.

If you want to know the pump efficiency of an existing system, delete the entry for efficiency and fill in the other variables, then click on the GO button.

To leave WINPUMP, click on the "exit" button, of type the letter X.

WINPUMP's usefulness is limited, but it can be very handy for making quick calculations of the effect of changing parts of a system's design. For example, you can quickly find out what difference it would make to the delivery flow if you added a metre to the drive head or reduced the drive flow.

An example is when a system design is actually going to deliver more water than is needed and by varying the entries for drive head and delivery head we can see whether we could save money on the drive pipe, or deliver to a point high enough above the originally planned delivery tank for the excess to be usefully gravity fed to a garden plot.

DOS users will find that the DOSPUMP programme is a very similar calculator to WINPUMP. It looks different because it runs under DOS, but it does the same things.

3 Using the DTU programme PUMPDATA

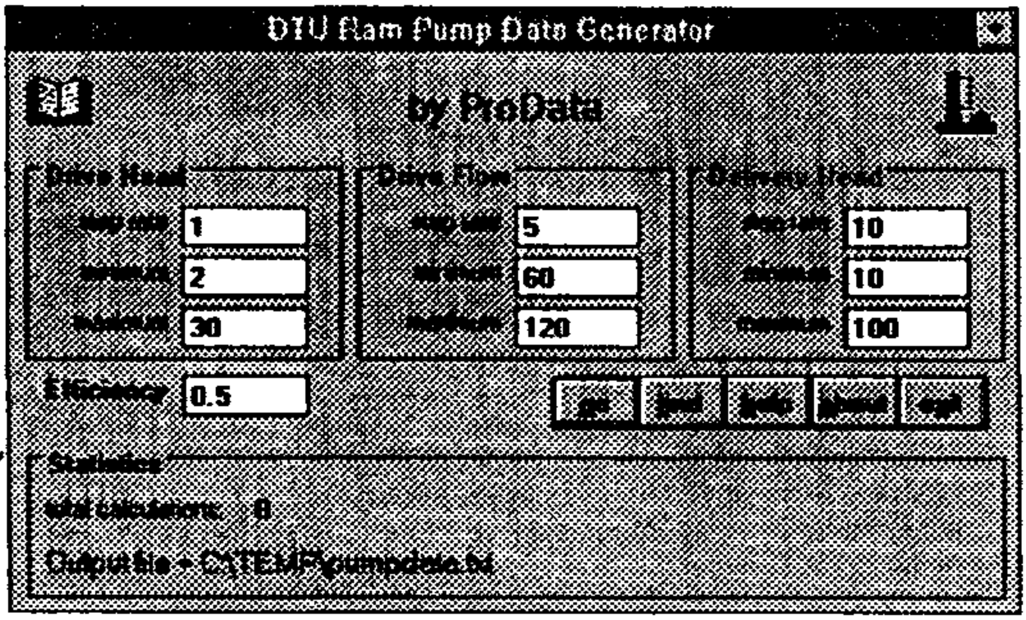
When you click on PUMPDATA the DTU Ram Pump Data Generator window opens. This window is used to enter variables to work out the equation

$$q = \frac{\eta Q H}{q h}$$

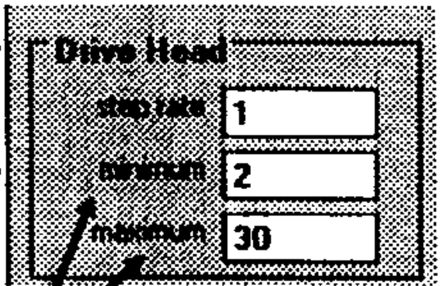
for a wide range of conditions. All possible answers within the ranges given are calculated. The range of results can then be sorted and printed.

This is the DTU Ram Pump Data Generator window:

A range of values for the Drive Head (H), Drive Flow (Q) and Delivery Head (h) are calculated for a particular pump efficiency. If you know the efficiency of the pump in use, begin by changing the efficiency value (which is given as 0.5 or 50% by default).



When the efficiency has been set, the "Drive Head" figures are entered (in Metres).

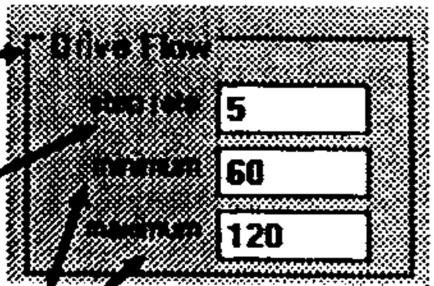


The "step rate" is the amount by which the value is advanced for each set of recalculations. For example, with a step rate of 1, the Drive Head will be calculated for every value between the minumum of 2 and the maximum of 30 (whole numbers only). If the step rate is changed to five, the minimum number will be advanced in fives to the maximum.

Change the step rate to a bigger figure to reduce the number of calculations carried out if your computer is very slow.

Change the minimum and maximum Drive Head to those available at the site being calculated.

When the "Drive Head" range has been set, the "Drive flow" figures are entered (in litres per minute).

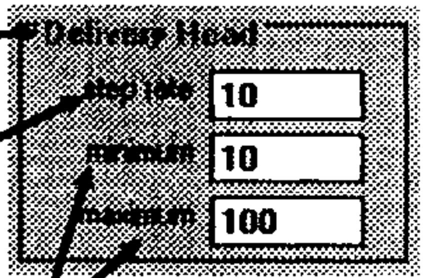


The "step rate" is the amount by which the value is advanced for each set of recalculations. For example, with a step rate of 5 as shown, the Drive flow will be calculated in fives from the minumum of 60 to the maximum of 120 litres per minute. If the step rate is changed to 10, the number will advance in tens from the minimum to the maximum.

Change the step rate to a bigger figure to reduce the number of calculations carried out.

Change the minimum and maximum Drive flow to those available at the site being calculated. If a single flow is known, type that flow into both the minimum and the maximum boxes.

When the "Drive flow" range has been set, the "Delivery Head" figures are entered (in metres).

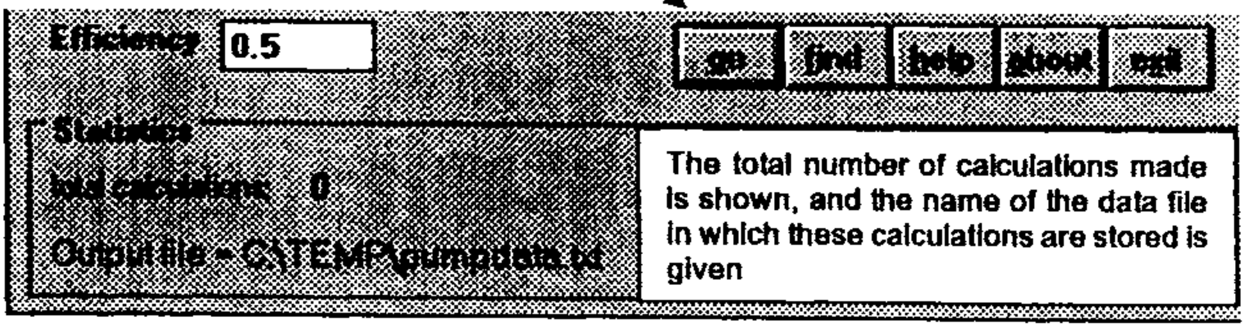


The "step rate" is the amount by which the value is advanced for each set of recalculations. For example, with a step rate of 10 as shown, the Delivery Head will be calculated in tens from the minumum of 10 to the maximum of 100 metres. If the step rate is changed to 20, the number will advance in twenties from the minimum to the maximum.

Change the step rate to a bigger figure to reduce the number of calculations carried out.

Change the minimum and maximum Delivery Head to those available at the site being calculated.

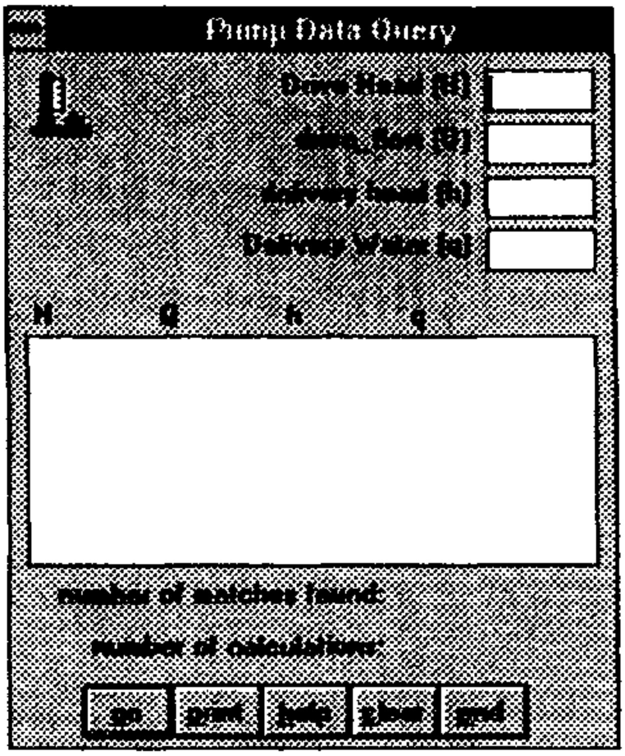
After setting the Step rates and the maximum and minimum entries for Drive Head (H), Drive Flow (Q) and Delivery Head (h) at a site, click on the "go" button to start making calculations.



The total number of calculations made is shown, and the name of the data file in which these calculations are stored is given

3.1 Looking at the results

When the calculations have been made, they are saved in the file called "pumpdata.txt" on your hard disk. You can look at this using a wordprocessor or other programme if you want to. To look at the calculations in this programme, and to select restricted ranges to look at, click on the "FIND" button to open the PUMP DATA QUERY window.



Asking the calculator a question

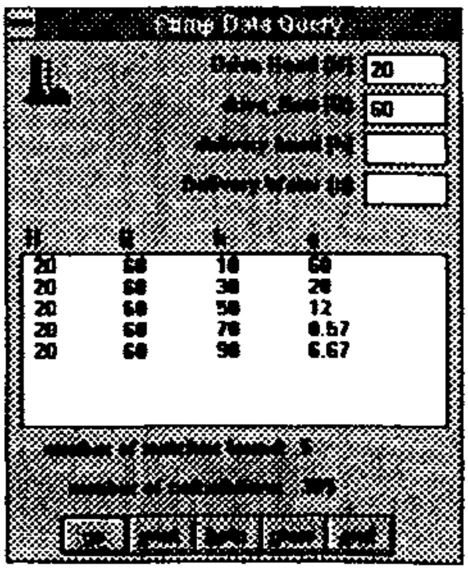
A Query is simply a question. You can ask to see the data for a restricted set of variables, or for all. To see all, just click on the GO button without putting anything in the boxes.

To see a restricted part of the calculations you have just made at the DTU Ram Pump Data Generator window, make at least one entry in the boxes alongside Drive Head, Drive Flow, Delivery Head and Delivery Flow.

For example, if you wanted to see how much drive flow would be needed at a site with a known drive head, delivery head and required delivery flow, type the known values in the boxes and click on the GO button.

EXAMPLE

In this example, a drive head of 20 metres was possible but the drive flow was restricted to 60 litres per minute. We wanted to know how much water could be pumped to a range of heights. The step rate for the delivery head had been given as 20 metres at the DTU Ram Pump Data Generator window, so only five calculations for delivery head were made for its range of between 10 and 100 metres.



Notice how it is possible to generate silly answers if you are not careful. The first calculation in the list shows a Drive Head of 20 metres and a Delivery Head of only 10 metres — gravity feed!

Printing the result

To print the information you have found, simply click on the PRINT button. Make sure that your printer is attached and turned on, with paper in it before trying to print anything.

It can be useful to print a range of calculations for a particular site before visiting it. For example, you might know that the site must deliver 10 litres a minute before the client will be interested, you might also know the efficiency of the pump to be installed, and the fact that there is plenty of drive water (120 litres a minute or more). You may also know that the delivery head is at least 50 metres. If you then find all possible combinations of drive Head and Delivery Heads of 50 metres or more that give a delivery flow of 10 litres a minute or more, this would be useful to have with you when assessing a site.

Asking another question

If you want to find the answer to another question, click on the CLEAR button to clear your first question and its answers from the window. Then type in the relevant values for your new question and click the mouse on the GO button.

Leaving the PUMPDATA programme

When you have finished making your calculations, click the mouse on the END button to close the DTU PUMPDATA programme.

4 Using the DTU programme DOSPUMP

Refer to section 1.2.1 To run the DOS DTU calculator programme - DOSPUMP to start the programme from your hard disk or from the floppy disk provided.

This is what appears on the DOSPUMP main screen.

```
DTU Ram Pump Calculator by ProData

Q=Drive Flow: q=Delivery Water: H=Drive Head: h=Delivery Head: E=Efficiency

Which variable to calculate: Q, q, H, h, E, ?=help, X to quit
```

Type which one of the five variables, Q, q, H, h, and E you want to calculate.

Be careful to type "Q","H" and "E" as upper case letters and "q" and "h" as lower case letters.

WINPUMP users will find that the DOSPUMP programme is a very similar calculator. Because it runs under DOS its appearance is different, but it does the same things as WINPUMP.

Whichever variable you select, a window will open asking you to enter a value for the other variables, one at a time. For example, if you type "q", the window shown below opens prompting you to give a value for E (the pump efficiency).

```
DTU Ram Pump Calculator by ProData

Q=Drive Flow: q=Delivery Water: H=Drive Head: h=Delivery Head: E=Efficiency

Which variable to calculate: Q, q, H, h, E, ?=help, X to quit

                    Delivery Water
                    enter value for E:
```

You are asked for each of the other four variables in turn, and when you have entered them (press [Return] after each entry), the programme displays the missing value lower on the screen. An example is shown below.

```
DTU Ram Pump Calculator by ProData

Q=Drive Flow: q=Delivery Water: H=Drive Head: h=Delivery Head: E=Efficiency

Which variable to calculate: Q, q, H, h, E, ?=help, X to quit

                    Delivery Water
                    enter value for E: 0.6
                    enter value for H: 5
                    enter value for Q: 90
                    enter value for h: 60

                    the value of q is  4.50

                      Again? (y)es or (n)o:
```

When you have entered all the variables, a window at the bottom of the screen a message prompts:

"Again? (y)es or (n)o:"

Type the letter Y to clear the screen and calculate a result for another set of variables.

Type the letter N to close DOSPUMP.

DTU

Ram Pump Programme

RAM PUMP SYSTEM CALCULATORS

TECHNICAL
10
RELEASE

DTU Technical Release TR10 *1996*

Cardboard Calculators for Designing Ram Pump Systems

1 INTRODUCTION

In the Development Technology Unit we have for many years been designing water systems that use hydraulic ram pumps. We have also trained water specialists from over 10 countries how to make the pumps and design systems to put them in. To design a good system, you have to do some calculations. We have found that most people do not enjoy using formulas or doing maths, so we have developed special cardboard 'calculators' that can be used instead of formulas. They give the same answers as the formulas in books but are much easier and quicker to use.

When we design a water system using ram pumps, we like to be able to know *before* we build it how much water it will deliver. This is called the "delivery flow". The first DTU calculator is the *Ram Pump System Design Calculator* and is meant mainly for working out such delivery flows.

The second cardboard calculator is called the *Friction Headloss Calculator*. We use it to help select the right sizes for the different pipes in a ram pump system. It can also be used for choosing the pipe sizes in a gravity-feed water system.

In this Technical Release we describe how to make the two calculators and how to use them.

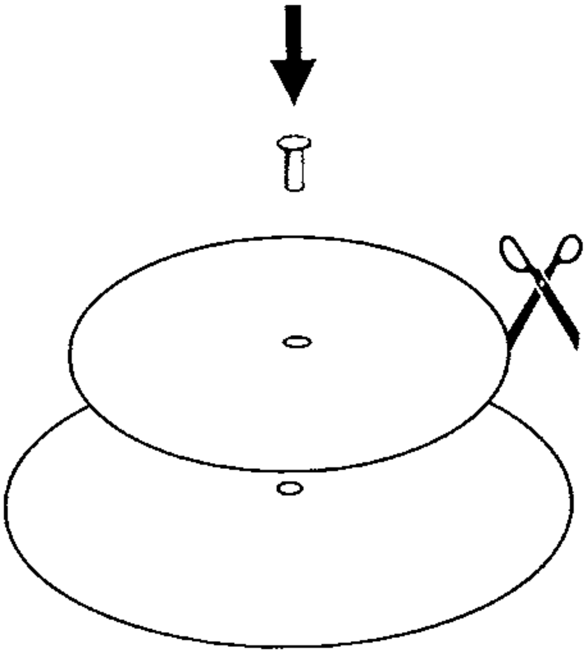
The last three pages of this Technical Release are single-sided master pages to be photo-copied each time you make a calculator. We suggest that the first photo-copy you make should be attached to this Release in case the copy masters get separated and lost.

Development Technology Unit
Department of Engineering
University of Warwick
Coventry CV4 7AL UK
Tel +44/0 1203 523523 ext 2339
(Director) 523122 Fax 418922
email dtu@eng.warwick.ac.uk

2 MAKING THE CALCULATORS

Start by photo-copying the calculator parts printed on the 3 pages at the end of this Release. The thicker the paper or card you can use for these copies, the better; card as thick as 180 gpm can be fed through some slow photo-copiers. The *Delivery Flow Calculator* has 2 parts, both on the same page. The *Friction Headloss Calculator* has 3 parts, spread over the two following pages.

Carefully cut out the parts of the *Delivery Flow Calculator.* You will be assembling them by pushing a rivet through the centre holes and turning the ends over, so the cut-out circles for these centre holes should be the same size as the rivet you will be using. It is important to make these holes round and put them in exactly the right place on the disc.



The rivet can be of plastic or soft metal. We have successfully made rivets from short strips cut from thick plastic piping, rolled on sandpaper to make them round and then turned over at the ends with a hot iron. You could also use a short plastic or steel bolt with washers and two nuts: the nuts have to be tightened against each other so that they hold the discs not quite tight.The holes can be reinforced with self adhesive "ring reinforcements" sold in some stationery shops to strengthen the holes on paper sheets held in ring-binders.

If the calculator is always to be used in one office, the 2 discs can be held by a drawing pin (through their exact centres) onto a notice board. The pin must be loose enough to allow the discs to rotate freely.

If the calculators are to be carried around or taken out of doors, they will soon get torn or dirty. If at all possible you should cover each one on both sides with transparent plastic. Clear adhesive plastic sheet is sometimes sold in stationery shops for use to protect schoolbooks. If you place any hole-strengthener under the plastic, this will help prevent it falling off. It also helps if you can make a waterproof pocket to carry the calculators in.

The *Friction Headloss Calculator* is made in the same way as the *Delivery Flow Calculator.* except that there are now 3 parts free to rotate on the same rivet or pin.

3 USING THE 'RAM PUMP SYSTEM DESIGN CALCULATOR'

The *Ram Pump System Design Calculator* has two discs and a swinging arm. It carries four scales

Feed head *H*	Range 1.5 to 30 meters
Delivery head *h*	Range 10 to 120 meters
Feed flow *Q*	Range 10 to 800 litres/minute
Delivery flow *q*	Range 0.5 to 80 litres/minute

Check that you can find each of these scales.

The last scale is in three parts called "efficiency = 50%", "efficiency = 60%" and "efficiency = 70%". If you do not know what your system efficiency might be, use the "60%" part. If you think it might be rather low, use the "50%" part and if it might be very good use the "70%" part.

The calculator is for calculating the flow from a complete *system*, not just from a single pump. If your system has several pumps working side by side, you should use as *Q* the feed flow to the whole system, not the drive flow to just one of the pumps. The calculator will then tell you the delivery flow from the whole system and not just the output of one pump. If your system does have only one pump in it, there is no problem; the feed flow you use will all go through that one pump.

To learn how to use the calculator we suggest you practice with the following example.

Suppose you have a system in which
feed head = 4 meters,
delivery head = 40 meters,
feed flow = 50 litres per minute,
efficiency is unknown so we use 60%,
then the calculator should show a delivery flow of 3 litres per minute.

Here are the two steps.

Step 1 Hold the outside disk and turn the inside disk until 50 litres per minute on scale *Q* is exactly in line with 4 meters on scale *H*.

Step 2 Pinch the two discs together with your fingers so that one cannot slip past the other and then turn both disks together until the *q* scale is at the top. Find 40 meters on scale *h* and read the flow above it on the "efficiency = 50%" part of scale *q*. You should get close to 3 litres per minute.

The calculator is not super-accurate. It will give you an answer that is "close enough". If you followed the example above and got an answer that was not close to 3 litres/min it might be that

- you let the disks slip past each other,
- you used the wrong scales,
- the holes in the middle of your disks have become badly worn and the disks are very loose.

4 USING THE 'FRICTION HEADLOSS CALCULATOR'

When water flows through a pipe there is some friction between the water and the pipe walls. This causes something called "friction headloss" which is measured in meters. A pipe with a small diameter has a bigger headloss than a larger pipe for the same flow of water. A big headloss usually means that the system delivers less water than it should.

The *Friction Headloss Calculator* has two disks and a swinging arm. It carries four scales which you might look for:

Length of pipe	Range 2 to 3000 meters
Flow through pipe	Range 1 to 500 litres per minute
Bore of pipe ("Bore" is the same as "Inside diameter")	Range 16mm to 200mm
Headloss	Range 10cm to 100 meters

and the two small arrows on the swinging arm called "steel" and "plastic". At the top of the swinging arm is a big arrow labelled "flow through pipe".

You can use this headloss calculator to help you choose the best size for the delivery pipe, the feed pipe and the drive pipes in a ram pump system. You can also use it for pipes in gravity feed systems. We will now describe each of these four uses.

Use 1: Delivery pipes

When you come to choose the size of a delivery pipe, you will usually already know how long it is, how high it rises and how much water is to flow through it.

Suppose for example you know that at a particular site:

pipe length = 500 meters,

flow through the pipe = 10 litres per minute and the pipe is plastic,

the delivery height h = 60 meters.

Standard pipe sizes (*outside* diameters) are 20 mm, 25 mm, 32 mm and 40 mm. For medium pressure pipes, their bores (their *inside* diameters) will be about 2 mm smaller - typically 18 mm, 23 mm, 30 mm and 37 mm. Your problem is to decide which size is right.

Suppose you think that the 30 mm bore pipe might be suitable. The calculator lets you find out if it is. The calculation goes as follows:

Step 1 Hold the outer disk and rotate the inner disk until 500 m on the Length scale is touching 30 mm on the Bore scale.

Step 2 Pinching the two disks together so that they do not slip, move the swinging arm until the big arrow at its top is touching 10 litres per minute on the Flow scale.

Step 3 Read the headloss scale underneath the "plastic" arrow. You should find about 1.6 m.

You now have to decide whether this headloss is OK. It should normally be between one fiftieth and one tenth of the delivery head h. If headloss is more than $h/10$ your system will be quite inefficient, so you should try a larger pipe. If it is less than $h/50$ you are using a delivery pipe that is too big: you could save money by trying a smaller pipe. In our example the headloss *is* between $h/50$ (1.2 m) and $h/10$ (6 m), so our pipe size of 30mm is OK.

Use 2: Feed pipes

Your ram pump system may have a feed pipe connecting a stream or spring to a drive tank from which drive pipes lead to each pump. The calculation for feed pipes is just like that for delivery pipes except that the flow is now the feed flow to the drive tank, and of course the length and diameter we use are those of the feed pipe. Feed pipes are usually plastic, so use the "plastic" arrow on the swinging arm. Common pipe sizes for feed pipes are 50 mm, 63mm, 75mm, 90 mm and 110mm outside diameter. The bores of these sizes for low-pressure pipe are about 47 mm, 59 mm, 72 mm, 86 mm and 105 mm. So first decide which bore you should try for the feedpipe, then use the calculator to work out its headloss.

When you have found the headloss, you should compare it with the *feed* head *H*). If it is over H/10, the feed pipe should be larger. If it less than *H*/50 the feed pipe could be smaller.

Use 3: Drive pipes

Each drive pipe carries the drive flow to a single pump: this is usually equal to the feed flow *Q* divided by the number of pumps running side by side. As the drive flow is not steady like the feed flow, but is constantly starting and stopping, the head loss in a drive pipe is higher than for other types of pipe. The *Friction Headloss Calculator* gives the right head loss for a steady flow. For a drive pipe with its unsteady flow we have to multiply the steady flow headloss by 2.

So choose a likely pipe size and for it calculate the steady flow headloss in the same way as for the delivery pipe, but using the length, bore, material and flow that belong to *one* drive pipe. Next multiply the calculator's answer by 2 to get the true headloss. Lastly check whether this true headloss lies between *H*/50 and *H*/10. If it lies outside this band, you should think of changing the drivepipe size. Notice that, as with the feed pipe headloss, we are comparing the drive pipe headloss with the *feed* head *H*.

Use 4: Pipes in gravity feed systems

In a gravity feed system, pipes are laid so that they slope downwards. The drop in height between where water enters a pipe and where it leaves it is what pulls the water through. For any one size of pipe, a bigger drop in height gives a bigger flow. We can use the *Friction Headloss Calculator* to work out what the flow will be. This is because in a gravity feed system, the friction headloss is exactly *equal* to the drop in height.

Let us take as an example a 1" galvanised iron pipe, whose length is 2 kilometres and which drops by 100 meters from one end to the other. We can now work out the flow through the pipe. The calculation goes as follows:

Step 1 Hold the outer disk still and turn the inner disk until 2 km on the length scale touches 26 mm on the bore scale. (A 1" GI pipe has a bore of about 26 mm)

Step 2 Keeping the two disks clamped together in your fingers, move the swinging arm until the "steel" arrow is touching 100 m on the Headloss scale.

Step 3 Read the flow opposite the big arrow on the Flow scale. You should get about litres/min.

If the flow is more than you wanted, try again with a smaller pipe. If the flow is too small, try a bigger pipe. Of course the flow through the pipe cannot be higher than the flow available at the entry to the pipe. So making the pipe bigger will only increase the flow if it is the pipe, and not the source, that is limiting it.

5 NOTE FOR ENGINEERS

The *Ram Pump System Design Calculator* is based on the simple formula:

delivery flow = efficiency x feed flow x feed head / delivery head ($q = Eff \times Q \times H/h$).

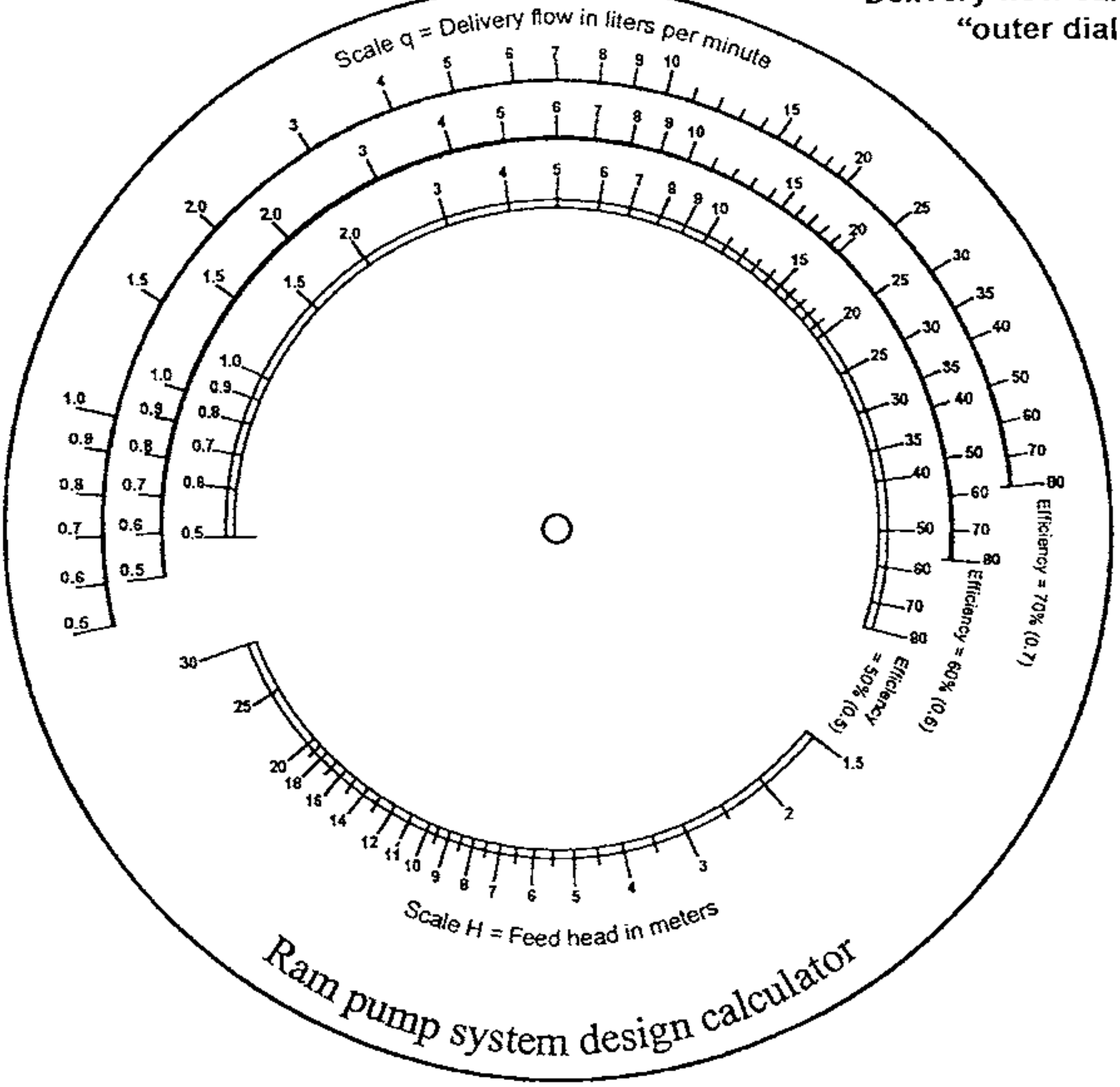
Most well-designed ram pump systems have an efficiency of between 0.5 and 0.7. Some pump manufacturers issue design tables based on the assumption that system efficiency is always 0.6. However you may meet systems that use undersize pipes or ram pumps operated at the very top of their drive-flow range or their delivery head range, where system efficiency is as low as 0.3.

The *Friction Headloss Calculator* is based on an approximate formula because a more exact formula would be too complex to convert into a slide rule (which is what these cardboard calculators are). The flow in pipes in practical water systems is always highly turbulent. In the range of flows of interest to system designers it is safe to use the approximate turbulent flow formula:

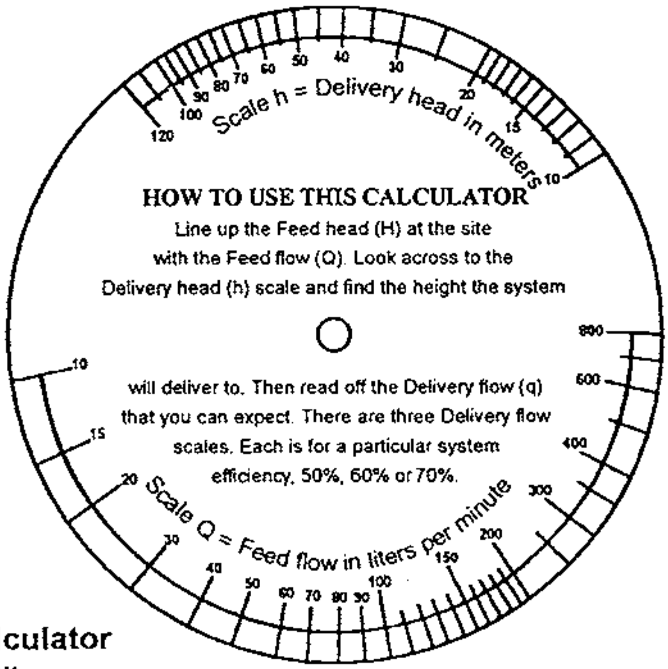
friction headloss = K x pipelength x flow $^{1.85}$ / internal diameter $^{4.9}$

If headloss and pipelength are measured in meters, diameter in mm and flow in litres per minute, then K = 650 for smooth plastic pipes and K = 800 for steel or GI pipes.

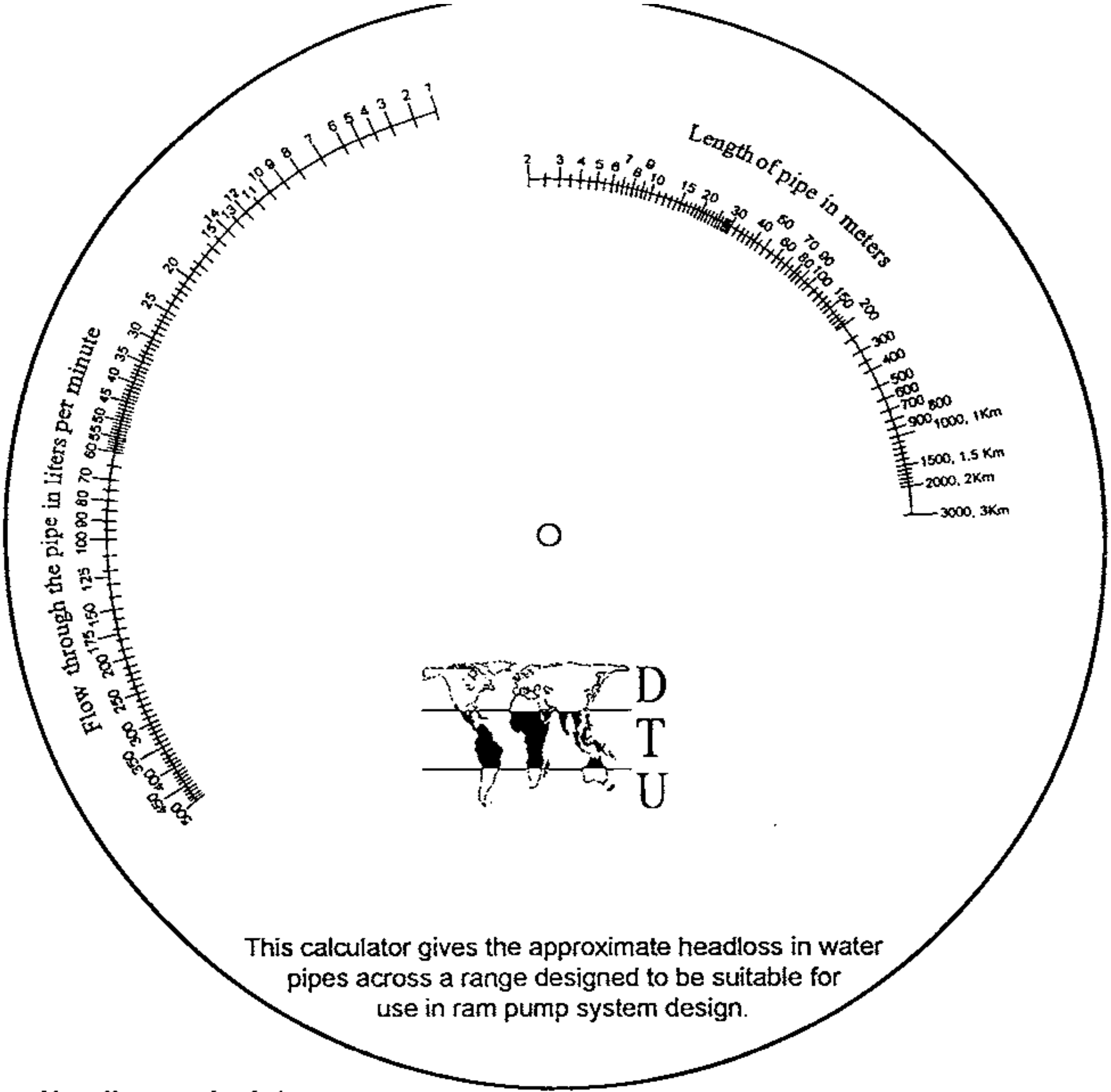
Headloss formulas are not very precise. The headloss may differ by +/-25% of the value predicted by the *Friction Headloss Calculator*. If you are using the calculator to predict flow in a pipe, you can expect to get within +/-10% of the actual value.



Delivery flow calculator
"outer dial"

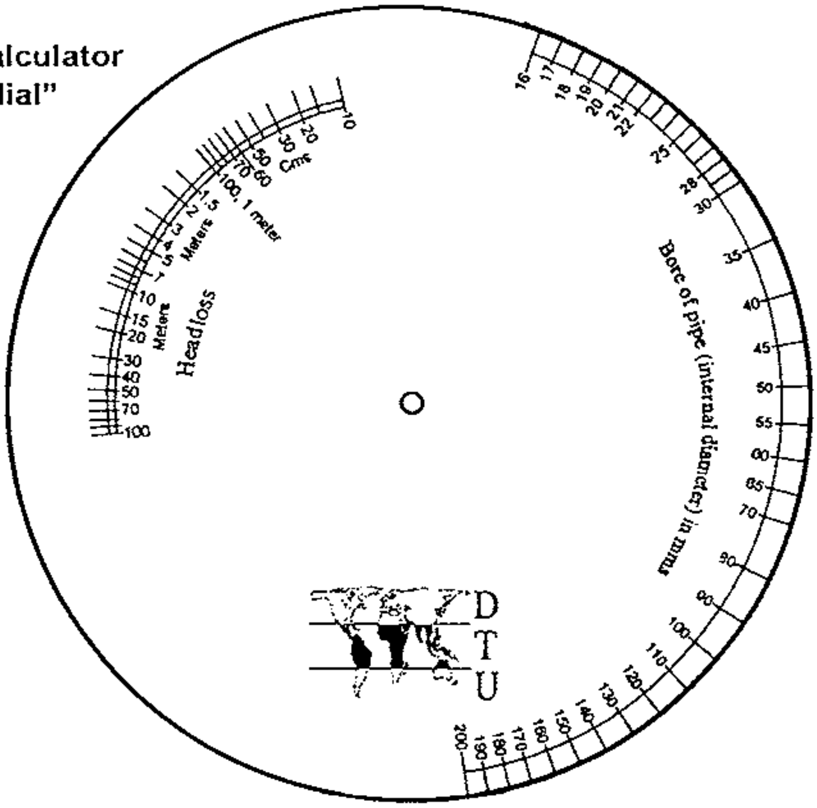


Delivery flow calculator
"inner dial"

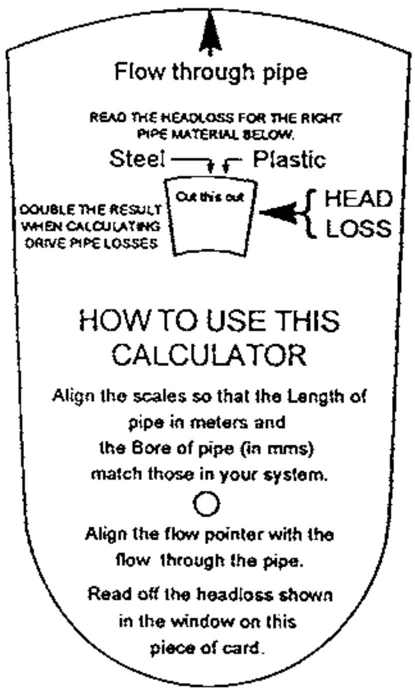


Headloss calculator
"outer dial"

Headloss calculator
"inner dial"



Headloss calculator
"swinging arm"



DTU

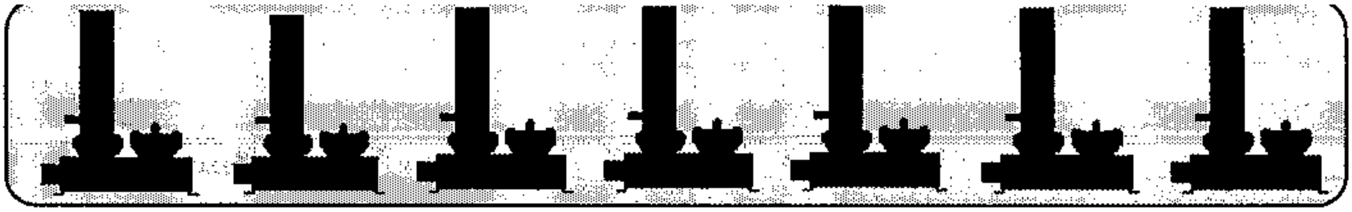
Ram Pump Programme

TECHNICAL

11

RELEASE

DTU S1 PUMP



DTU S1

hydraulic ram pump

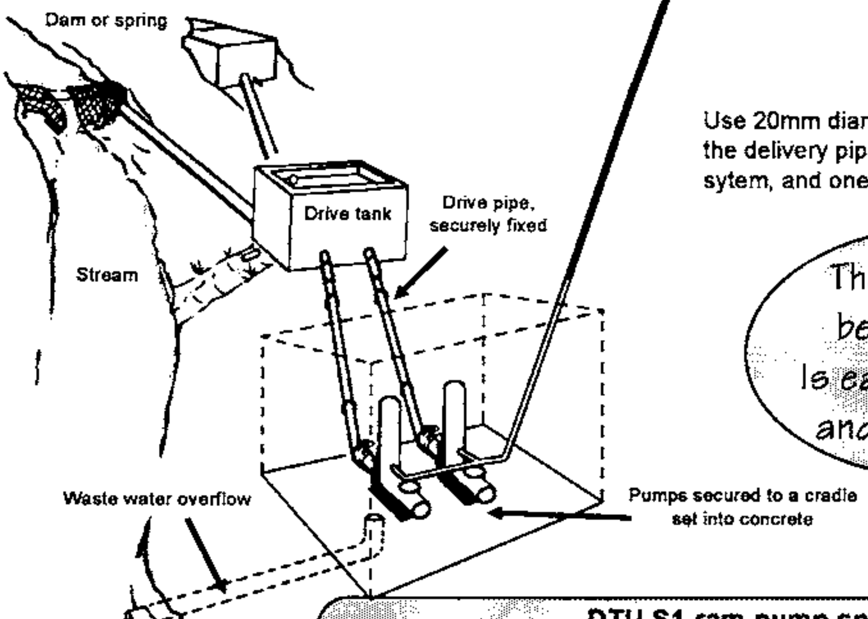
The name "S1" stands for a Steel pump with a drive pipe up to 1" in diameter.

A ram pump is powered by falling water. Water from a stream or spring is diverted and dropped through a drive pipe into the pump. The power of the falling water is used to pump some of the water where it is wanted. The amount of power in the falling water limits how high you can pump, and how much water you can pump. Generally, the more water you drop and the further you let it fall, the more power there will be.

The DTU S1 hydraulic ram pump is a steel machine, using a ¾" or 1" diameter galvanised drive pipe, that can lift water up to a height of 80 meters. It was designed for water supply to small groups of houses from minor sources of water such as springs and small streams. It is being used successfully in many African countries.
The pump has been designed to be made in small workshops with welding equipment and a pillar drill. A lathe can be useful but is not essential.
In areas where the water source flow varies greatly during the year, more than one pump can be installed, all sharing the same delivery pipe as shown in the drawing below.

Distribution system, for domestic use. A tank is always recommended.

Delivery pipe, rising all the way along its length (no ups and downs). The pipe should be buried where possible and protected if it has to be above ground.



Use 20mm diameter plastic pressure pipe for the delivery pipe if there is one pump in the sytem, and one size larger if there are two.

The DTU S1 can be locally made is easy to maintain and cheap to run!

DTU S1 ram pump specifications

The normal specifications of the DTU S1 ram pump are given here.
Sometimes you can operate pumps outside these limits, but they may not work well.

drive head range	—	2 to 15 meters
drive flow range	—	20 to 60 liters a minute
drive pipe material	—	Galvanised iron
drive pipe diameter	—	¾" for flows from 20 to 35 liters a minute
drive pipe diameter	—	1" for flows from 30 to 60 liters a minue
delivery head range	—	up to 80 meters
typical delivery range	—	0.5 to 10 liters a minute
delivery pipe diameter	—	20mm

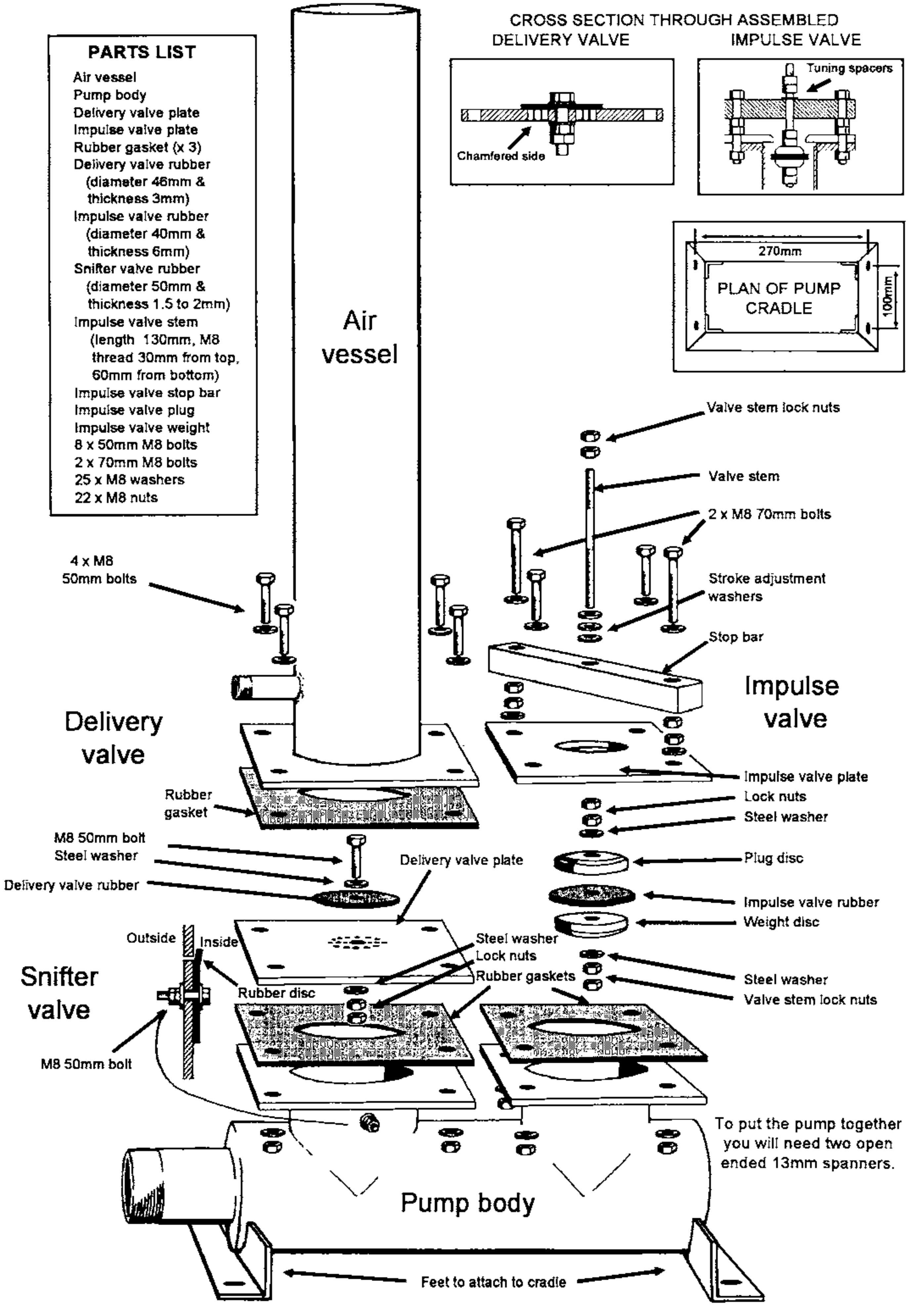
TECHNICAL
11
RELEASE

DTU S1 PUMP: USER INSTRUCTIONS

AN EXPLODED VIEW OF THE DTU S1 PUMP

PARTS LIST

- Air vessel
- Pump body
- Delivery valve plate
- Impulse valve plate
- Rubber gasket (x 3)
- Delivery valve rubber (diameter 46mm & thickness 3mm)
- Impulse valve rubber (diameter 40mm & thickness 6mm)
- Snifter valve rubber (diameter 50mm & thickness 1.5 to 2mm)
- Impulse valve stem (length 130mm, M8 thread 30mm from top, 60mm from bottom)
- Impulse valve stop bar
- Impulse valve plug
- Impulse valve weight
- 8 x 50mm M8 bolts
- 2 x 70mm M8 bolts
- 25 x M8 washers
- 22 x M8 nuts



To put the pump together you will need two open ended 13mm spanners.

Pump repair

If the pump stops or starts delivering less water than usual, it may require adjustment or repair.

Look at the pump and if there is no obvious fault start it again if you can. Watch the pump and listen for irregular pumping or unusual noises. A worn impulse valve, for example, is usually obvious because water squirts through when the valve is closed. Some parts of your ram pump may need occasional replacement. the frequency of this will depend on how hard the pump is working and on the cleanliness of the drive water.

Tools you will need:

2 x 13mm ring/open end spanners to disassemble and assemble the pump
2 x Adjustable wrenches - to loosen a union joint on the delivery pipe (if fitted)

Taking the pump apart

Depending on the fault it may be necessary to disassemble the impulse valve and/or the air vessel. Before attempting to take apart the pump:

1. Make sure that the drive pipe valve is closed and the impulse valve is open. This will allow you to work on the impulse valve ONLY.
2. Depressurise the air vessel.

WARNING - Before attempting to remove the air vessel, always release the pressure in it slowly. An ideal system will have a gate valve or one-way valve and a union fitted between the air vessel and the bottom of the delivery pipe and the optional bleed screw fitted to the air vessel. With the pump stopped, close the gate valve in the delivery pipe to stop it draining back. If a one-way valve is fitted it will close automatically. Then loosen the bleed screw to release the pressure in the air vessel. If none of the above are fitted, the only other way to release the pressure in the air vessel is to loosen each of the air vessel flange bolts one turn at a time until the water and air escapes through the join at the flange. You will certainly get wet this way.

Checks

1. Check the delivery valve rubber for wear and blockage of the valve holes.
2. Check that the snifter valve is in good condition.
3. Remove the impulse valve and check the impulse valve rubber. Check the nuts on the valve stem and check for excessive wear of the stem. Replace things if necessary.
4. Check the pump body is firmly bolted down, then reassemble the pump, ensuring that all bolts are greased.

Putting the pump back together

Assembly of the pump is shown in the exploded view drawing, but the following important points need to be kept in mind:

1. Assembling the delivery valve
   Put together the delivery valve plate, the rubber and the bolt. Make sure the side of the plate with the chamfered holes is on the opposite side to the rubber, and that the rubber is on top.
   Screw on the first nut until it is finger tight and then undo it by one turn. Care must be taken not to overtighten the bolt and nuts as this will affect the performance of the valve. Next, screw on the other nut and tighten it up against the first. Use the spanners to tighten them firmly together. This will lock them together, and also allow a small up-and-down movement of the bolt and rubber.
2. Assembling the snifter valve
   Put the 'shaped' bolt and washer together, feed the bolt through the valve rubber, then push this through the pump body. Make sure that the shaped curve of the bolt head and washer align with the curvature of the body.
   Screw on the first nut until it is only finger tight. If the nut is on too tight the rubber will curl away from the pump body and will need to be slackened off slightly. Then screw on the second nut and tighten the two nuts firmly together using the spanners. Then check that the rubber has not distorted. If it has, slacken the nuts half a turn and tighten the outer one again.
3. Assembling the air vessel and delivery valve
   Align the delivery valve, air vessel, pump body and rubber gasket mounting holes and feed through the bolts. Make sure the delivery valve is the correct way up (the valve rubber facing upwards) and then tighten the nuts by hand. Use the spanners to tighten each nut and bolt a little at a time, working around the flange. This will draw the assembly together evenly.
4. Assembling the impulse valve
   The first parts to assemble are the valve stem, discs and rubber. Screw two nuts onto the longer threaded end of the stem up to the end of the thread. Push a steel washer on up to the nuts. Follow this with the valve plug disc, with the chamfered side towards the nut. Slide the valve rubber up against this, then the weight disk with the chamfered side facing away from the rubber. Follow this with another steel washer. Screw a nut up to them until it is finger tight. Thread on another nut and use the spanners to tighten the nuts together. This part of the assembly is sometimes known as the valve plug.
   Hold the impulse valve plate and the valve plug together, with the chamfered side of the plate opposite to the side against which the valve rubber presses. Slide the stop bar onto the top of the stem and thread a nut loosely on the stem.
   Push the two longer bolts through the ends of the stop bar and thread two nuts onto each. Use spanners to lock the nuts tightly. Align the valve assembly, pump body and rubber gasket and feed through the flange bolts.
   Thread on the four washers and nuts by hand, then use the spanners to tighten the two shorter bolts that hold down the valve plate. Again care must be taken to ensure that these nuts are tightened evenly. The next step is to make sure the closed valve plug is centred in the valve plate hole before tightening down the two remaining nuts that secure the stop bar. To check the alignment, open and close the valve manually turning the valve plug to make sure it does not catch on the hole in the valve plate.

Now you only need to set the stroke length of the valve for the pump to be ready for use.

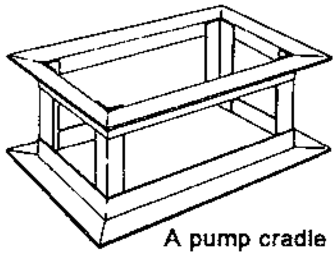
Spare parts to keep on the site

- impulse, delivery and snifter valve rubbers
- an impulse valve stem
- a few spare M8 nuts, bolts and washers

The Development Technology Unit (DTU)
Department of Engineering
University of Warwick,
Coventry CV4 7AL UK
Tel: 44 (0)1203 523122
FAX: 44 (0)1203 418922

Installation notes

The DTU S1 pump should be installed in a properly designed system. To prevent vibration causing breakages, it should be firmly bolted to a steel frame (called a pump cradle) that is half buried in a concrete base. The cradle is usually made from 40 x 40mm angle iron and will vary in size depending on the number of pumps installed. Hole locations for just one pump are shown on the previous page.
All pipes in the system should be supported firmly, and buried where possible. The drive and delivery tanks should be constructed on good foundations by experienced tradesmen. Pipe joints to the drive tank should allow the pipes to move slightly without damaging the tank walls or leaking badly.



A pump cradle

Starting and stopping the ram pump

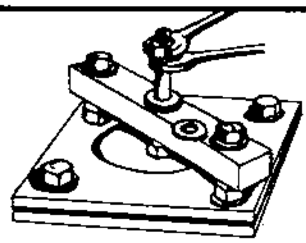
Although ram pumps often start very easily, they can be awkward the first time they are run.
To start the pump:

1. Make sure that any valve fitted on the delivery pipe is open and then open the drive pipe valve. Water will flow out of the open impulse valve until it suddenly shuts. If it reopens automatically, the pump should continue to run on its own. If it does not, you must prime the delivery system as described in Step 2 alongside.
2. Push down on the top of the impulse valve stem with your foot to reopen it (wear strong boots). Again, water will flow out of the open impulse valve until it suddenly shuts, then push down immediately to re-open the valve. Keep helping the valve to re-open until it will do so by itself.

To stop the pump, hold the impulse valve stem up to close it or shut the valve at the bottom of the drive pipe.

Tuning for best performance

The DTU S1 can be tuned to adjust performance. This is done by changing the up and down movement of the impulse valve, which is normally set to about 12mm. Tuning is usually done to achieve either the maximum delivery flow or the most efficient use of the drive water available.



Maximum delivery

When there is plenty of drive water available, the pump can be tuned to deliver as much water as possible. To do this, remove all washers from the impulse valve stem so that the valve has as much up and down movement as possible.

WARNING: - this also puts the pump parts under greater stress and makes them wear more quickly.

Low drive flow

If the pump uses more drive flow than is available it will soon stop. If this happens it must be tuned to use less. The impulse valve should be tuned down to use 90-95% of the water available from the source. To tune the pump down, add washers onto the impulse valve stem so that the valve has less up and down movement. The shorter the stroke, the smaller the amount of drive flow needed, and the less water is delivered. The minimum stroke length is about 7mm.

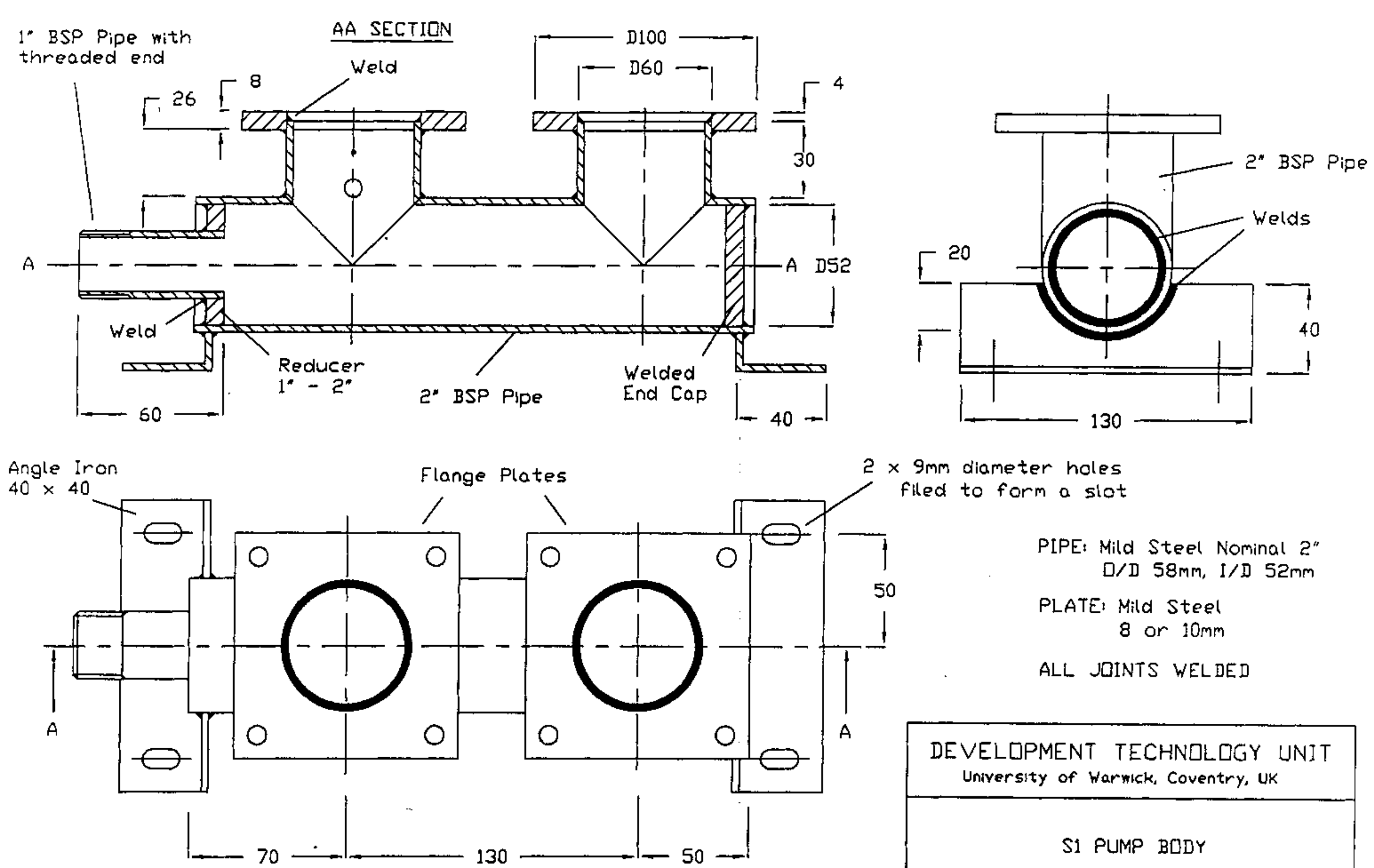
Routine maintenance

While the pump is running normally, a visit should be made once a week to check that bolts are tight and that there are no leaks. Once a month an inspection of the whole system should be carried out. It is also recommended that a log book is kept to record the checks and repairs that have been made.

Monthly maintenance check list (without stopping the pump):

1. Inspect all the joints to check for leaks.
2. Check if there is sufficient air in the air vessel. This can be done by listening carefully to the pump. If there is insufficient air in the air vessel, the pump will be much louder than usual. This means that the snifter valve is probably blocked and will need to be cleared.
3. Clean any filters installed in the system.
4. Remove excess silt or debree from tanks or from behind the intake dam or weir if necessary.
5. Walk along all pipes looking for damage. Also, inspect the tanks for leaks, particularly at pipe joints.

The following Technical release contains more information about making the DTU S1 pump
TR 11b DTU S1 pump drawings
Similar information is available for the other DTU pumps.



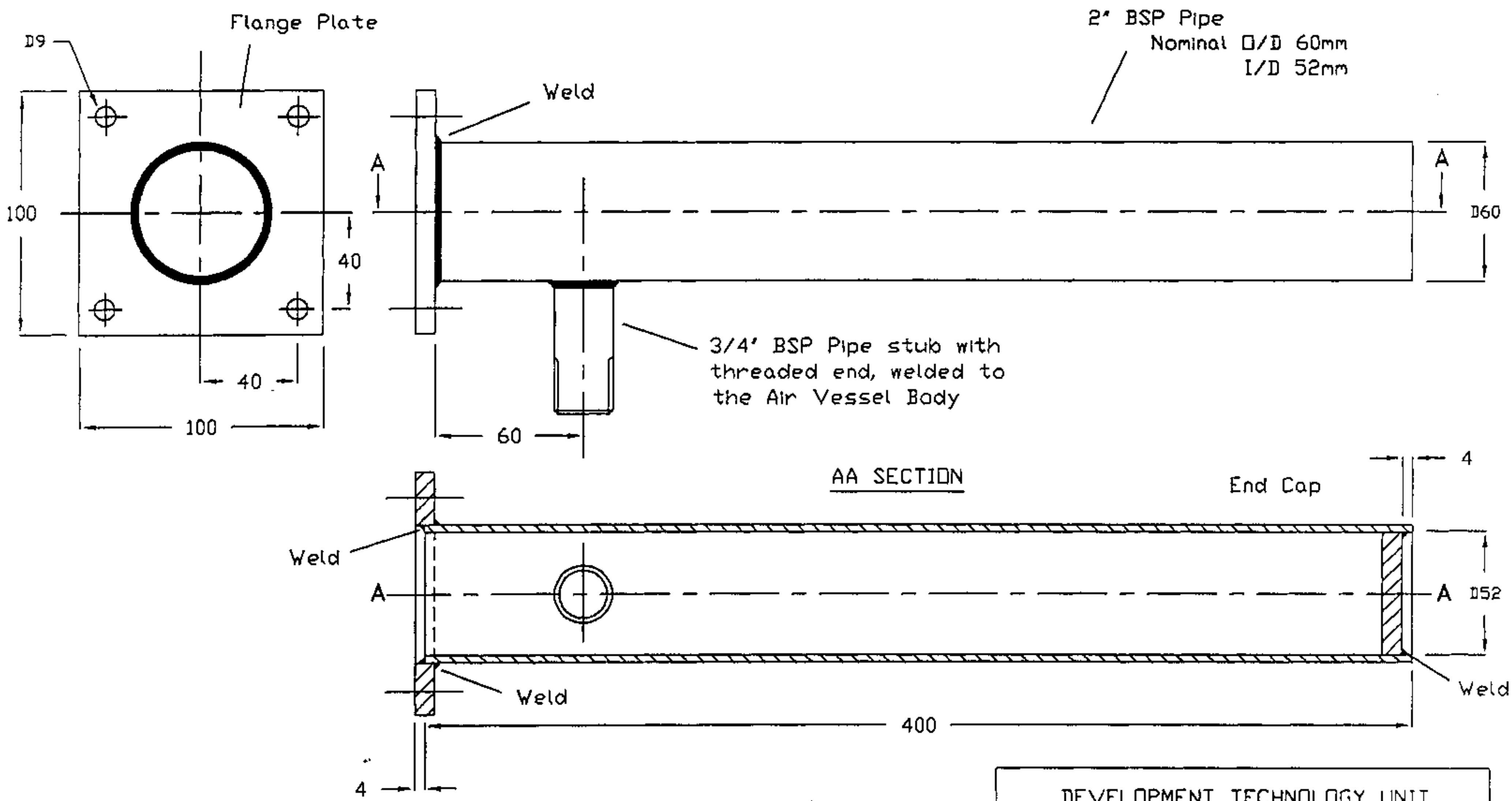
PIPE: Mild Steel Nominal 2"
O/D 58mm, I/D 52mm

PLATE: Mild Steel
8 or 10mm

ALL JOINTS WELDED

ALL DIMENSIONS IN mm

DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
S1 PUMP BODY
Drawing number 1 of 7 NOT TO SCALE



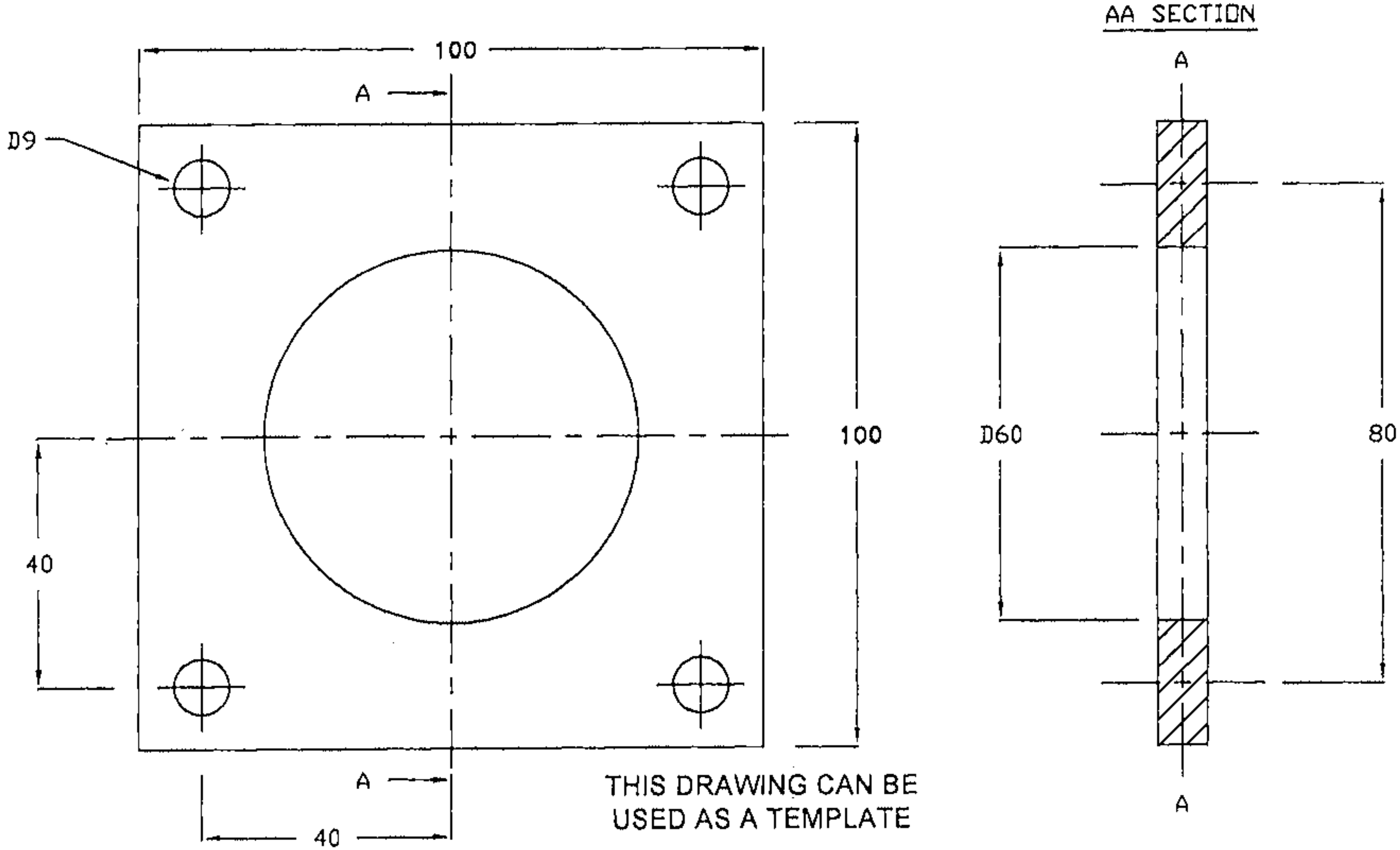
NOTE

The Flange Plate & End Cap are 8 to 10mm in thickness

To make the Flange Plate See the separate Drawing/Template

ALL DIMENSIONS IN mm

DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
S1 AIR VESSEL Drawing number 2 of 7 NOT TO SCALE



THIS DRAWING CAN BE USED AS A TEMPLATE

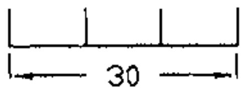
NOTE

Flange Plate thickness is 8 to 10mm

The internal diameter of the Flange Plate (60mm) may need to be different depending on the outside diameter of the steel pipe used for the pump body.

ALL DIMENSIONS IN mm

Measurement Check



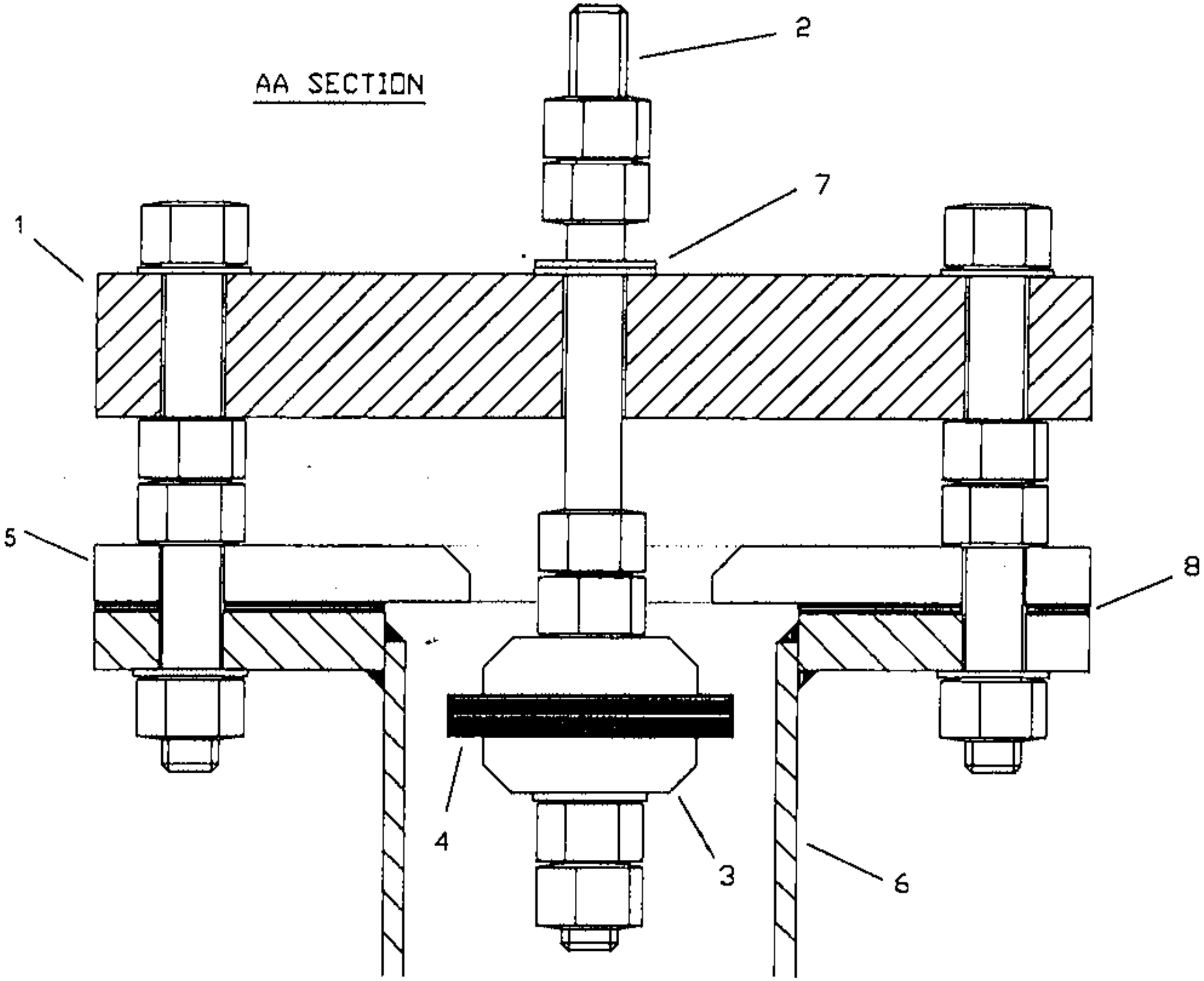
DEVELOPMENT TECHNOLOGY UNIT
University of Warwick, Coventry, UK

S1 FLANGE PLATE

Drawing number 3 of 7

SCALE 1:1

AA SECTION

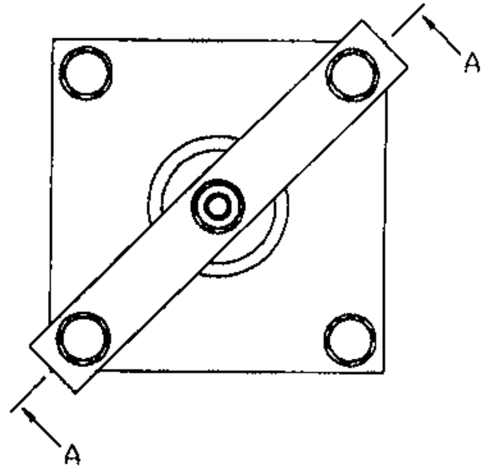


NOTE

The Rubber Disc (4) is 40mm in diameter and 6mm thick DO NOT use a diameter larger than this as the flow through the valve will be restricted

The Rubber Gasket (8) can be 1.5 to 3mm thick and is cut to match the Pump Body Flange

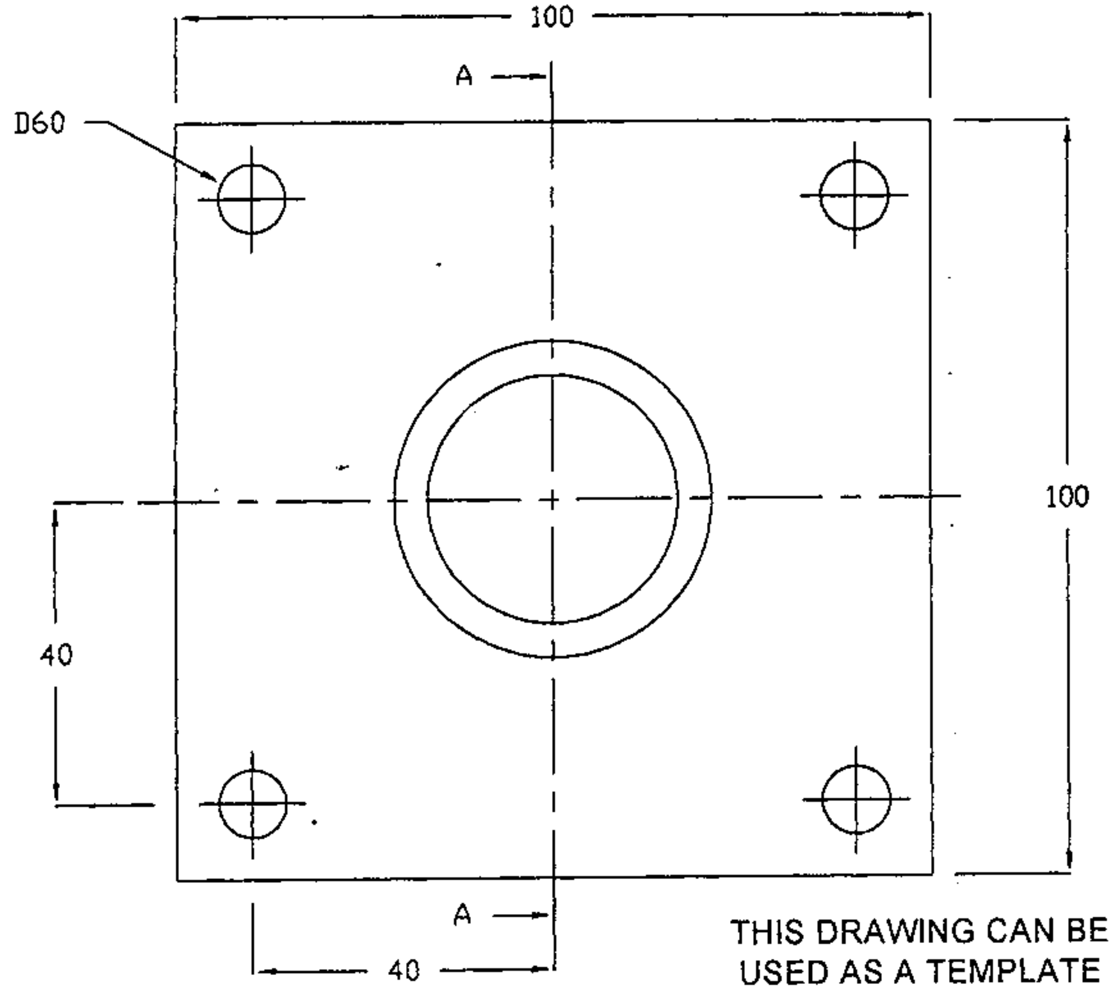
IMPULSE VALVE TOP VIEW



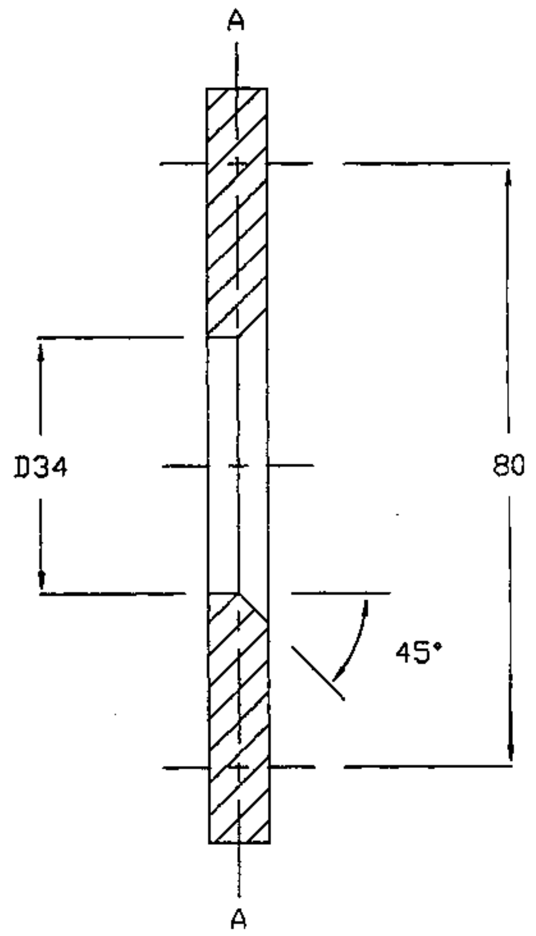
PARTS LIST

1 - Cross-bar
2 - Valve Stem
3 - Valve Disc (x2)
4 - Rubber Disc
5 - Valve Plate
6 - Pump Body
7 - Stroke Adjustment Washers
8 - Rubber Gasket
9 - M8 Bolts, Nuts and Washers
Bolts: M8 x 70mm (2 of)
M8 x 50mm (2 of)

DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
S1 IMPULSE VALVE ASSEMBLY Drawing number 4 of 7 AA SECTION - SCALE 1:1



THIS DRAWING CAN BE USED AS A TEMPLATE

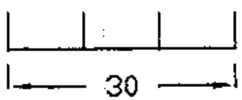


NOTE

The impulse valve plate thickness is 8 to 10mm.
It is important to ensure that the Impulse Valve Plate and the Impulse Valve Discs are of the same thickness.

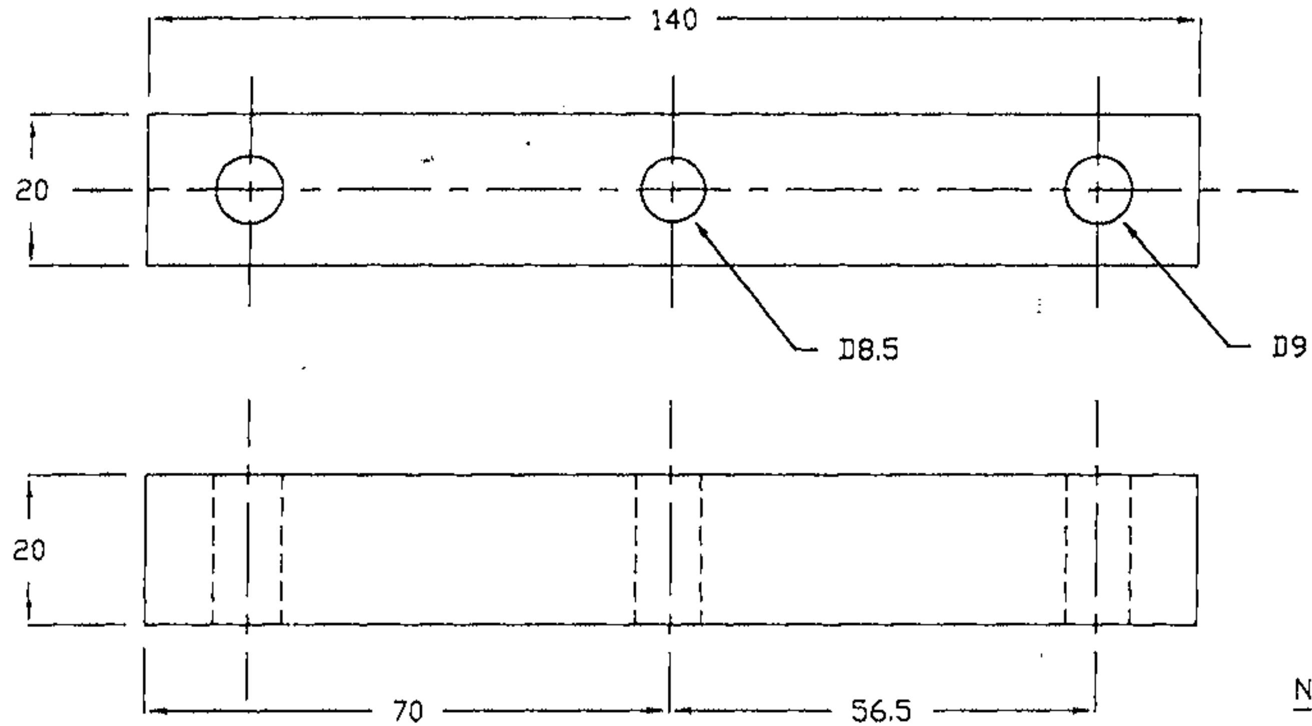
ALL DIMENSIONS IN mm

Measurement Check



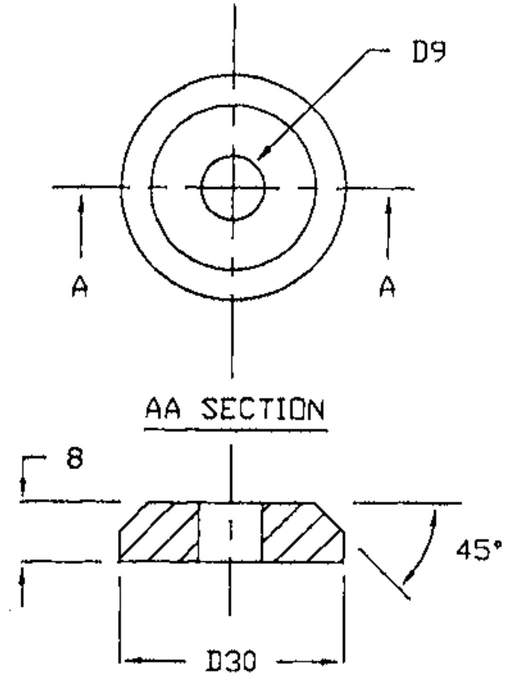
DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
S1 IMPULSE VALVE PLATE Drawing number 5 of 7 SCALE 1:1

IMPULSE VALVE CROSS-BAR



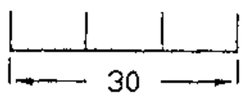
NOTE
Cross-bar is mild steel

IMPULSE VALVE DISCS



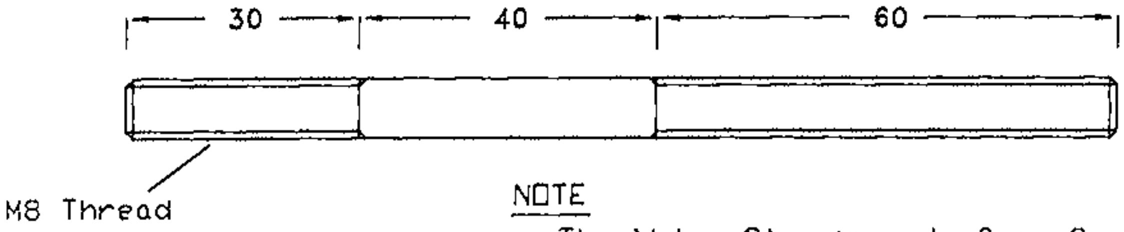
NOTE
2 Impulse Valve Discs are required.
Discs are of mild steel

Measurement Check



IMPULSE VALVE STEM

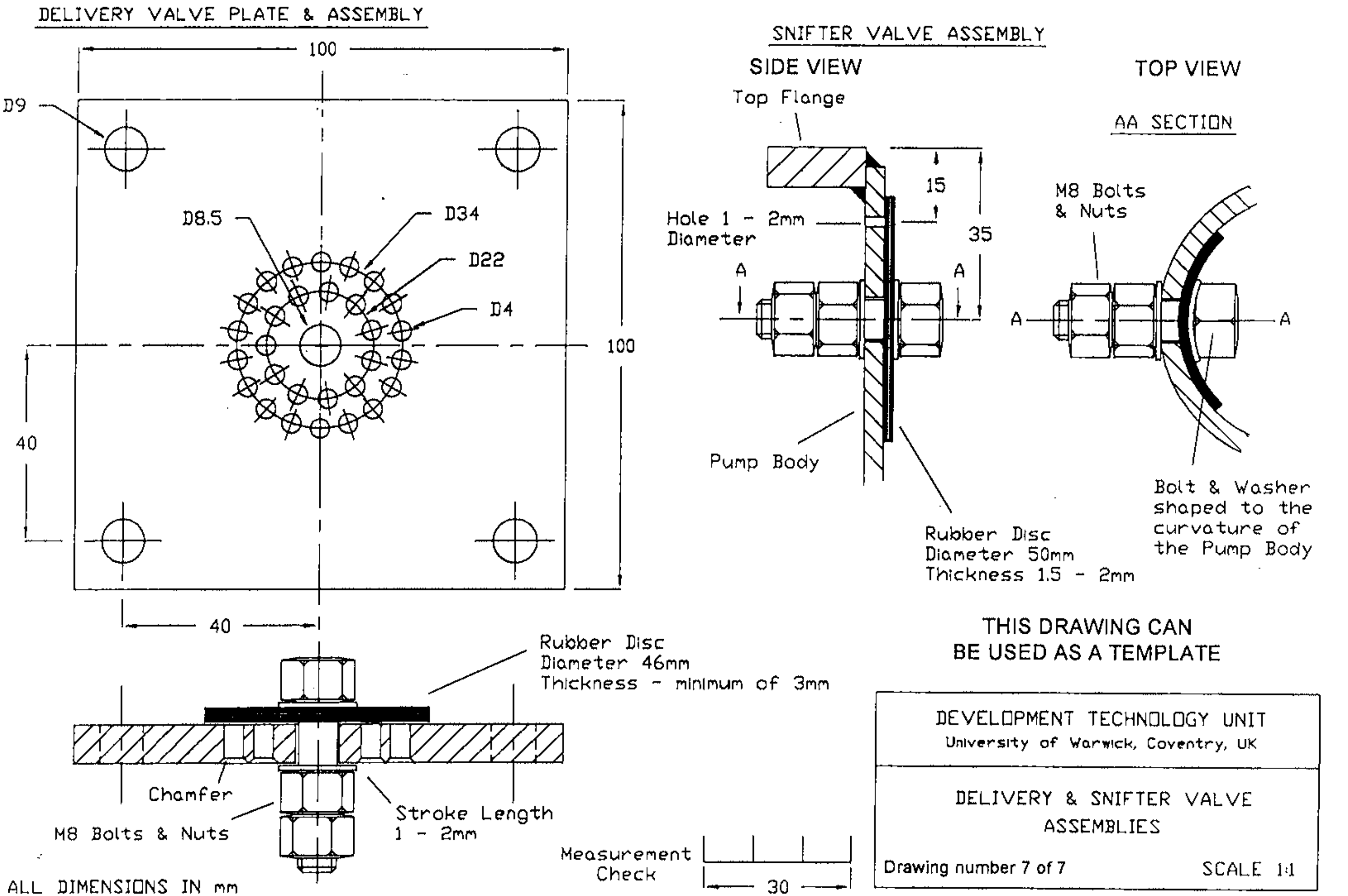
THIS DRAWING CAN BE USED AS A TEMPLATE



NOTE
The Valve Stem is made from 8mm steel bar or reinforcing bar. Use Stainless steel if possible

ALL DIMENSIONS IN mm

DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
S1 IMPULSE VALVE COMPONENTS Drawing number 6 of 7 SCALE 1:1



DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
DELIVERY & SNIFTER VALVE ASSEMBLIES
Drawing number 7 of 7 SCALE 1:1

DTU

Ram Pump Programme

TECHNICAL
12
RELEASE

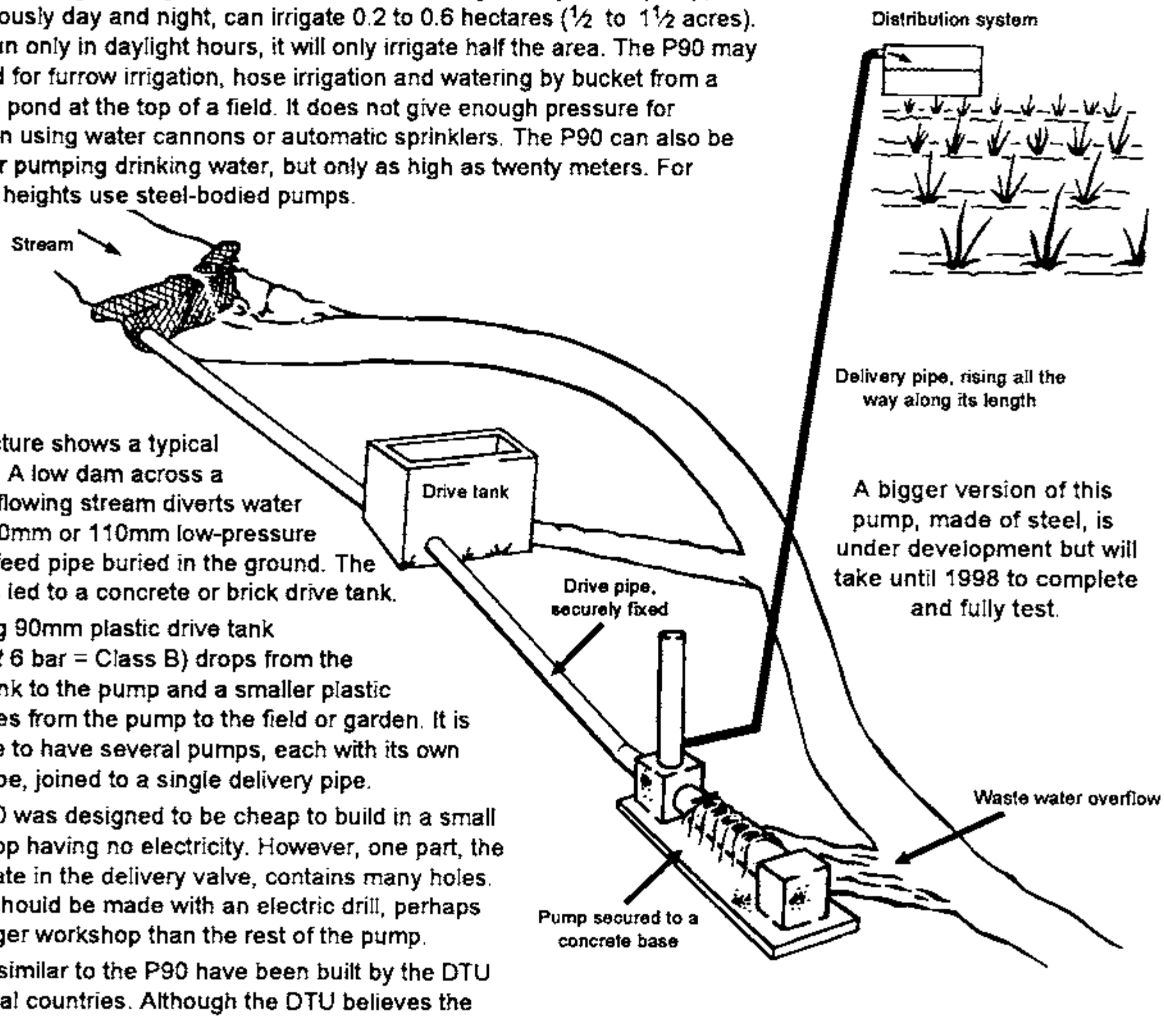
DTU P90 PUMP

DTU P90

The name "P90" stands for a Plastic pump with a drive pipe of 90mm in diameter.

hydraulic ram pump

The P90 is a plastic-bodied hydraulic ram pump designed mainly for the irrigation of vegetable gardens from streams running nearby. Each pump, used continuously day and night, can irrigate 0.2 to 0.6 hectares (½ to 1½ acres). If it is run only in daylight hours, it will only irrigate half the area. The P90 may be used for furrow irrigation, hose irrigation and watering by bucket from a storage pond at the top of a field. It does not give enough pressure for irrigation using water cannons or automatic sprinklers. The P90 can also be used for pumping drinking water, but only as high as twenty meters. For greater heights use steel-bodied pumps.



This picture shows a typical system. A low dam across a rapidly flowing stream diverts water into a 90mm or 110mm low-pressure plastic feed pipe buried in the ground. The water is led to a concrete or brick drive tank.

A strong 90mm plastic drive tank (*at least* 6 bar = Class B) drops from the drive tank to the pump and a smaller plastic pipe rises from the pump to the field or garden. It is possible to have several pumps, each with its own drive pipe, joined to a single delivery pipe.

The P90 was designed to be cheap to build in a small workshop having no electricity. However, one part, the steel plate in the delivery valve, contains many holes. These should be made with an electric drill, perhaps in a bigger workshop than the rest of the pump.

Pumps similar to the P90 have been built by the DTU in several countries. Although the DTU believes the P90 will operate for two years, only one pump trial (in Zimbabwe) has been run for that long.

A bigger version of this pump, made of steel, is under development but will take until 1998 to complete and fully test.

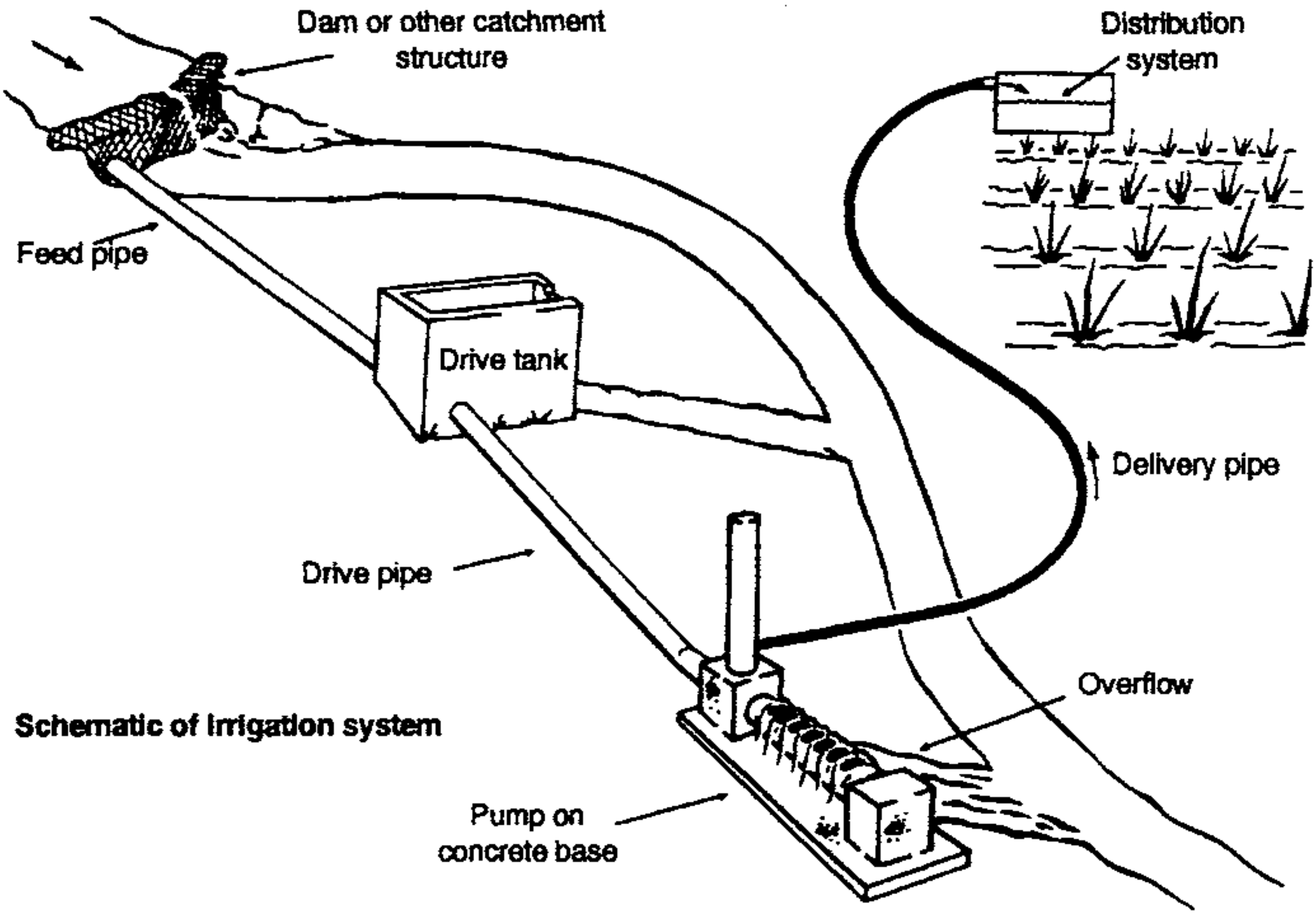
DTU P90 ram pump specifications

The normal specifications of the DTU P90 ram pump are given here. Sometimes you can operate pumps outside these limits, but they may not work well.

drive head range	—	up to 3 meters
drive flow range	—	100 to 360 liters a minute
drive pipe material	—	PVC or ABS (*at least* 6 bar or Class B)
drive pipe diameter	—	90mm
delivery head range	—	up to 20 meters
typical delivery range	—	3 to 40 liters a minute
delivery pipe diameter	—	25mm

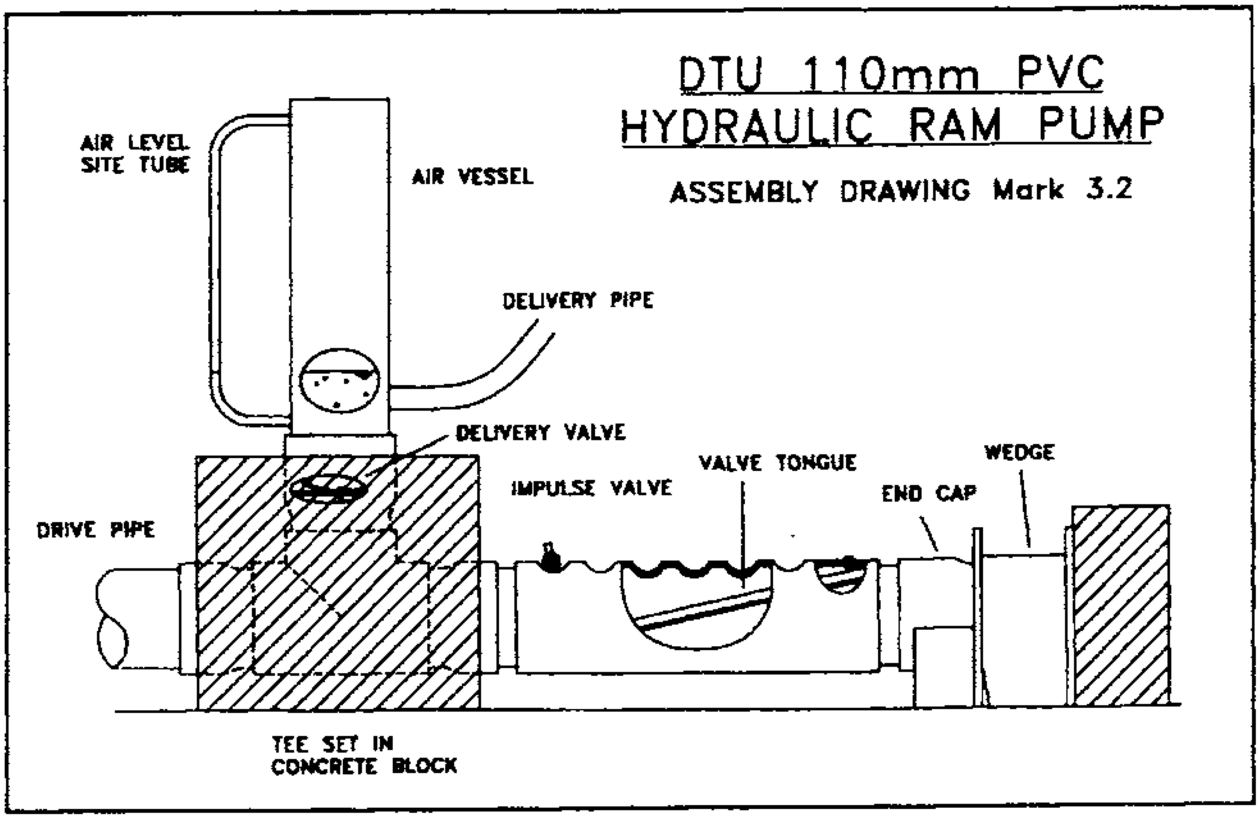
The DTU plastic irrigation pump

Most ram pumps being manufactured are designed to supply water for domestic use because the low flows produced and the costs generally make ram pump systems unsuitable for small-scale crop irrigation. Sufficient water for 1000-1500 people, for example, would only irrigate a plot of around 1 hectare! Some very large bore (12") ram pumps have been built to supply water for irrigation but the capital costs of such schemes restricts their application. In many parts of the world there is a large potential for low-lift, low-cost irrigation of small garden plots to improve yields, allow alternative crops to be grown and increase crop security in unpredictable weather conditions.



Schematic of irrigation system

The material the DTU selected as the only viable alternative to steel is plastic pipe. PVC 110mm (4") diameter pipe is now widely available in many developing countries and provides the scale of flow necessary for small-scale irrigation.



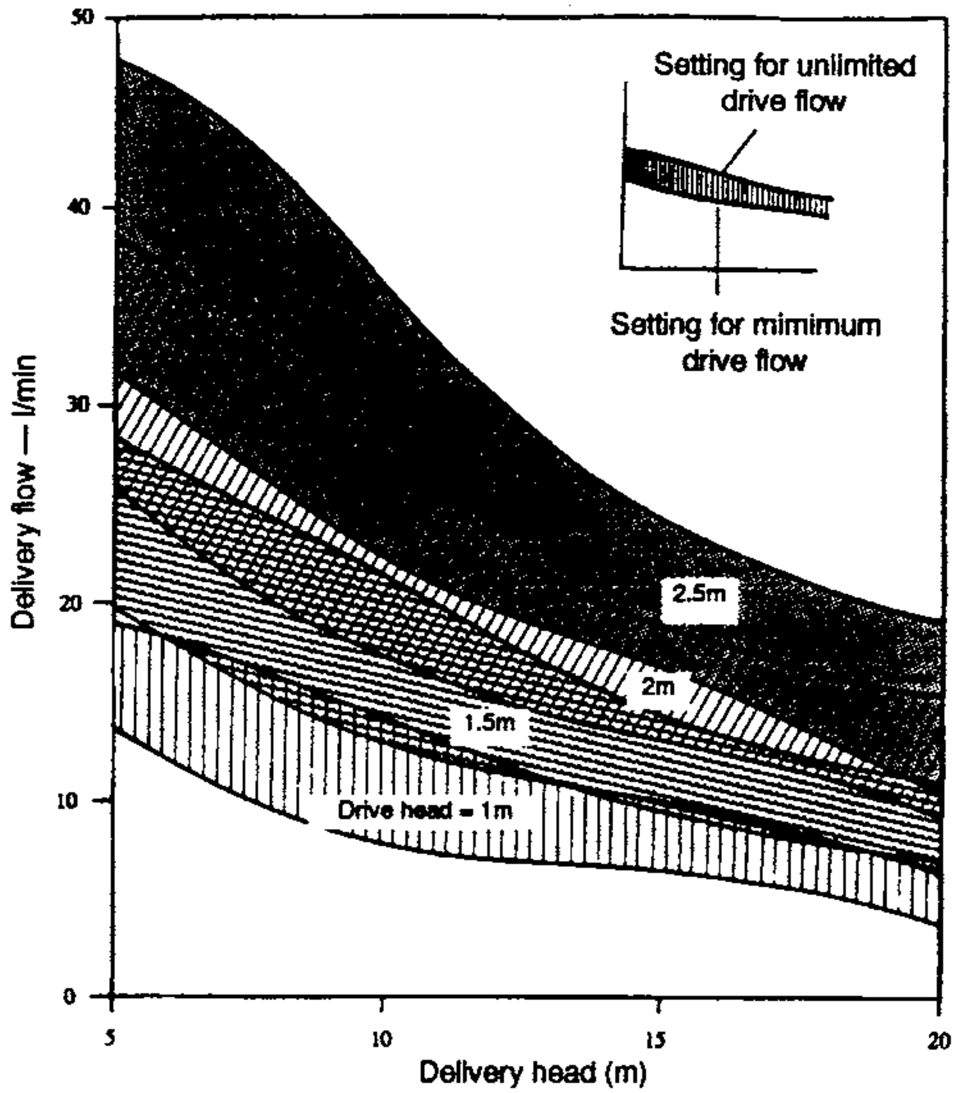
The DTU pump has the following specification:

- all major parts can be manufactured from 110mm pipe (with a minimum of other materials such as bolts, rubber, etc.);
- all parts can be manufactured using hand tools in small rural workshops;
- the system can be maintained by the user and all parts are capable of simple replacement in case of wear, damage or theft;
- designs have been extensively tested for performance and endurance (including extensive field tests);
- pumps should be easy to tune to suit a broad range of site conditions.

It meets the following performance specifications:

1. The drive head (fall of water) can be between 1 and 3 metres.
2. The drive flow available can be between 3 and 7 litres/second.
3. The maximum delivery head is 12m.
4. The energy efficiency exceeds 50%.

At 2 metres drive head, maximum delivery to 15 metres is 35,000 litres per day, adequate to irrigate ⅔ of a hectare.



DTU PVC ram pump — Typical performance data

The shaded areas on the performance data chart above indicate the normal operating range for different values of drive head.

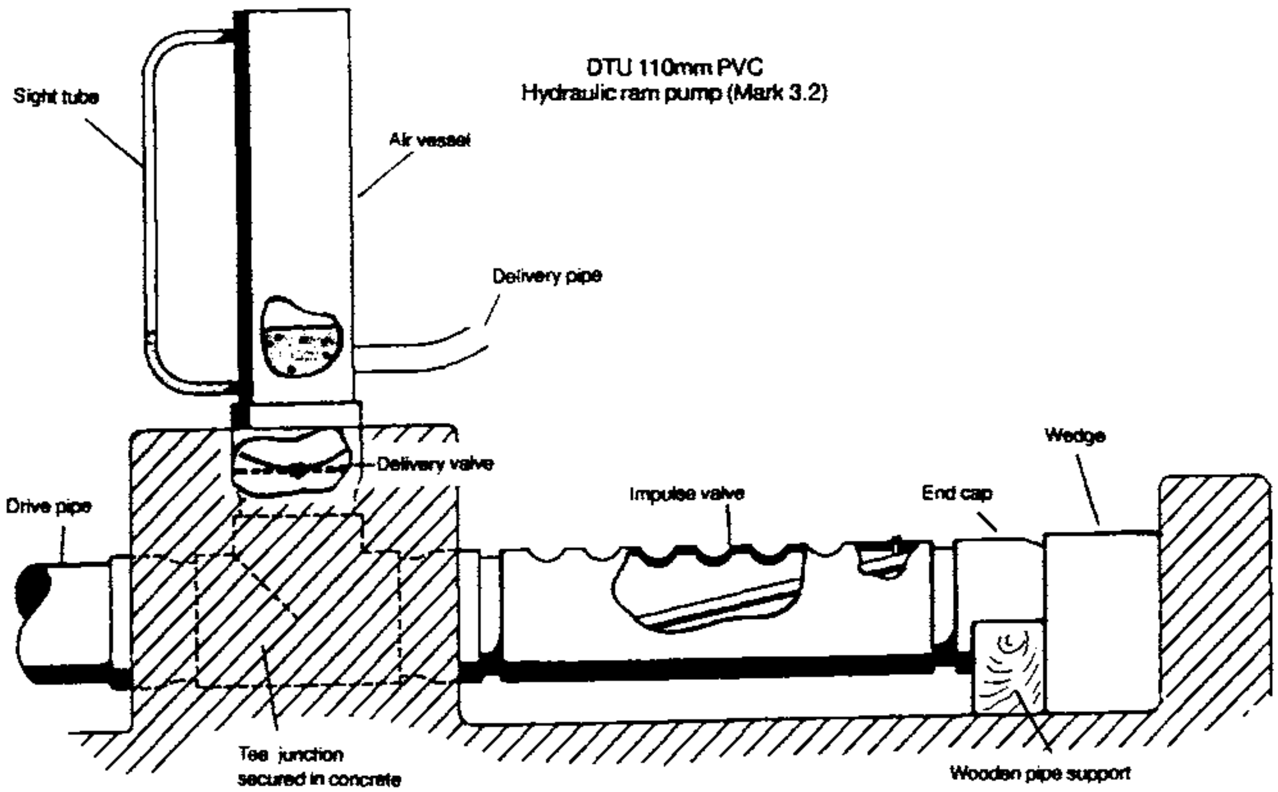
Pump designs have been developed over a number of years with increasing performance and component life. Plastic pipe is more prone to fatigue failure than steel, reducing the life of some components to two years. The current design has been extensively tested in Zimbabwe but is still (1993) at the proving stage of development.

Manufacturing notes

The first DTU ram pump to be built using plastic materials was tested in 1986 as part of an undergraduate project. It was constructed largely from standard PVC drain pipe fittings with a wooden impulse valve. The major interest in the use of plastic for ram pumps stemmed from the low cost of materials and the ease of manufacture using plastic components. There was some question initially whether the thin PVC material would withstand the continual fatigue loading and simply break under the first cycles. With the success of this first design, research was continued on its development and the horizontal axis impulse valve was introduced. Since then laboratory tests and field trials have significantly refined the design of the pump and produced data concerning its durability and pumping limitations.

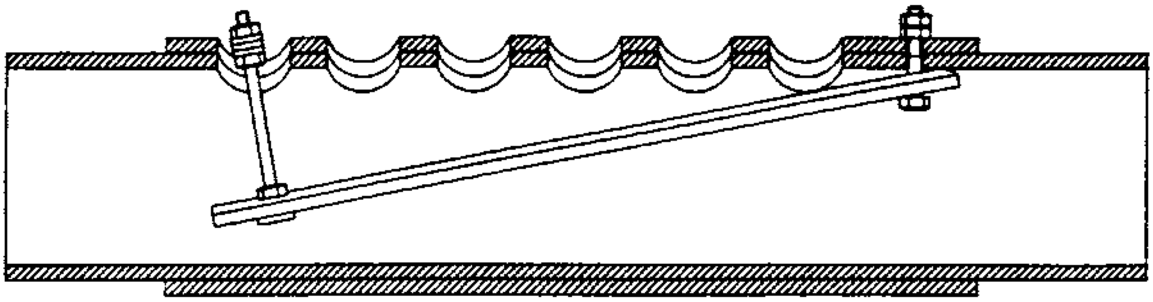
Plastic ram pumps can provide a cheap and simple means of supplying water to elevations of less than 15m. The scale of flow produced and the limitations on their pumping head make them ideal for local manufacture and use for small scale irrigation in developing countries.

This Working Paper details the design of the M3.2 pump. It lists each component and describes any significant features and considerations. Drawings of all the components are also included along with some graphs showing the typical performance of the pump.



Impulse Valve

The design of impulse valve on the DTU M3.2 is unique amongst current ram pump designs. It aims to achieve a flow area through the valve equal to, or greater than, the area of the drive pipe whilst using the drive pipe material for its construction. This greatly simplifies the materials required for pump construction and reduces the need for steel components.



The M3.2 impulse valve

The valve operates under conditions similar to those that cause the aerodynamic lift of an aircraft wing. The velocity of the water flowing over the top of the tongue and out to waste, creates a slightly lower pressure on the top side than that underneath it. As the water accelerates, this pressure differential across the tongue increases until it is sufficient to begin to close the valve. As the tongue rises the velocity over it is increased, which in turn increases the pressure differential, causing the valve to close faster and further increasing the velocity. In this way the tongue accelerates and closes very rapidly producing the sudden deceleration of the water necessary to raise the pressure in the pump body to the delivery pressure.

The valve design works well using PVC materials due to its slight flexibility. When the valve closes the subsequent pressure rise actually flexes the tongue ensuring a good seal even though the tongue may not fit perfectly to the valve body. The same valve design has been tried using steel pipe material but was found not to work well as the tongue could not easily be contoured to the exact shape of the inside of the valve body and didn't have sufficient flexibility to deform under the normal working pressure. Provided that the tongue of a PVC pump fits sufficiently well to just operate it will deform over a number of days of operation until it seals properly.

The development of the this design from the basic concept has taken some time in using different configurations; hole sizes, spacing, valve length and in finding a design with sufficient strength to withstand the continual loading. PVC is not a material recommended for use in situations where a high fatigue loading is likely. Its use would therefore not normally be advisable in ram pumps where there is a highly fluctuating load from the pressure transients in the pump and drive pipe. Research has shown that provided the delivery pressure is kept within recommended limits, the pump and pipe material are sufficiently thick, and the pump well manufactured with no potential stress concentrators, that the normal fatigue loading experienced is well within the tolerance of the material. A Working Paper is available from the DTU detailing the research and findings of work undertaken on fatigue in PVC.

Impulse Valve Design and Manufacture Notes

1) Great care is needed in the manufacture of the holes in the valve body to ensure that there are no small cuts, indentations or cracks from the initial shaping of the holes. Once the holes have been roughly shaped they should be carefully filed until they are completely smooth to the touch and show no visual defects. This will help prevent the formation and propagation of cracks.

2) The tongue hinge is a potential source of rapid wear of the PVC material used both for the tongue and the valve body. The hinge is simply a bolt about which the tongue has some freedom to move. To prevent wear of the tongue, the hole that goes through it is lined with a piece of metal, either a small section of pipe with the correct I/D or a specially fabricated liner made from sheet steel.

3) On two occasions impulse valve cracks were found running down the valve parallel to the pipe axis at the upstream end that was pushed into the tee piece. These cracks were caused by the valve being wedged too hard into the tee piece socket. This created a significant additional hoop stress at that end of the valve and led to a crack propagating from some small imperfection in the first valve hole back to the end of the pipe. The main cause of this problem was the method of wedging the impulse valve into the tee piece against the concrete end block. It is possible to prevent this type of cracking by wedging the impulse valve more carefully. However this does not provide a fool proof solution and still allows the possibility of valve failure. The simplest method of preventing this problem is to shorten the length of the pipe that can fit into the tee socket so that the double skin of the impulse valve butts up against the tee before the valve is pushed in too tight.

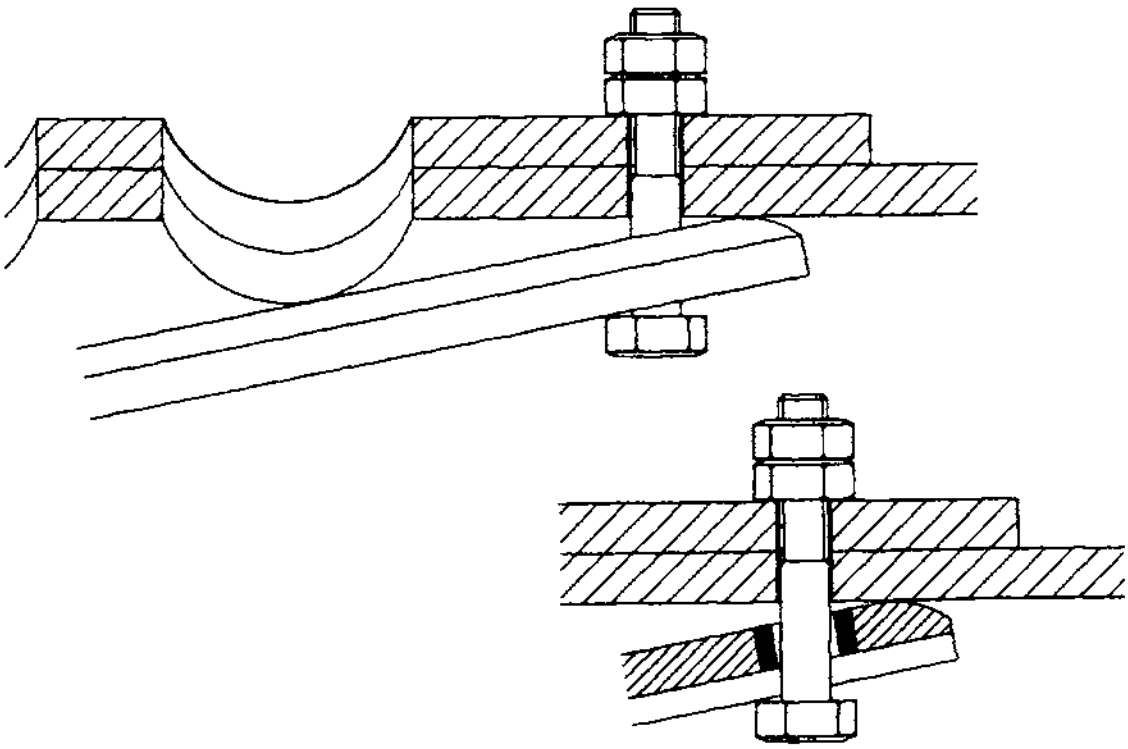


Diagram showing hinge arrangement of impulse valve tongue

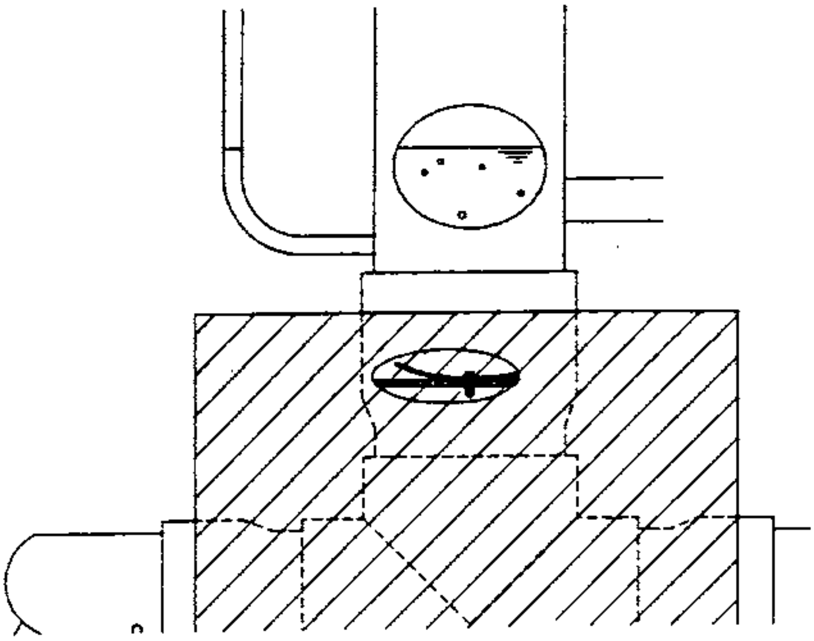
Delivery Valve Design

The delivery valve for a low cost ram pump must satisfy all of the following requirements;

1) produce good performance by; opening quickly with a low pressure differential across it; have a low resistance to flow through it; allow little backflow by closing quickly with a small pressure differential.

2) have an acceptable working life with any disposable rubber parts lasting a minimum of 6 months.

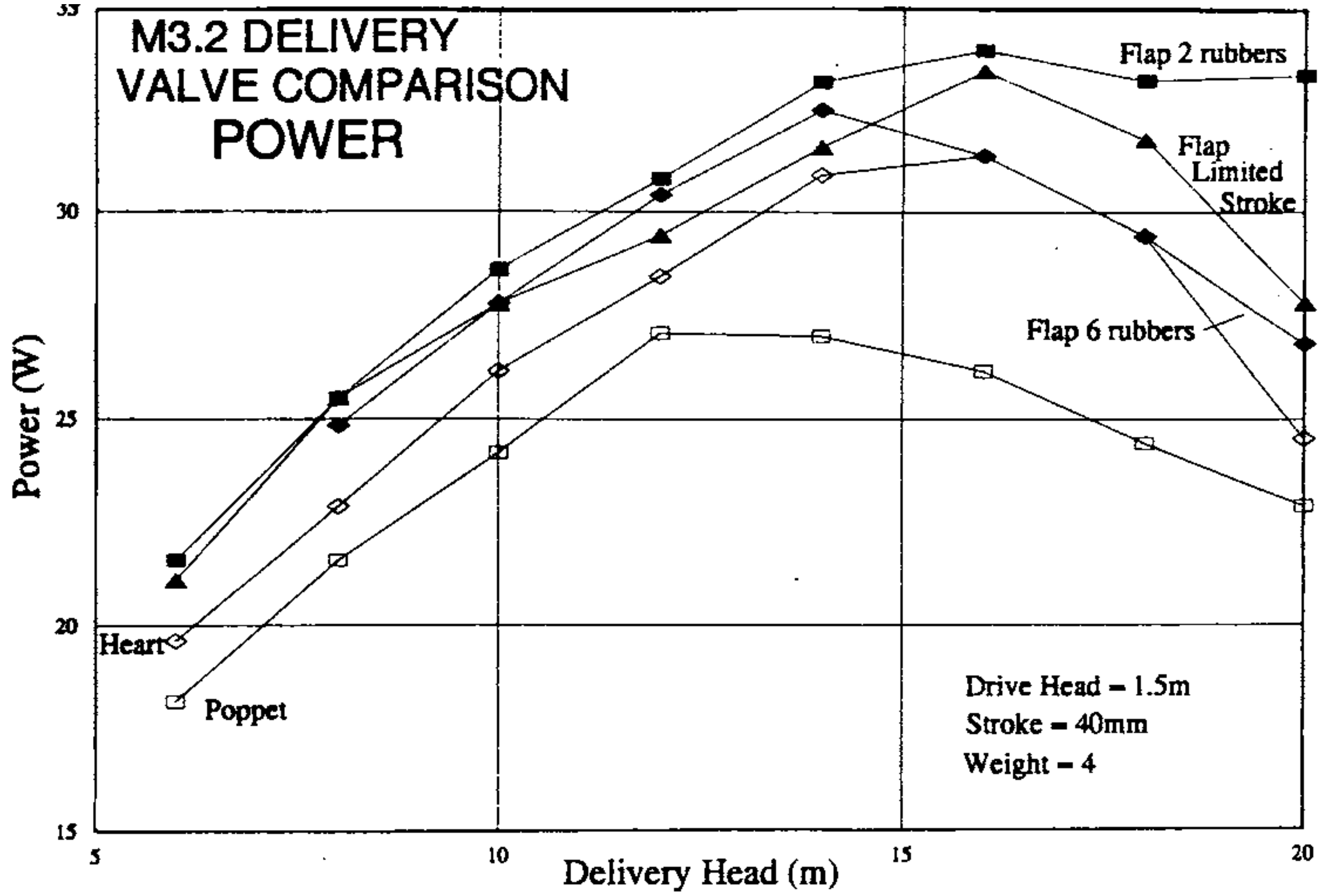
3) be cheap and simple to manufacture.



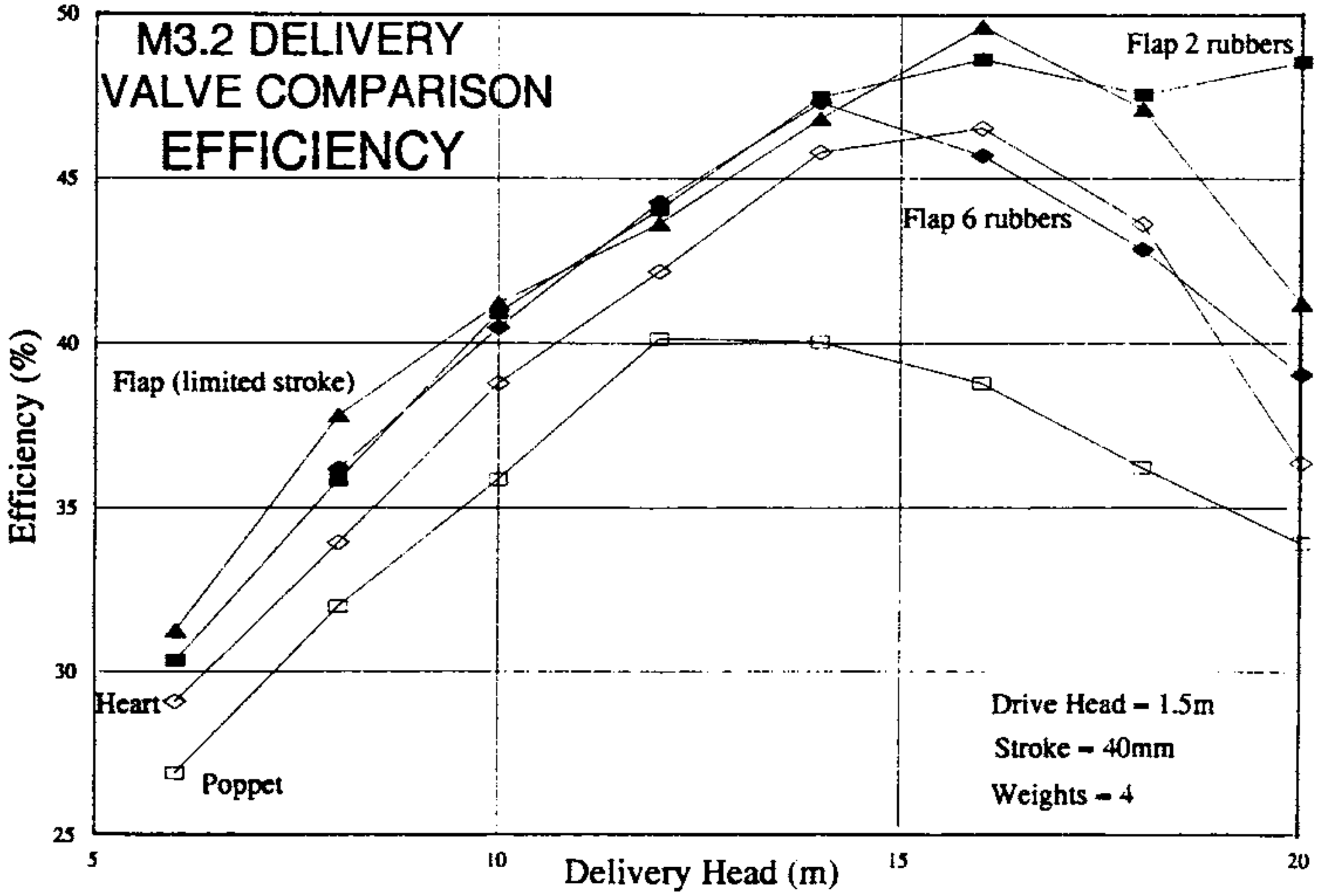
Position of the Delivery Valve

Laboratory tests have been carried out on a number of different types of delivery valve to assess their performance. These were simply designed to compare the throughput of the valve under a controlled set of conditions. Backflow of water through the valve from the air vessel during the period of recoil had been found to be significant on tests of steel ram pumps, reducing the delivery flow by up to 40% in the worst cases. With identical impulse valve settings, the delivery head and flows were measured for 5 different delivery valves. The results of the tests are shown in the following graphs.

The original flap type delivery valve proved to give the best performance over a wide range of conditions with two thicknesses of rubber to increase the rigidity of the valve. The rubber used in the tests was 3mm thick commercial sheet. One thickness of rubber gives lower performance due to its lower response to closure allowing more backflow to occur. Increasing the number of rubbers above 2 also reduces the delivery flow as the resistance of the valve to opening increases and the friction through the valve during delivery reduces the flow. Unfortuneately the quality and availability of rubber sheet varies enormously around the world and it is important to be able to make use of locally available materials to guarantee access to spare parts. Good quality inner tube from truck wheels is often the most available and reliable source of rubber. It is recommended that a number of thicknesses be used to make the total thickness of rubber between 4 and 6mm. Some experimentation with the materials available is advised. measuring performance with varying amounts of rubber to find an optimum and assess the rubbers' vulnerability to tearing and stretching.



M3.2 DELIVERY VALVE COMPARISON POWER
Flap 2 rubbers
Flap Limited Stroke
Flap 6 rubbers
Heart
Poppet
Drive Head = 1.5m
Stroke = 40mm
Weight = 4
Power (W)
Delivery Head (m)
15
20
25
30
5
10
15
20



M3.2 DELIVERY VALVE COMPARISON EFFICIENCY
Flap 2 rubbers
Flap 6 rubbers
Flap (limited stroke)
Heart
Poppet
Drive Head = 1.5m
Stroke = 40mm
Weights = 4
Efficiency (%)
Delivery Head (m)
25
30
35
40
45
50
5
10
15
20

One problem experienced with the standard flap type valve, that has been extensively used, is splitting of the rubber at the point where it is clamped. The continual movement of the valve has often been found to lead to a stress related failure at the point about which the rubber trys to hinge. This occurs most frequently when the retaining bolt holding the valve rubber is over tightened, compressing the rubber. Using a curved washer reduces the problem by not having a sharp edge about which the rubber bends and eventually is cut. The rubber instead follows a more gentle curve along its length reducing the likelihood of stress concentration and splitting of the rubber.

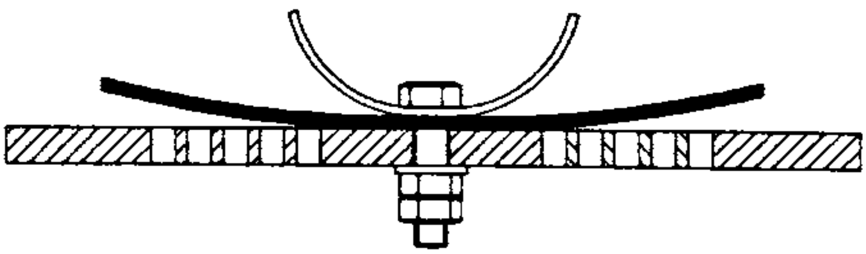


Diagram showing curved washer used in delivery valve.

Another important factor in manufacturing the rubber discs is the cutting of the central hole. This must be done cleanly with no ragged edges or flaws where cracks could start and then spread with the continual movement and stretching of the rubber. To do this it is recommended that a hole punch be bought or manufactured ensuring holes with clean edges can be accurately and repeatably made.

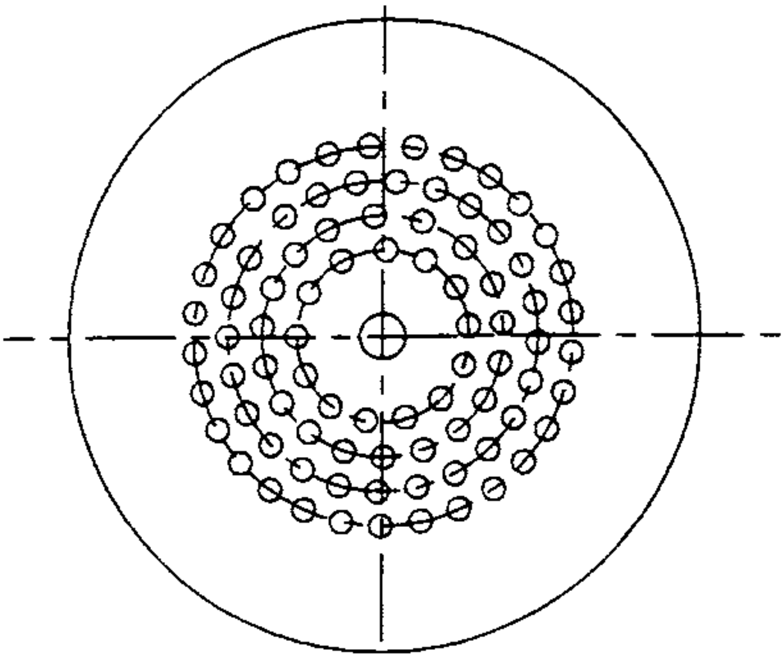


Diagram showing arrangement of delivery valve holes

The valve disc itself should be manufactured from steel plate with a minimum thickness of 3mm (10 swg) and preferably be up to 6mm. The arrangement and size of the holes in the plate is important for a number of reasons;

1) The hole size must be large enough to allow high flows with little resistance.

2) The hole size must be small enough to prevent the pressure of water in the air vessel from pushing the rubber into the holes thus deforming it and causing potential cracks.

3) The radius from the center in which the holes are confined needs to be as large as possible to maximise throughput but limited to allow sealing of the rubber around the outside of the valve plate.

4) The holes should not be too close together to prevent cracking of the valve disc between holes due to the continuous fatigue loading.

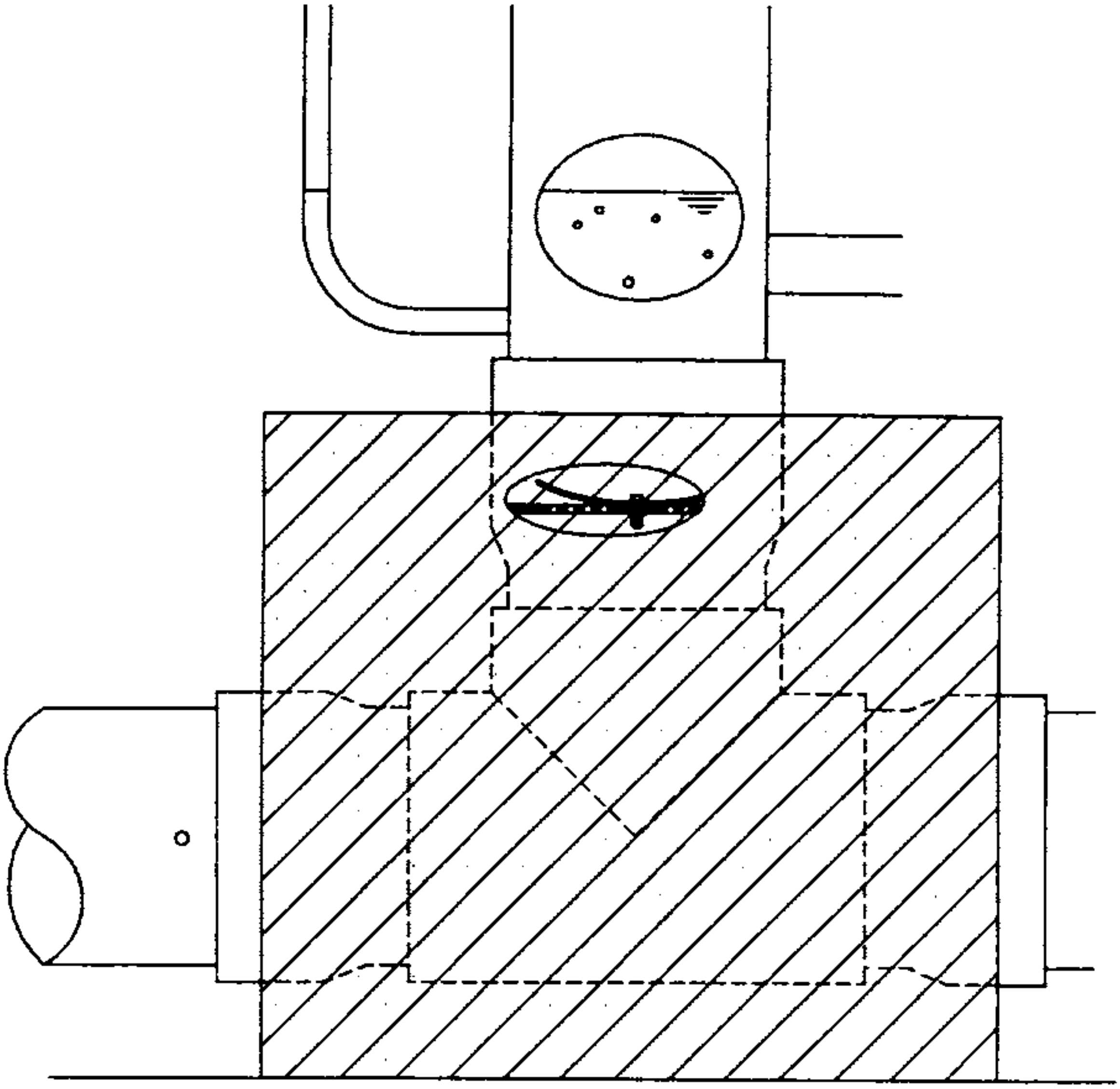
Valve plates using thick plastic material have been tested in an attempt to do without the need for a metal component. The life of such discs however was found to be too short under the loading experienced.

Marking out and accurately drilling the arrangement of the large number of holes required in the valve plate can be an arduous and time consuming process. If a number of pumps are to be made it is well worth making a jig, both to cut down on the time taken and to ensure accuracy. Under the manufacturing aids section there is a drawing of one design of a valve plate jig. It ensures both the accuracy of the circumference of the plate and the accuracy of the spacing and number of holes.

Snifter Valve

During operation of the pump, air will be constantly taken into solution at the air/water interface in the air vessel. This slowly reduces the amount of air in the air vessel leading to inefficient operation and pressure spikes significantly higher than the static delivery head. Air therefore has to be constantly and reliably introduced into the air vessel to replenish that being lost in order to keep the air vessel full and the pump working at peak efficiency. This is done by the snifter valve allowing air to enter into the body of the pump each time the pressure recoil opens the impulse valve. The slight negative pressure during the recoil phase of the cycle draws air in through the snifter valve. It is placed to enable the air to travel to the under side of the delivery valve waiting to enter the air vessel at the next delivery cycle.

There are a number of types of snifter valve that have been used on ram pumps over the years. The main design considerations are that the valve is reliable and lets in the correct amount of air each cycle. On initial start-up of the pump the volume of air in the air vessel (at atmospheric pressure) is compressed as the delivery pressure builds up. The flow of air into the air vessel from the snifter valve needs to be sufficient to re-fill the air vessel over a period of time. If too much air is entering through the snifter, the air vessel will fill up quickly and then the excess air will simply be taken up the delivery pipe where it may create problems of air locks. The snifter therefore needs to be large enough to allow sufficient air to ensure replenishment of the air vessel but not so large that the delivery pipe has a large amount of air travelling in it.



The simplest type of snifter is a hole drilled in the pump body close to the underside of the delivery valve. In the DTU M3.2 the best place for this is just upstream of the tee piece that connects the drive pipe to the pump. A small jet of water will squirt through the hole with the pressure inside the pipe until the recoil cycle occurs when air will be sucked through the hole into the pipe. As the water accelerates beginning the next cycle, some of the air rises up and sits under the delivery valve. This snifter hole should be drilled in the side of the pipe after the pump has been started for the first time.

Use the following procedure;

1) On initial start-up of the pump allow it to settle and stabalise at the system delivery pressure.

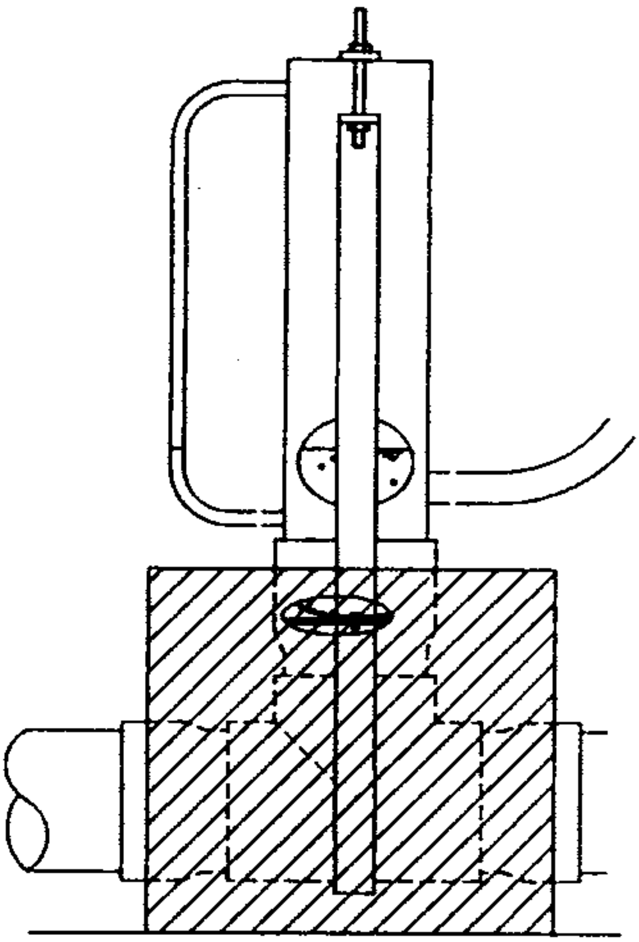
2) Make a note of the level of the air in the air vessel using the air sight tube.

3) Drill a 1.5mm dia hole in the tee/pipe as specified above.

4) Wait for 30 mins to 1 hour to see if the amount of air changes. If the air level continues to increase until it reaches the top of the delivery pipe connection, then the snifter valve is probably about the right size. If the amount of air remains constant or reduces then increase the hole diameter to 2mm. Repeat this process until the hole is big enough to allow the air level to drop down to the delivery pipe connection.

Air Vessel Size.

Over the years that ram pumps have been used there have been a number of different theories proposed and used to design air vessels. One purpose of the air vessel is to turn the intermittent flow through the delivery valve into a steady, continous flow up the delivery pipe. The air vessel provides the pump with a constant head to pump against, limits the size of the damaging pressure spikes, and removes the inefficiencies associated with intermittent flow in the delivery pipe. The size of the air vessel therefore should ensure that conditions (ie pressure) in the air vessel are little affected by the sudden inflow of water each cycle coming through the delivery valve. The volume of air in the air vessel therefore should be at least 20 and preferably nearer 50 times the expected delivery flow per cycle. An air vessel with a volume many times that of the water entering per cycle will experience little change in conditions at each delivery. Pumps running to low heads with large delivery flows therefore actually require air vessels larger than ones pumping smaller flows to high delivery heads.



For example; A pump delivering 30 l/min and operating at 60 cycles per minute has a flow per cycle of 0.5 litres. The minmum air vessel volume for this case should be 20 x 0.5 = 10 litres.

The design of the DTU M3.2 110mm PVC pump is attempting to use the fewest number of components possible. The air vessel has therefore been restricted to the same 110mm pipe used in the rest of the pump. This pipe typically has an internal cross sectional area of around 7500mm^2 and therefore a 130mm length of pipe is required per litre volume of air vessel. The maximum expected delivery flow under normal conditions is 0.5 litres per cycle requiring a 10 litre air vessel that is 1300mm long.

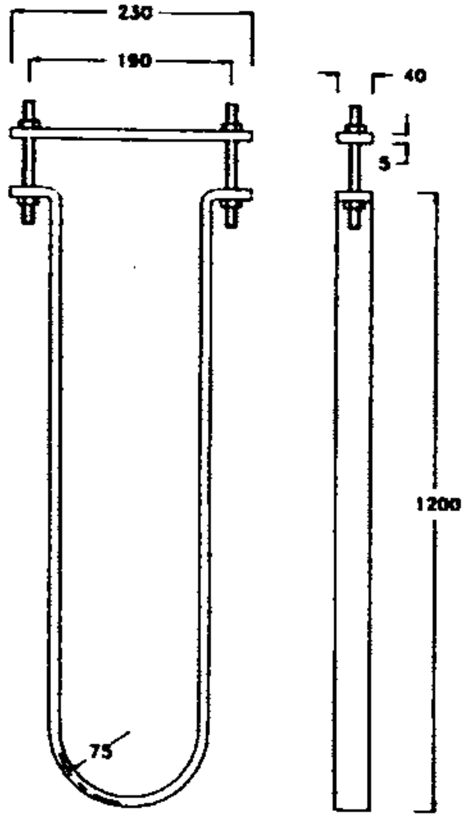
Sight tubes on an air vessel provide a very simple way of checking the level of air and therefore the effectiveness of the snifter valve. A number of problems have been encountered with the manufacture and use of air vessels that are worth mentioning:

1) Small bore, 'see-through' plastic tubing is not always easy to find.

2) Most tubing that is available deteriorates over a few months, becoming cloudy so that it is very difficult to distinguish the water level inside. Fitting a sight tube is still worthwhile under these circumstances as it is during commissioning of the pump and sizing the snifter valve that the sight tube is most useful.

3) Making connections onto the air vessel can be quite difficult. Reinforcing strips of pipe can be used over areas where the connections are to be made to increase the wall thickness of the pipe for glueing or threading.

Air Vessel Strap

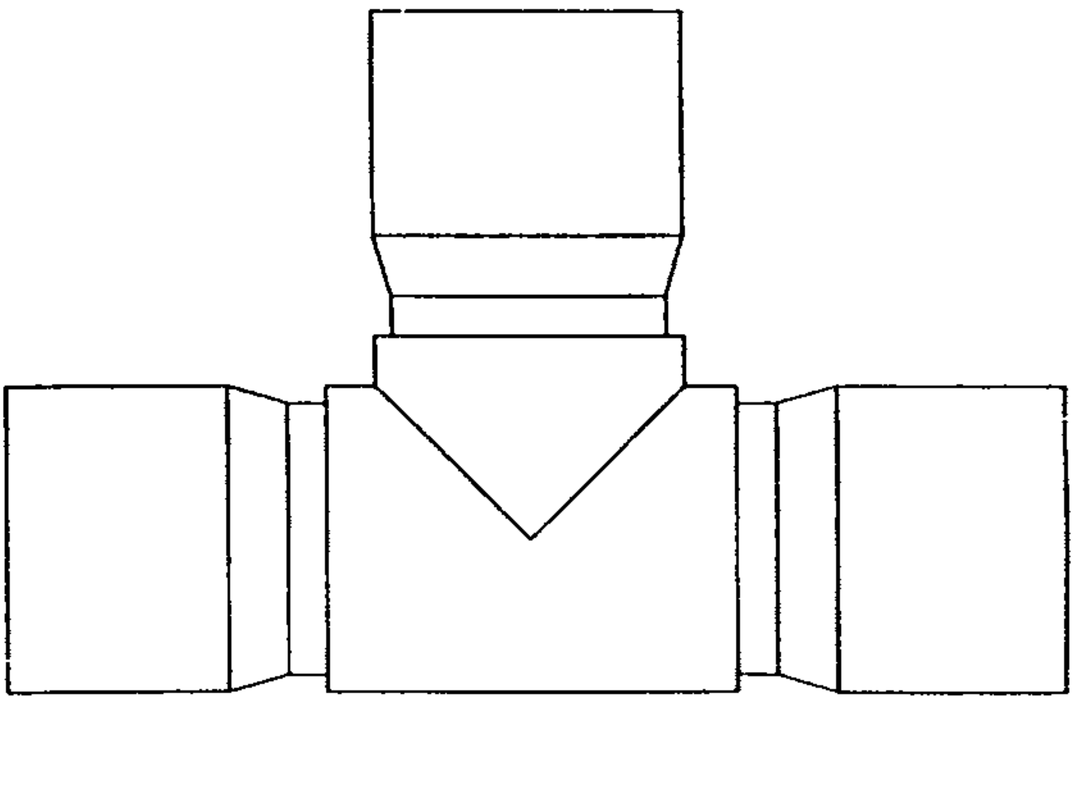


The air vessel has to be held down to prevent the pressure inside it blowing out from the tee piece. The concrete block holding the tee piece makes a good solid anchor for this strap and allows the air vessel to be securely clamped. A variety of types of clamp have been used and the one shown in the drawings section of this paper is probably the simplest.

Pipe Fittings

Tee Piece

The original design of the DTU PVC ram pump had the drive pipe connected directly with the impulse valve. The impulse valve then pushed into an elbow that was incased in concrete and fixed to the concrete pump base. The delivery valve sat in the top of the elbow and the air vessel was pushed down on top of it. With an increased understanding of pump operation and after some laboratory testing it was decided to place the delivery valve and air vessel upstream of the impulse valve. On the DTU steel pumps this had given a number of operational advantages, particularly in reducing peak pressures and so it was also applied to the plastic pumps. This altered configuration overcame some installation problems but created others.



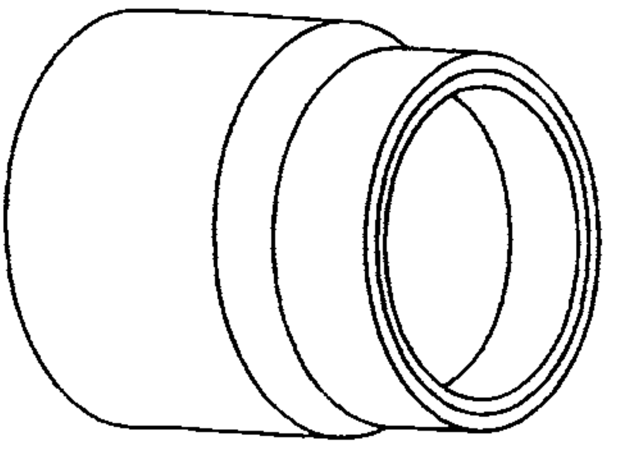
With the elbow solidly fixed downstream of the impulse valve the shocks during each pump cycle were transfered through the elbow into the concrete block. This provided good anchoring for the pump but led in some instances to cracking of the elbow. As the end of the drive pipe entering the drive tank is also fixed (and the pipe often buried) a removable section of pipe was required to allow fitting and removal of the impulse valve. This often proved to be a weak point as a good seal had to be made once the impulse valve was installed. With the revised arrangement the impulse valve is downstream of the concrete block and so provides no anchorage to prevent movement of the valve. So a further anchoring block has to be installed at the end of the impulse valve leaving enough room to allow the valve to be slid in and out of the tee piece. The gap between this second anchoring block and the valve is filled using a wedge once the valve is in place. The end of the valve also required an end cap for sealing (see below) which had to be capable of withstanding the constant shock loading. The elbow that previously experienced a substantial shock loading from the movement of the impulse valve was replaced by a tee piece that was free of this fatigue problem.

Tee fittings are commonly used in plastic pipework installations and are normally made by injection moulding. These fittings are often available in developing countries but in many cases, even where plastic pipe is manufactured, they have to be imported. They tend to be extremely expensive and are often of low quality. Experience of installations in Zimbabwe showed that the cost of a suitable tee piece was the equivalent to 3-4 meters of drive pipe - a significant cost. Such fittings were not only expensive but also in very short supply. In line with the goal of manufacturing pumps from standard pipe material a method for tee manufacture using small sections of pipe was developed. Sockets for the connection of pipes are formed by heating the ends of the pipe in an oven or oil bath and forcing them over a specially made, tapered former. Diagonal cuts are then made at 90 degress and the two sections glued together. (see drawing section). Fittings made in this way are much cheaper than those available commercially and overcome the problems of availability.

End Cap

The new pump configuration required that the end of the impulse valve be sealed with a cap. Commercially manufactured end caps are generally available and provide a good seal. However the constant movement of the impulse valve at each pump cycle crushes the end cap between the valve body and the solid end block.

It is not possible to significantly prevent this movement as the plastic material has a high degree of flexibility and so joints tend to move under the pulsing pressure experienced in the pump. Thin walled, injection moulded end caps were found to last a very short time before cracking due to the high fatigue loading experienced. On the first site installed in Zimbabwe with this configuration a number of end caps of increasing strength were fabricated. The final solution was to make a thick walled socket using the socket former, reinforced at the end with a number of extra rings of pipe. A 5mm metal disc was then shaped to fit into the end of the socket and sealed firmly in place using plumbers putty.

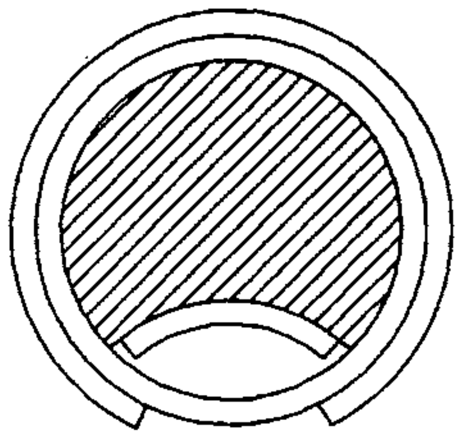


Drive Pipe

Drive Pipe Diameter

Throughout the DTU research into PVC ram pumps, the diameter of drive pipes used has been standard 4" 110mm O/D. Pumps have been designed specifically for this common size of pipe that has been found to be widely available in developing countries. The pump components themselves are now made almost entirely from this basic pipe material in order to reduce cost and simplify the raw material requirement.

The body of most ram pumps, and particularly the section housing the impulse valve, tend to be of a larger cross-sectional area than the recommended drive pipe. In the DTU steel designs for instance a 4" diameter body is used to match a 2" drive pipe diameter. This ensures that the frictional loss of water flowing through the impulse valve is low enough to allow sufficient flow in the drive pipe thus matching pump to drive pipe. A general rule that simplifies the initial sizing and design of an impulse valve is to ensure that, at all points through the pump body and impulse valve the area available for water flow is equal to, or greater than the area of the drive pipe. The design of the DTU PVC rams however is restricted to using the same 110mm pipe for the body of the impulse valve as is used for the drive pipe. The flow area through the holes of the valve is greater than the area of the drive pipe but at the leading edge of the tongue even at maximum stroke the flow area is reduced by around 20%. This is illustrated in the diagram below.

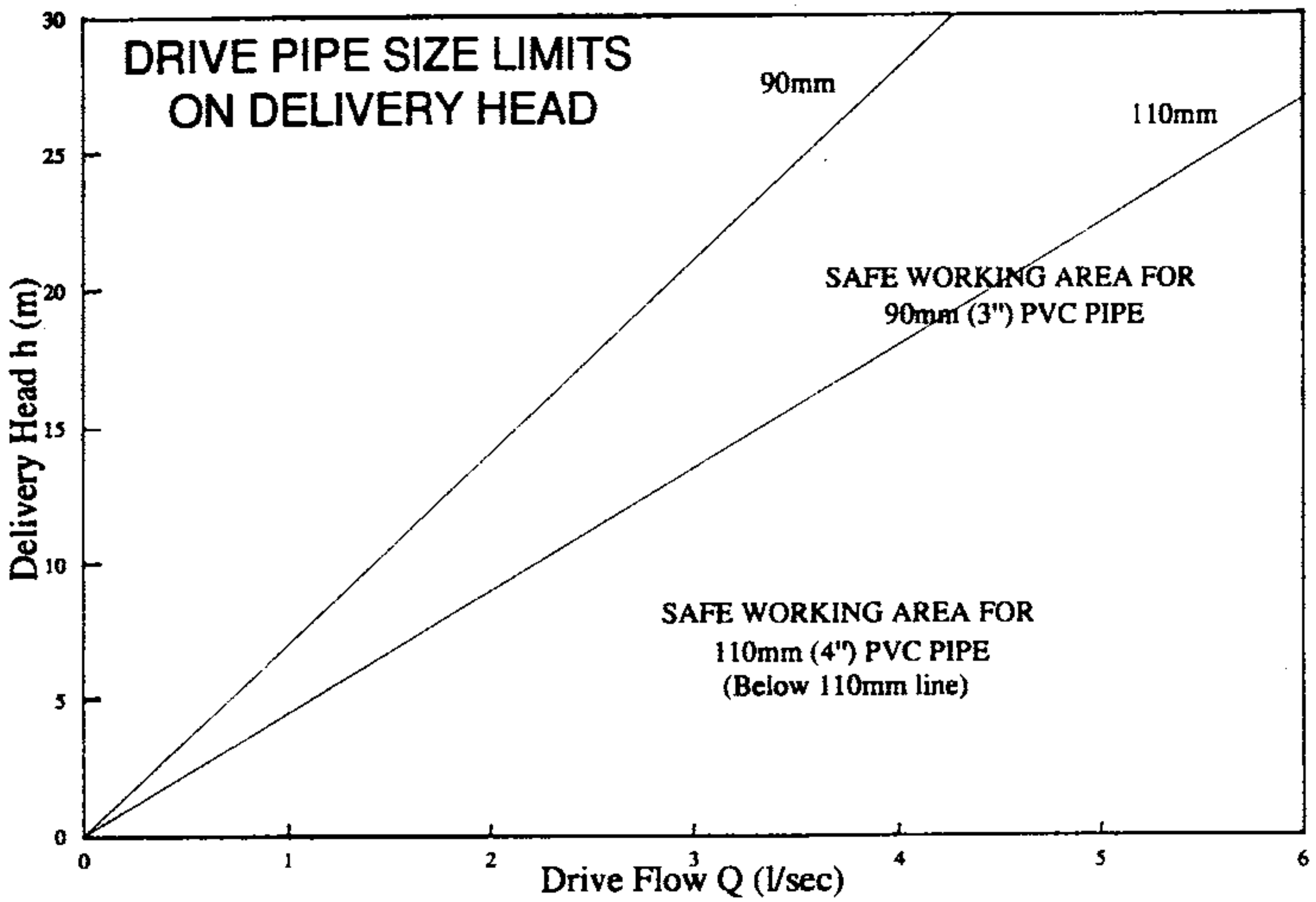


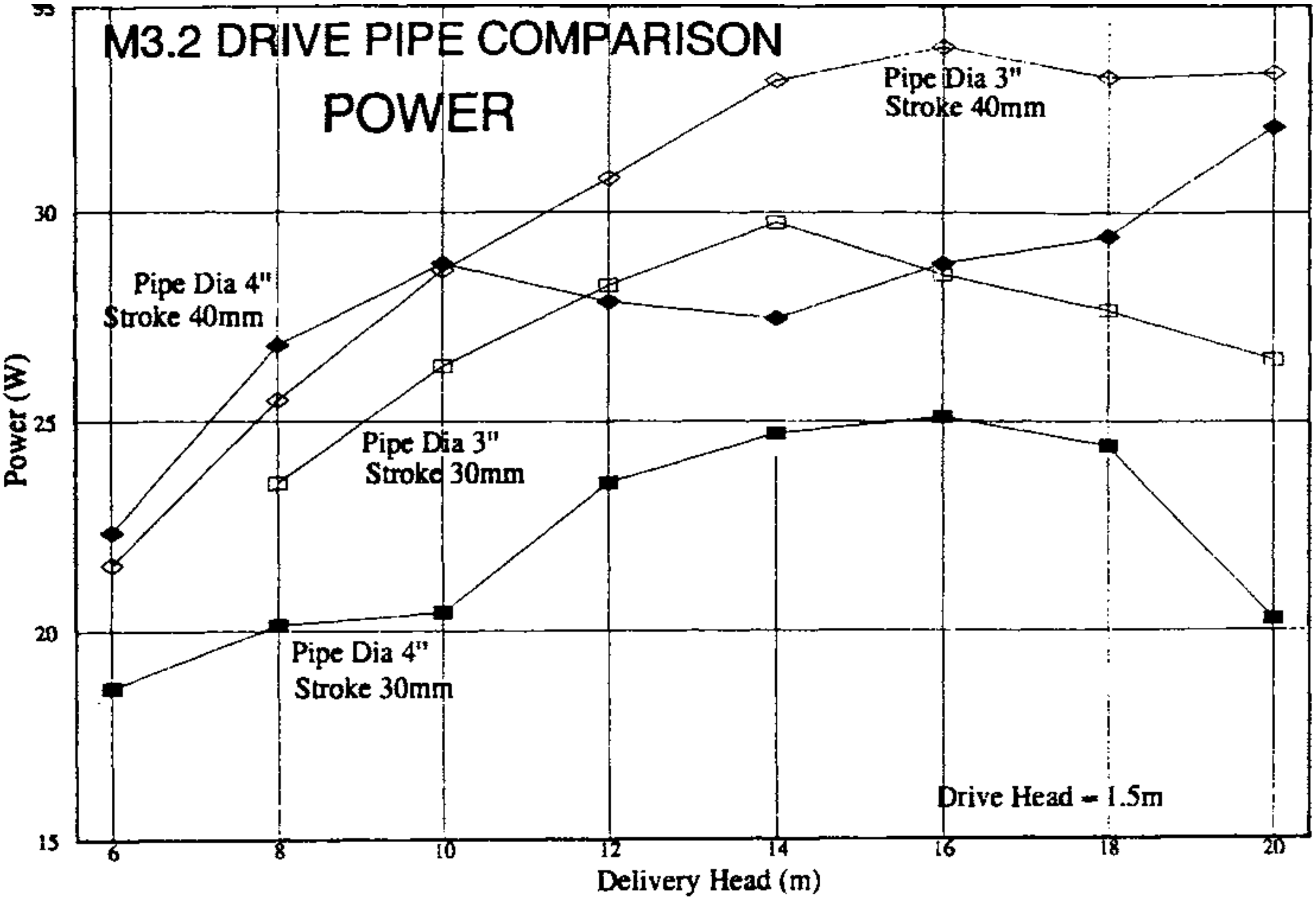
Cross-section of Impulse Valve showing restriction to flow.

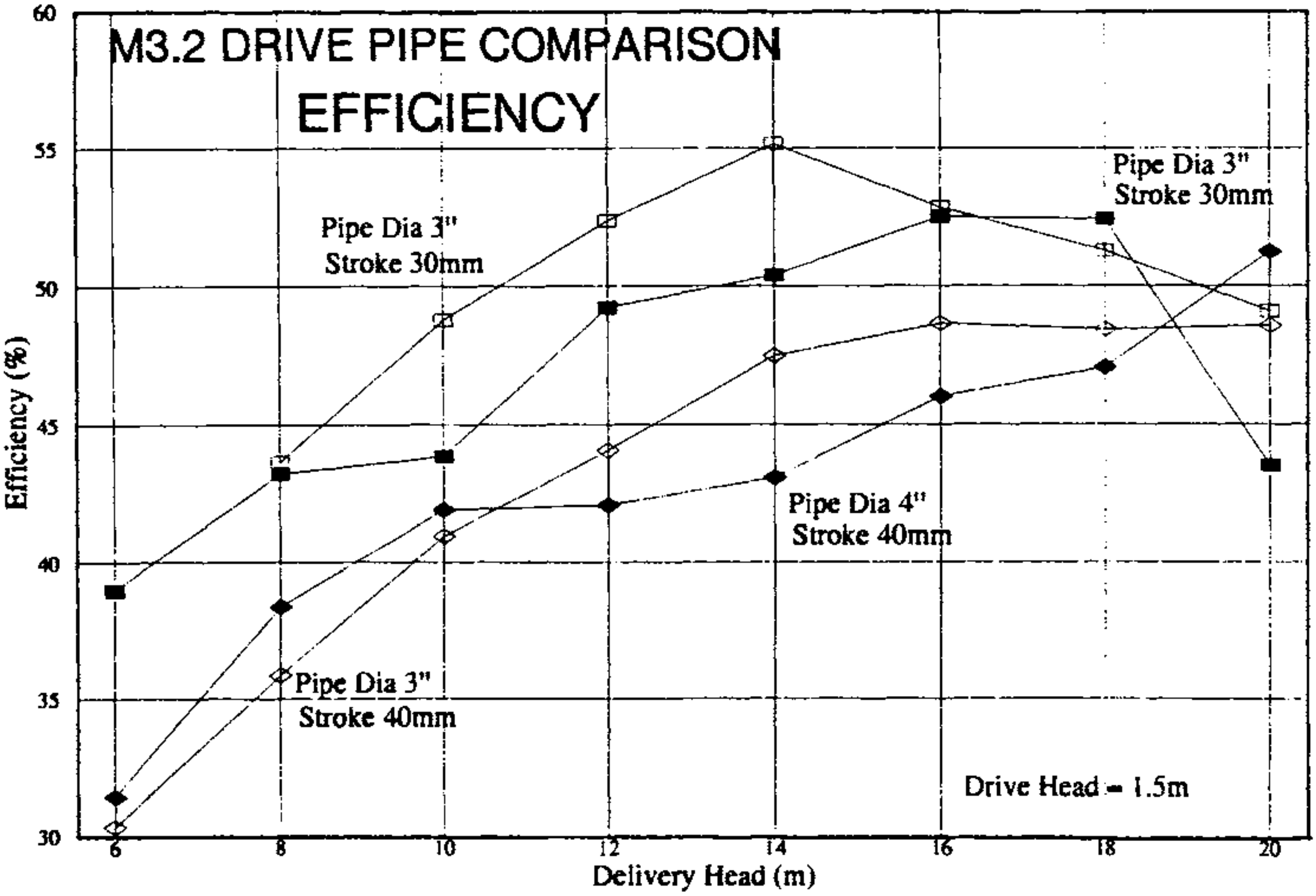
Whilst it is possible to achieve drive flows in excess of 350 l/min with this arangement it became clear that particularly at low strokes the size and design of the impulse valve was not allowing full use of the diameter of the drive pipe. This situation is greatly exaggerated at lower strokes when the pump is tuned down in order to use less drive water. In effect the velocity of water in the drive pipe when the valve begins to shut is considerably lower than the maximum capacity of the pipe. As the kinetic energy available for pumping is proportional to the mass of the water and to its velocity squared, it makes sense to keep velocities high and lose a little mass by reducing pipe diameter.

Laboratory tests have been carried out in order to compare the 110mm drive pipe with the next common size down: 90mm. The results of these tests are summarised in Graph Nos X,X and show quite clearly that under most conditions both the efficiency and power output of the pump are improved by using the smaller drive pipe size.

Whilst the pump will work perfectly well with a 110mm drive pipe over the complete range of operating conditions it is now recommended that a 90mm pipe be used if available. This may also give a small saving in system cost and gives a greater potential range of pipe that can be used if one size is not available. In situations of particularly low drive flow the pump may not actually be able to operate with the larger sizes of drive pipe. The maximum head to which a pump can deliver is dependant upon the velocity of the water in the drive pipe. To reach a given delivery head there has to be a corresponding drop in velocity turning kinetic energy to potential energy. If the velocity drop required to reach a given pressure is larger than the maximum velocity of the water in the drive pipe then the pump will never reach the required delivery head although it can appear to be operating. Smaller diameter drive pipes will have a higher water velocity for a given flow and therefore will permit operation in situations where larger diameter pipes would not work. The graph below shows the typical lower boundary of the commonest pipes used for DTU M3.2 PVC pumps



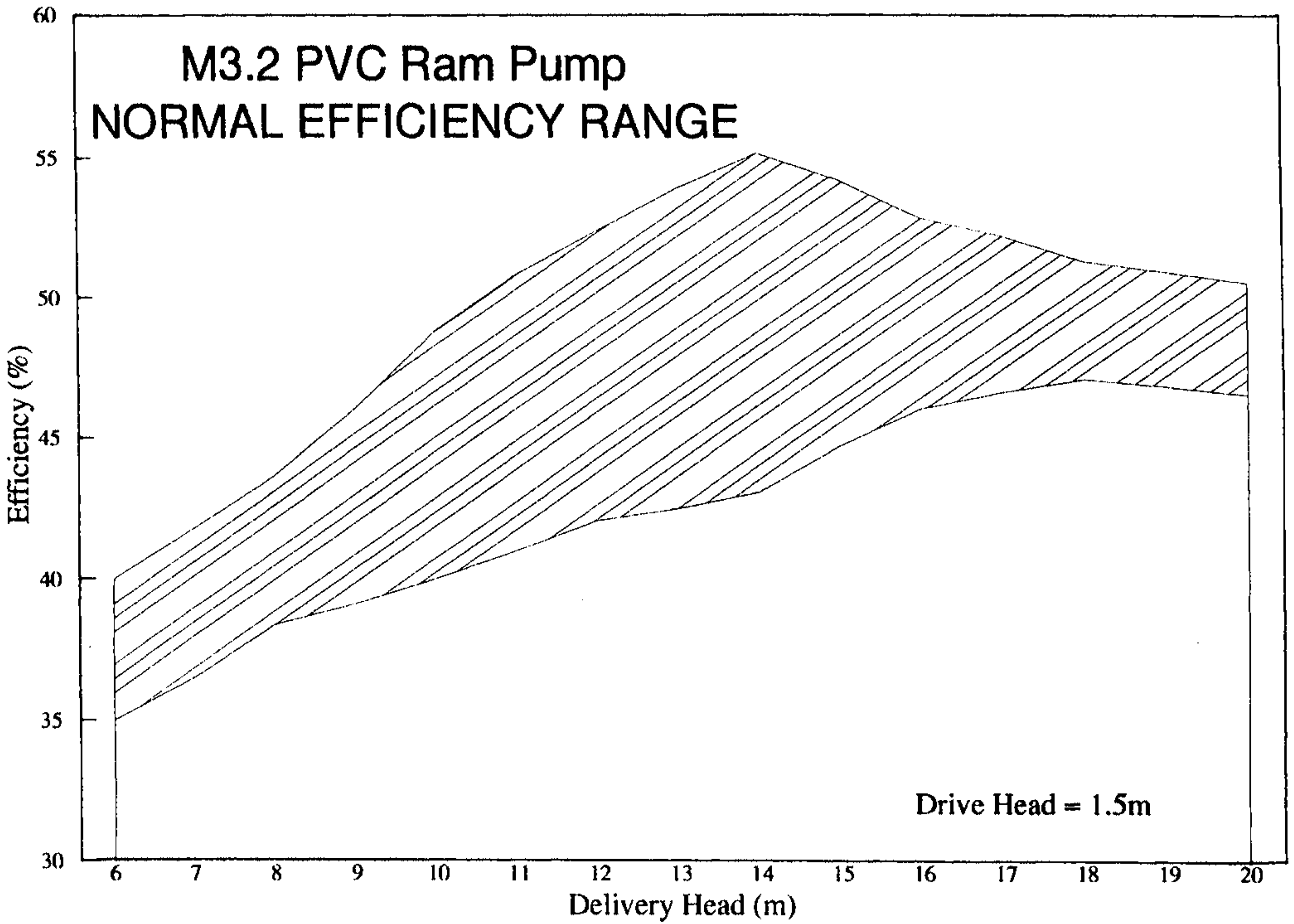

M3.2 DRIVE PIPE COMPARISON
POWER
Pipe Dia 3"
Stroke 40mm
Pipe Dia 4"
Stroke 40mm
Pipe Dia 3"
Stroke 30mm
Pipe Dia 4"
Stroke 30mm
Drive Head = 1.5m
Power (W)
Delivery Head (m)
15
20
25
30
6
8
10
12
14
16
18
20

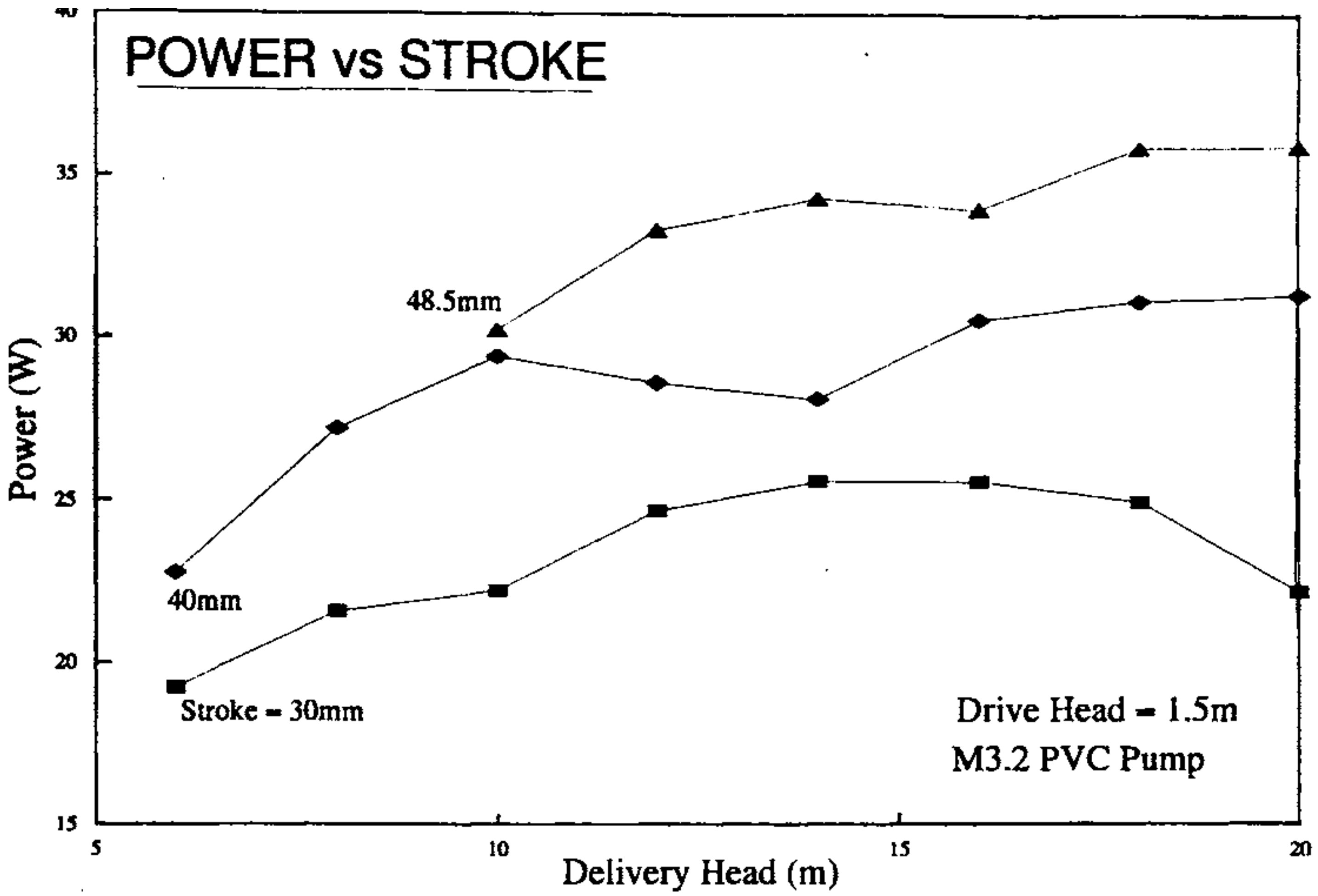

M3.2 DRIVE PIPE COMPARISON
EFFICIENCY
Pipe Dia 3"
Stroke 30mm
Pipe Dia 3"
Stroke 30mm
Pipe Dia 4"
Stroke 40mm
Pipe Dia 3"
Stroke 40mm
Drive Head = 1.5m
Efficiency (%)
Delivery Head (m)
30
35
40
45
50
55
60
6
8
10
12
14
16
18
20

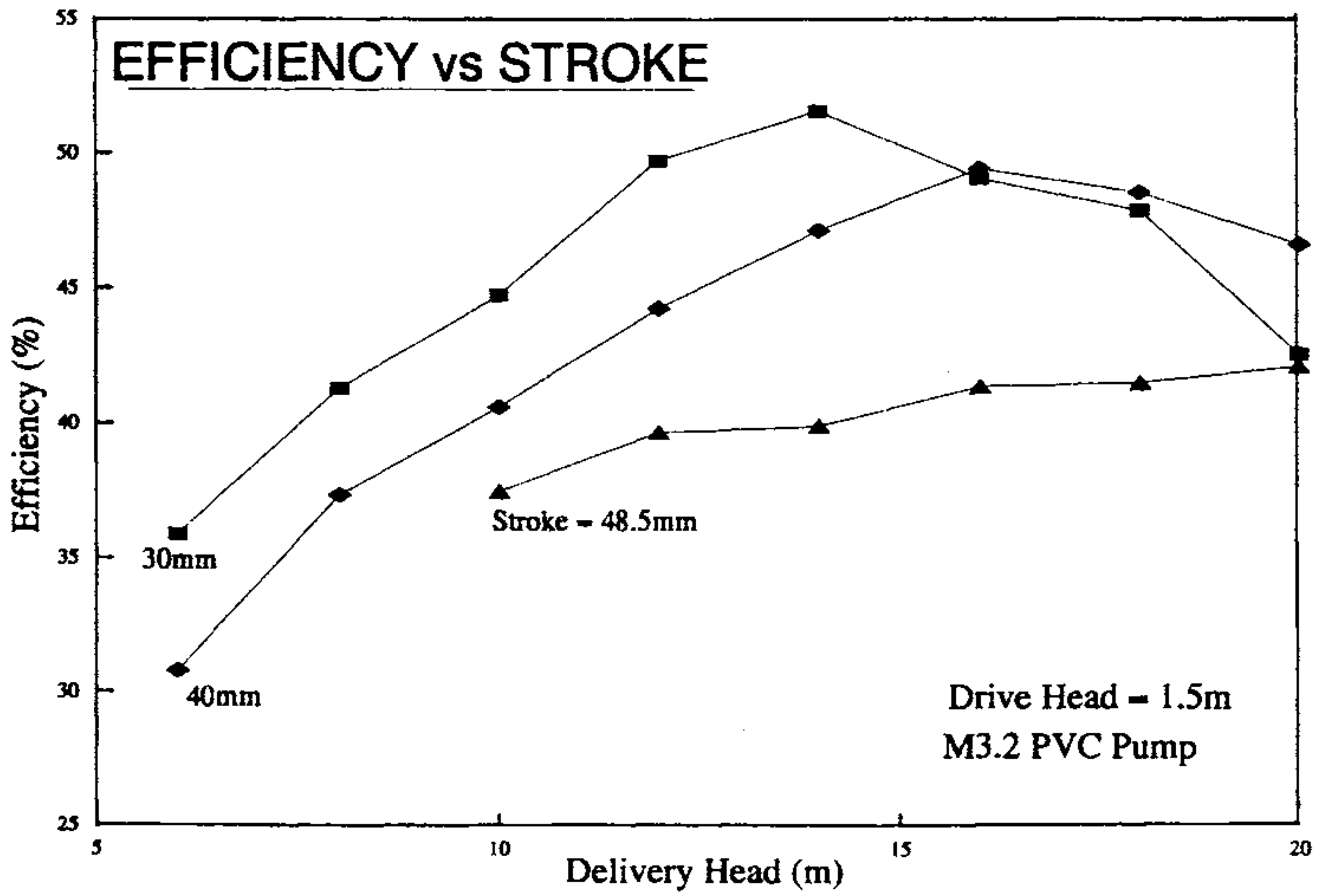
Drive Pipe Length

One critical factor with PVC ram pumps for small irrigation schemes is cost. The pump and system constraints have been designed to give good performance, reliability, local manufacture, availability of spare parts, all for the smallest possible cost. One of the major cost components of any ram pump system is the drive pipe which can often equal or even exceed the cost of the pump unit itself! As drive heads are comparatively small for PVC pumps, the physical layout of the site can often allow drive pipe lengths to be very short. Systems have been installed and successfully run with drive pipe lengths of only 6m. Plastic pumps generally can utilise shorter drive pipes than their steel equivalents due to the much lower wave speeds of the transients moving in them. Longer drive pipes can increase the pumping energy available by increasing the mass of water moving in the drive pipe and increasing the length of the delivery cycle during operation. Longer drive pipes introduce greater friction into the system so that less of the drive head is available to the pump. This consideration of pipe friction imposes an upper limit on the length of drive pipe that can be used for a given drive head.

Bearing the all important cost factor in mind it is recommended that drive pipe length normally be kept in the range 6 to 30m with 12m being a rough guide to the optimum when balancing cost and performance. If a combination of a necessarily long drive pipe (greater than 20m), high drive flow (greater than 300l/min), and low delivery head (less than 8m) occurs it is recommended that a 110mm rather than the standard 90mm drive pipe be used as the friction in a 90mm pipe would be more significant in such a situation.


M3.2 PVC Ram Pump
NORMAL EFFICIENCY RANGE
Efficiency (%)
60
55
50
45
40
35
30
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
Delivery Head (m)
Drive Head = 1.5m

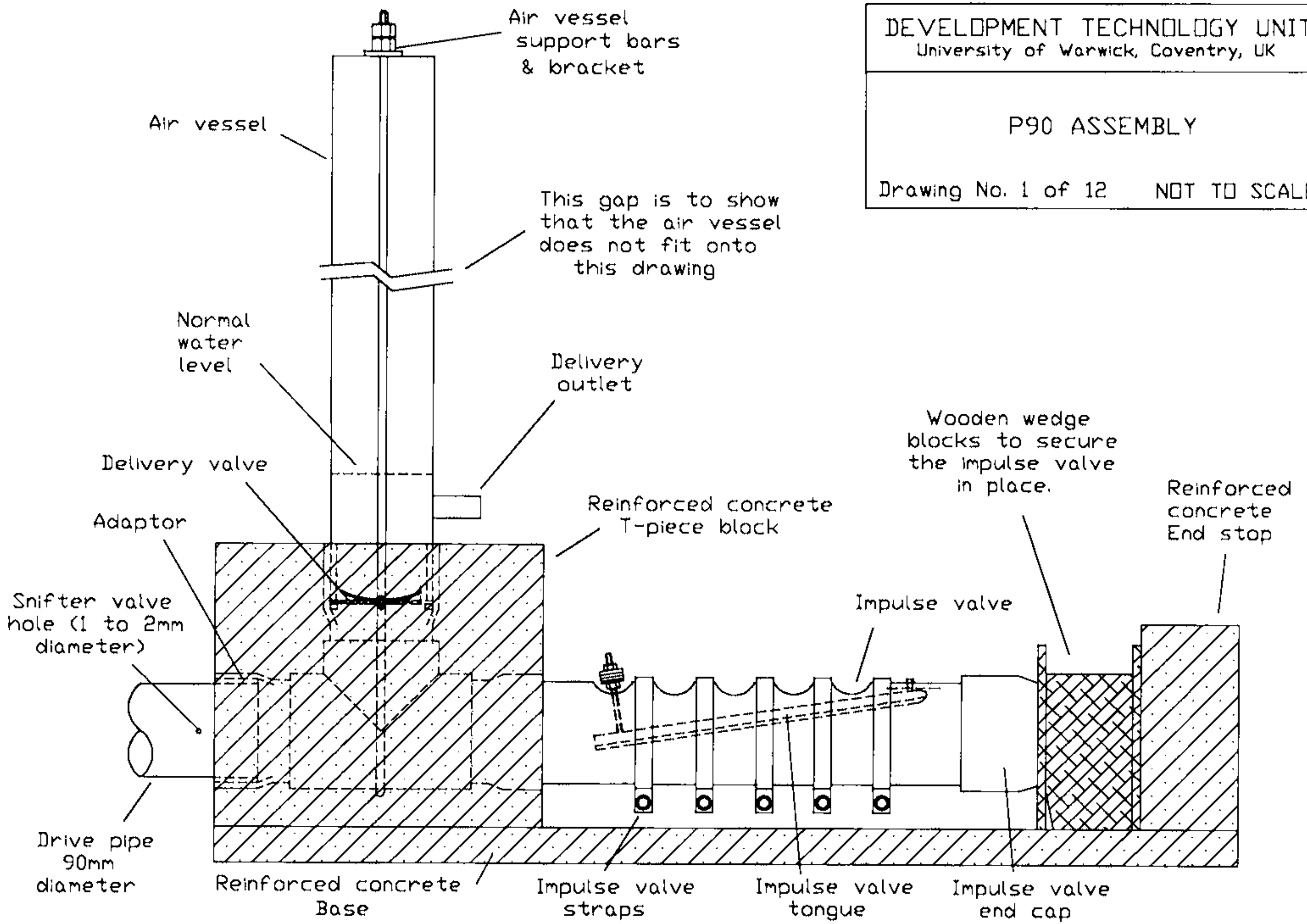

POWER vs STROKE
Power (W)
Delivery Head (m)
48.5mm
40mm
Stroke = 30mm
Drive Head = 1.5m
M3.2 PVC Pump
15
20
25
30
35
40
5
10
15
20

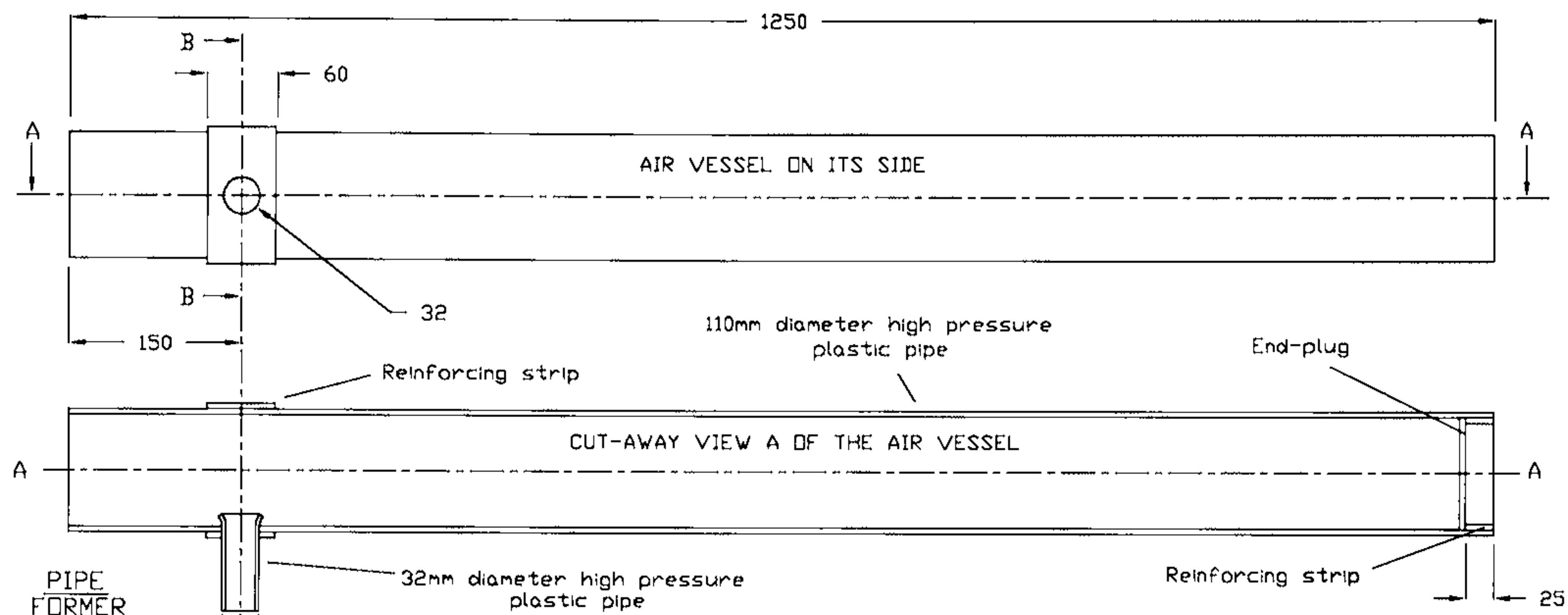

EFFICIENCY vs STROKE
Efficiency (%)
Delivery Head (m)
30mm
40mm
Stroke = 48.5mm
Drive Head = 1.5m
M3.2 PVC Pump
25
30
35
40
45
50
55
5
10
15
20

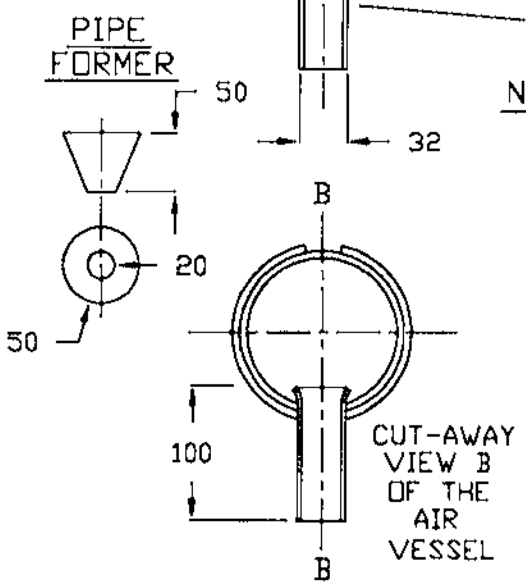
DEVELOPMENT TECHNOLOGY UNIT
University of Warwick, Coventry, UK

P90 ASSEMBLY

Drawing No. 1 of 12 NOT TO SCALE







ALL DIMENSIONS IN mm

NOTES:

1 - The air vessel is made using 110mm diameter high pressure plastic pipe and 32mm diameter plastic pipe for the delivery pipe outlet.

2 - The reinforcing strip is also 110mm diameter high pressure plastic pipe. It is cut and glued into place using a good plastic adhesive

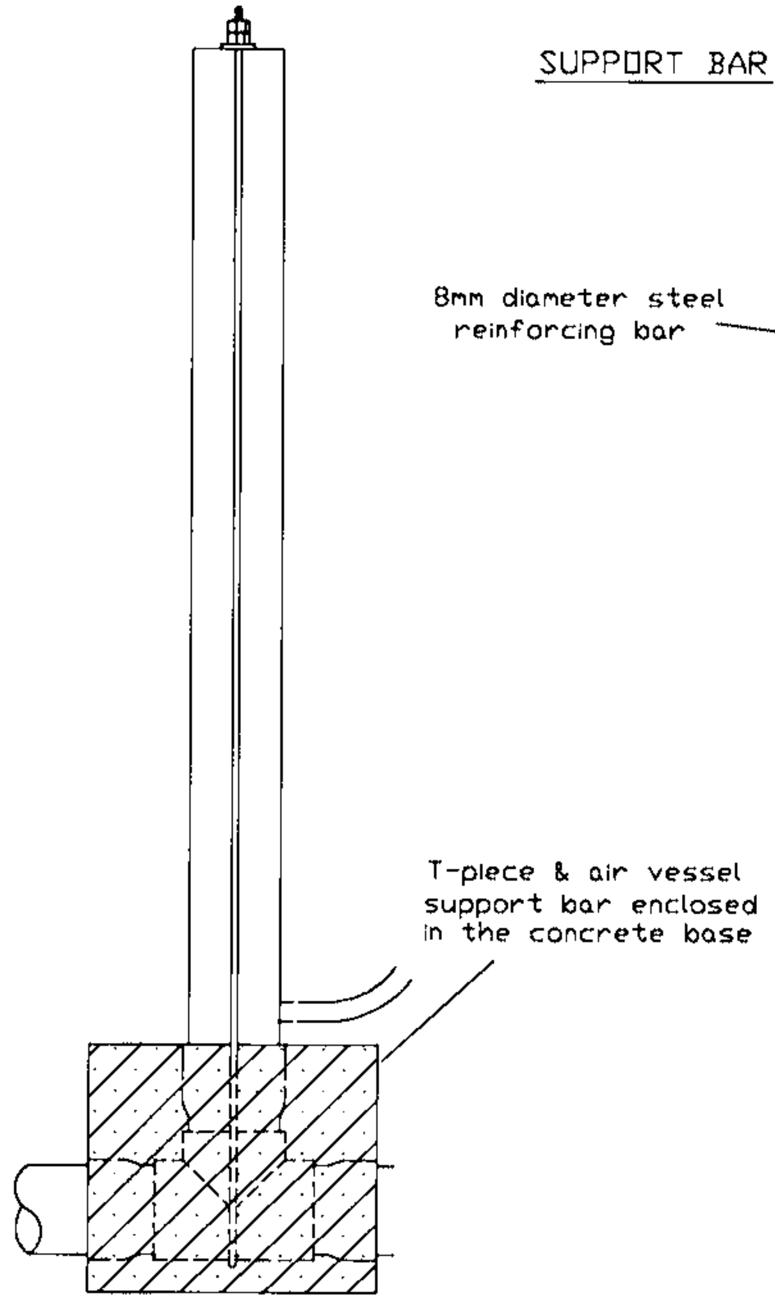
3 - To make the end-plug, a section of pipe is cut along its length, heated and flattened. Once cooled, a disc is cut out of this to form the correct shape.

4 - To secure the delivery outlet pipe to the air vessel, firstly, heat the end of the pipe until it starts to soften. Push the pipe through the hole in the air vessel and widen the pipe end. using a tapered wooden or steel former. (as shown).

5 - Use a good plastic adhesive to bond all the sections together.

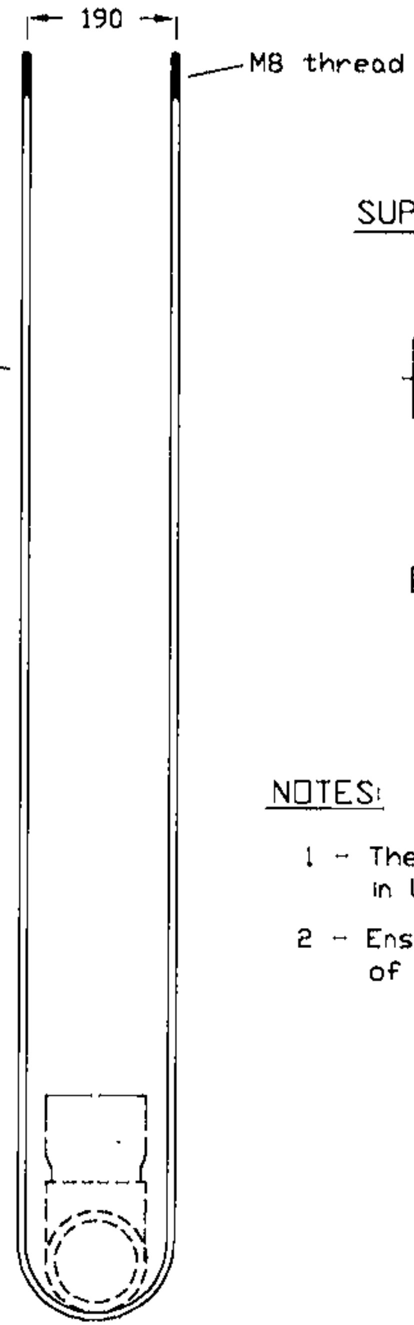
DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK	
P90 AIR VESSEL	
Drawing No. 2 of 12	NOT TO SCALE

AIR VESSEL ASSEMBLY

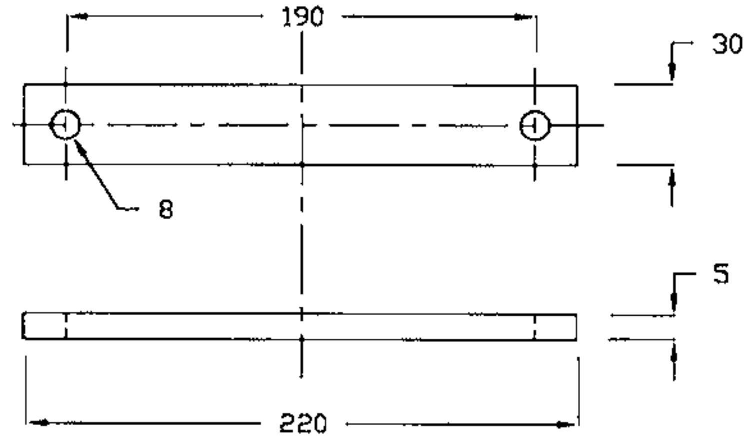


ALL DIMENSIONS IN mm

SUPPORT BAR



SUPPORT BRACKET



NOTES:

1 - The support bar is approximately 3.5 meters in length threaded at the ends (M8 thread)

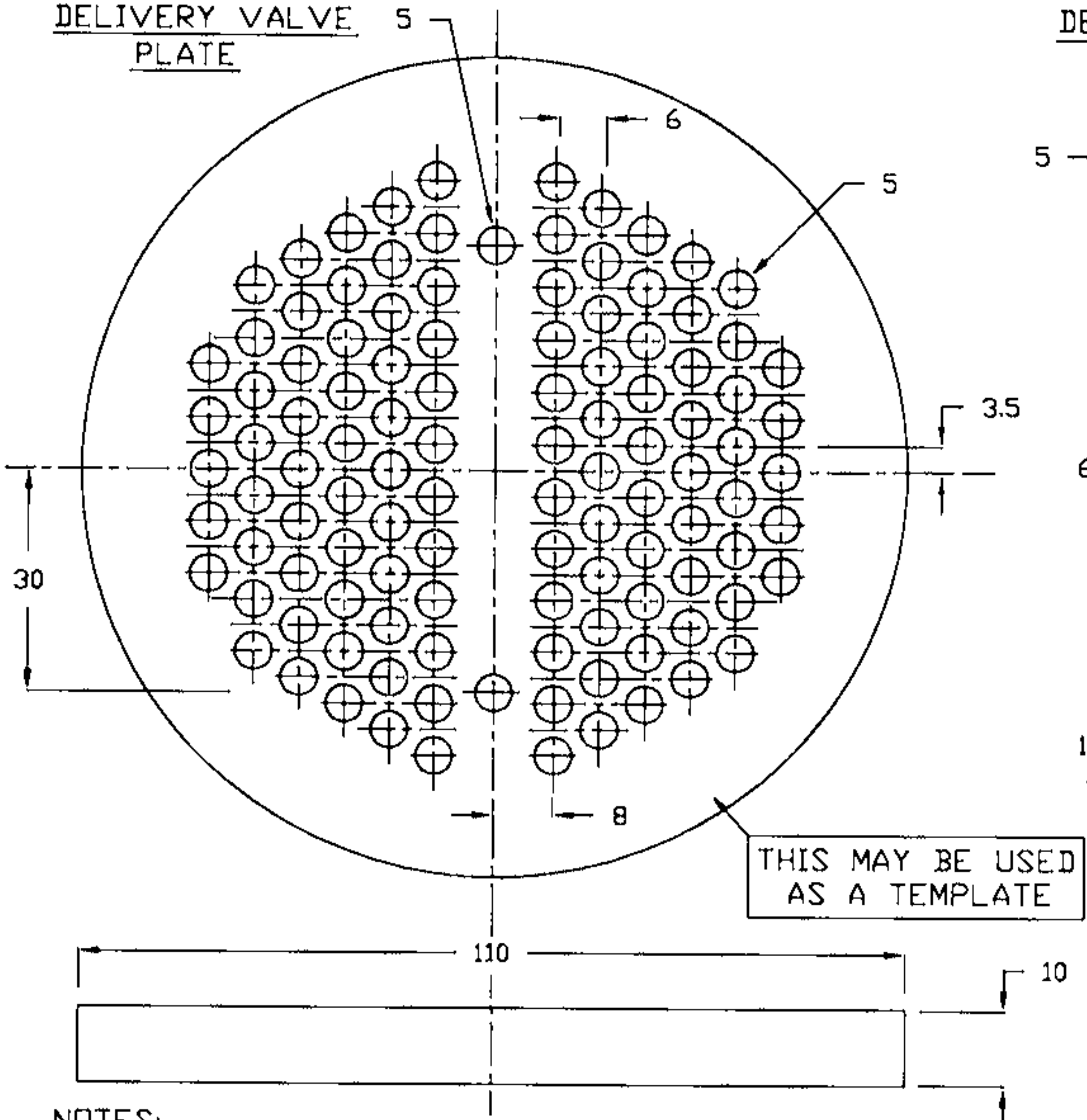
2 - Ensure a secondary lock nut is used on each end of the support bar when assembling the pump

DEVELOPMENT TECHNOLOGY UNIT
University of Warwick, Coventry, UK

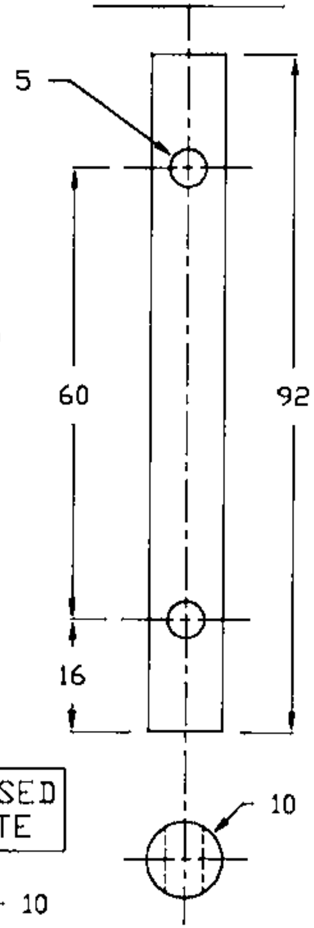
P90 AIR VESSEL
SUPPORT BAR & BRACKET

Drawing No. 3 of 12 NOT TO SCALE

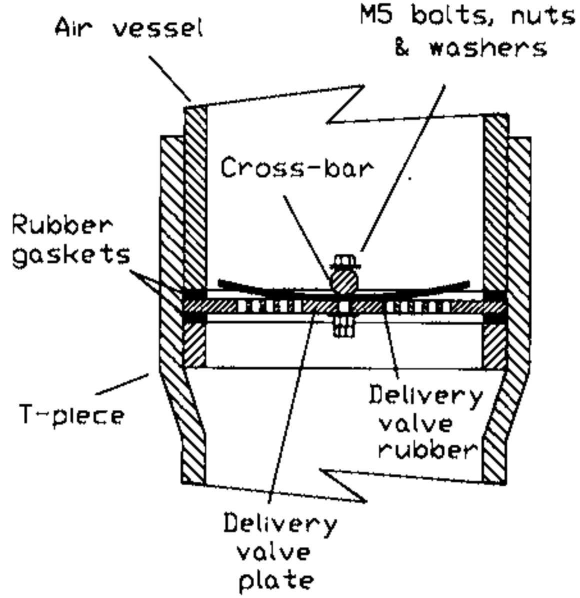
DELIVERY VALVE PLATE



DELIVERY VALVE CROSS-BAR



DELIVERY VALVE ASSEMBLY



ASSEMBLY INSTRUCTIONS

1 - Place the valve rubber on the unchamfered side of the plate
2 - Push the bolts through the cross-bar, rubber and plate
3 - Add a nut onto each bolt and just finger tighten
4 - Add a second nut onto each bolt and tighten each pair of nuts securely together

NOTES:

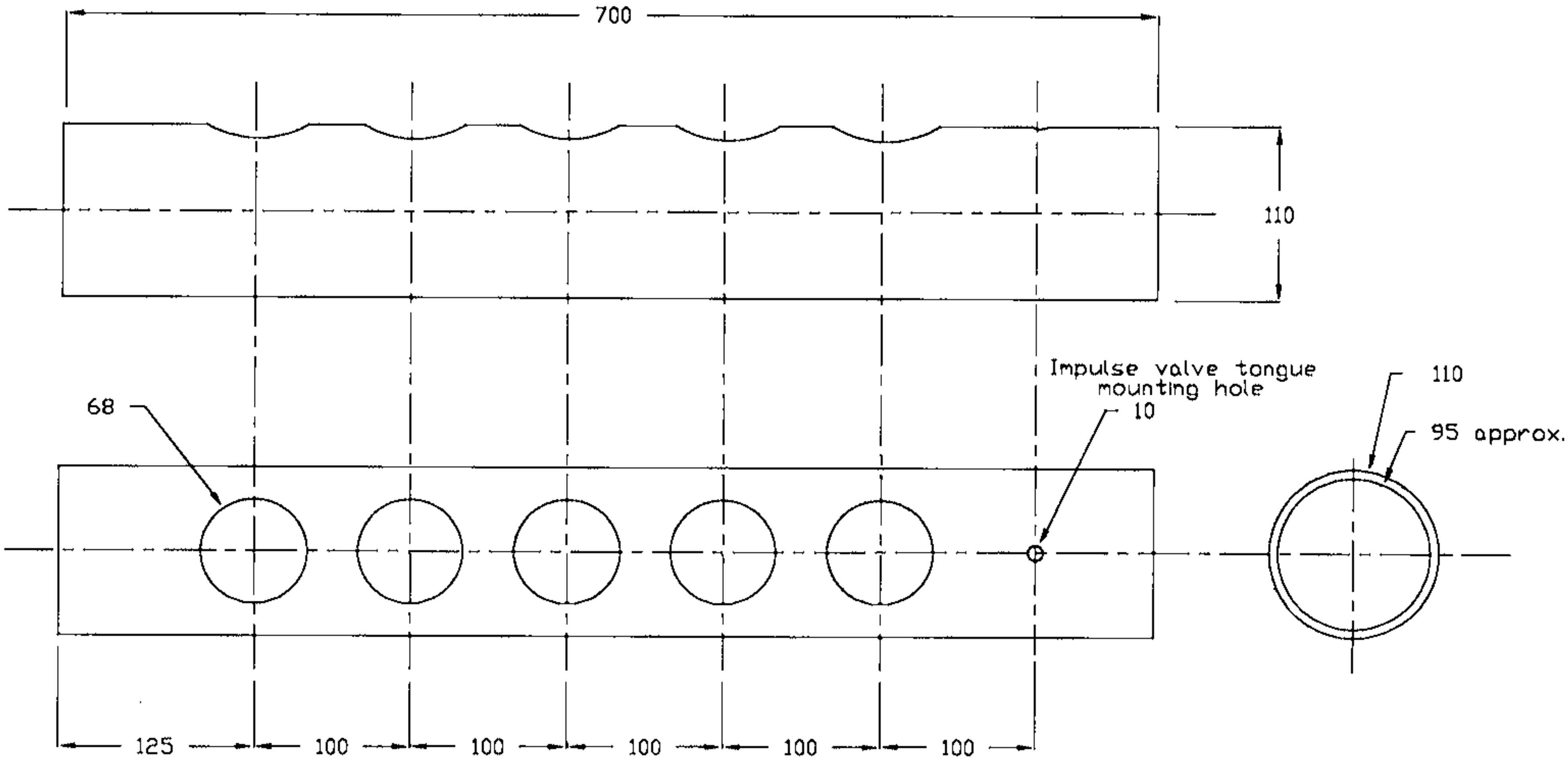
1 - Delivery valve plate is 8 or 10mm mild steel plate
All holes should be chamfered on one side of the plate
2 - Delivery valve cross-bar is 10mm diameter steel reinforcing bar
3 - The delivery valve rubber has a diameter of 92mm and should be about 3mm thick

ALL DIMENSIONS IN mm

DEVELOPMENT TECHNOLOGY UNIT
University of Warwick, Coventry, UK

P90 DELIVERY VALVE

Drawing No. 4 of 12



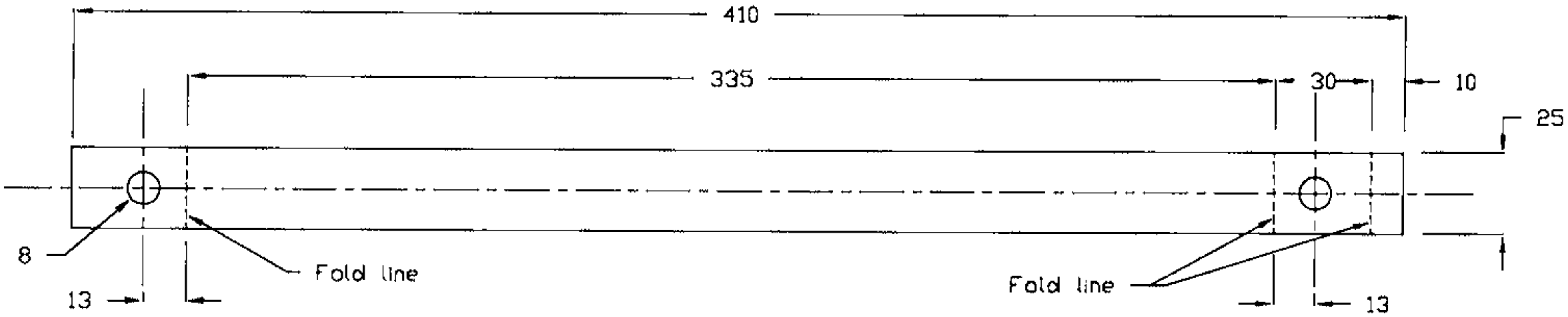
NOTES:

1 - The material used for the impulse valve is CLASS 16 high pressure PVC pipe. The wall thickness of the pipe is usually about 7mm.

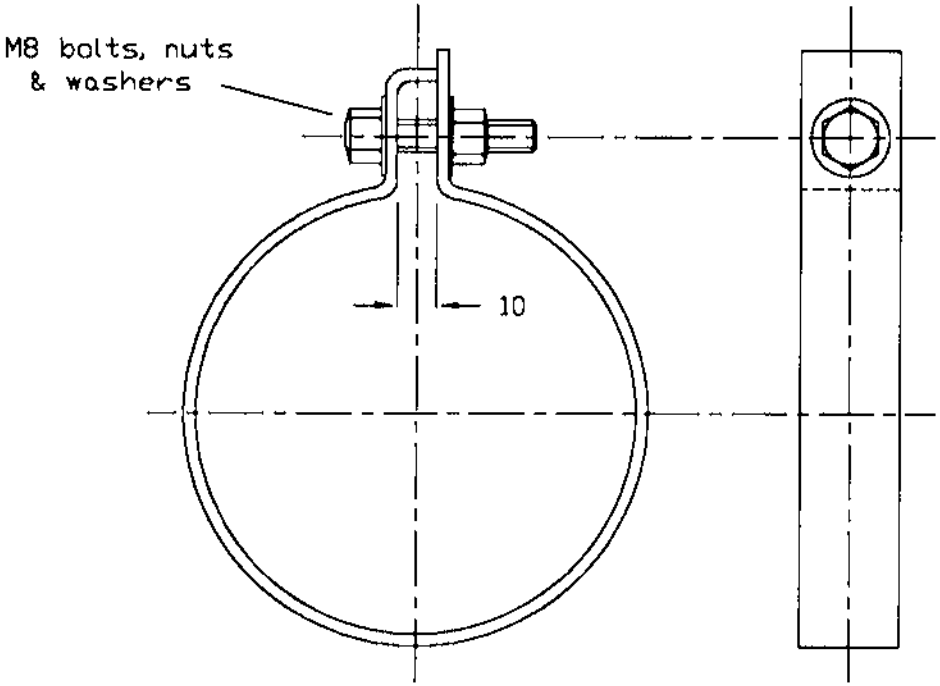
2 - A tank cutter of between 66 and 70mm may be used to cut out the large impulse valve holes.

ALL DIMENSIONS IN mm

DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
P90 IMPULSE VALVE
Drawing No. 5 of 12 NOT TO SCALE



ASSEMBLED IMPULSE VALVE STRAP



ALL DIMENSIONS IN mm

NOTES:

1 - The material used to make the strap is 25 x 3mm mild steel.

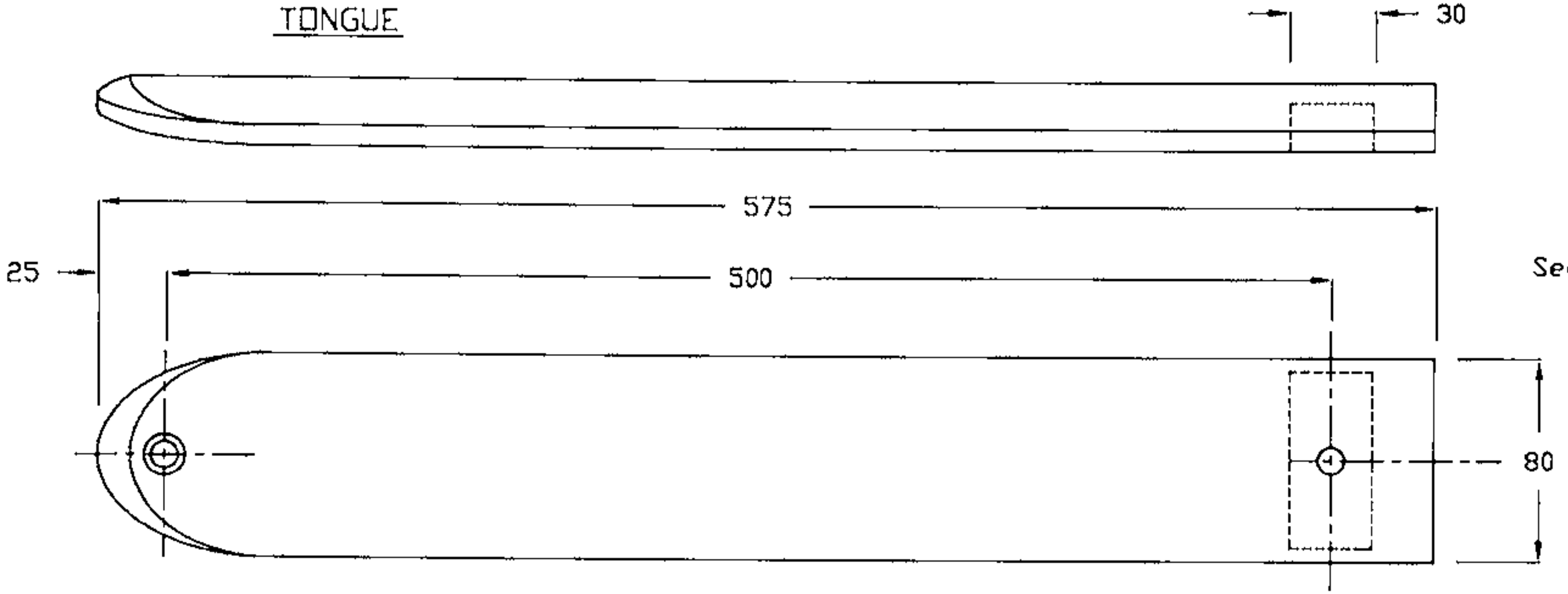
2 - 5 straps are needed for the impulse valve.

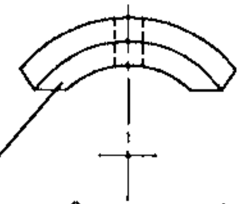
3 - Mark the measured fold lines on the cut lengths of steel. After folding the straps wrap them around the impulse valve and check that there is a minimum gap of about 10mm between the folded up-rights.

4 - When the straps are assembled, paint them before adding them to the impulse valve.

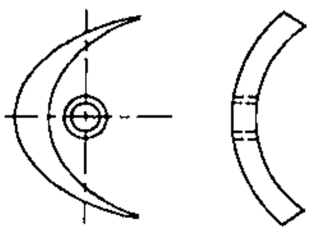
DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
P90 IMPULSE VALVE STRAP
Drawing No. 6 of 12 NOT TO SCALE

TONGUE

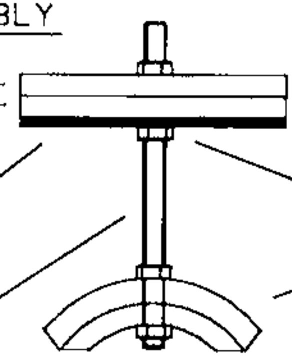




TONGUE MOUNTING



TONGUE STEM ASSEMBLY



A steel ring with an internal diameter of approx. 10mm (3/8' pipe) is pushed into a hole in the tongue using an interference fit. Use a 50mm M8 bolt, 2 nuts and washers to fit the tongue to the impulse valve.

FITTING THE TONGUE

Hold the tongue in place inside the impulse valve. Feed the bolt through a washer and through the impulse valve and tongue mounting holes. Hold the tongue in the 'full-open' position, add another washer onto the bolt and screw on a nut until it just touches the tongue. Add a second nut and tighten the two nuts securely together.

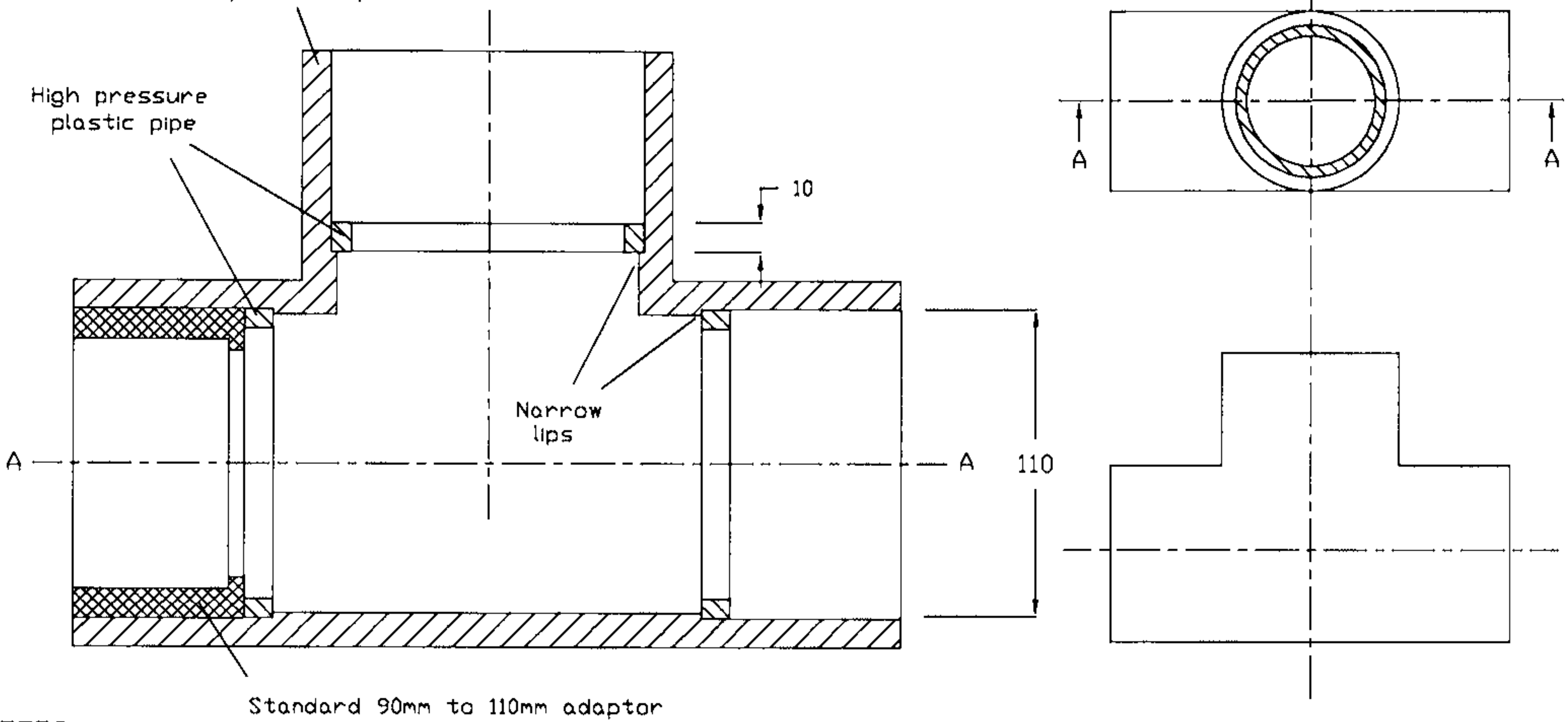
ALL DIMENSIONS IN mm

DEVELOPMENT TECHNOLOGY UNIT
University of Warwick, Coventry, UK

P90 IMPULSE VALVE TONGUE & ASSEMBLY

Drawing No. 7 of 12 NOT TO SCALE

STANDARD T-PIECE



NOTES:

1 - The T-piece for the P90 may be bought commercially or made by hand. The drawing above shows an example of a commercial T-piece (high pressure plastic) that is suitable for 110mm diameter pipe.

2 - T-pieces available on the market tend to have narrow support lips as shown above. These need to be reinforced and this is done by gluing into place a 10mm length of high pressure plastic pipe as shown.

3 - When purchasing a 110mm high pressure plastic T-piece, also buy a 90mm to 110mm adaptor. This is because the drive pipe for the pump has a 90mm diameter and will have to be adapted to fit the T-piece.

ALL DIMENSIONS IN mm

DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
P90 - USING A STANDARD T-PIECE & ADAPTOR
Drawing No. 8 of 12 NOT TO SCALE

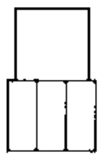
MAKING A T-PIECE

PART A

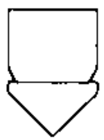
Cut 150 & 400mm lengths of pipe. Cut the shorter pipe down its length and heat it until workable. Open the heated shorter pipe and fit it over the center of the longer section using glue and clamp it in place.

Using the T-piece former, form sockets at each end of PART A. Next, cut out a 90 degree 'V-Notch as shown. Make sure the gap on the shorter outter pipe is opposite the @V-Notch'.

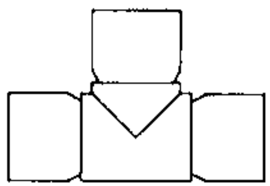
PART B



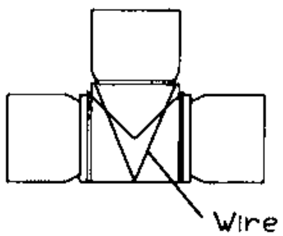
Cut 75 & 200mm lengths of pipe. Cut, heat & glue the shorter pipe flush with one end of the longer. Cut a piece of pipe to fit the gap left on the outer pipe. Heat & glue this into place.



Form the socket on the end of PART B and cut the other end to fit the 'V-Notch' on PART A.



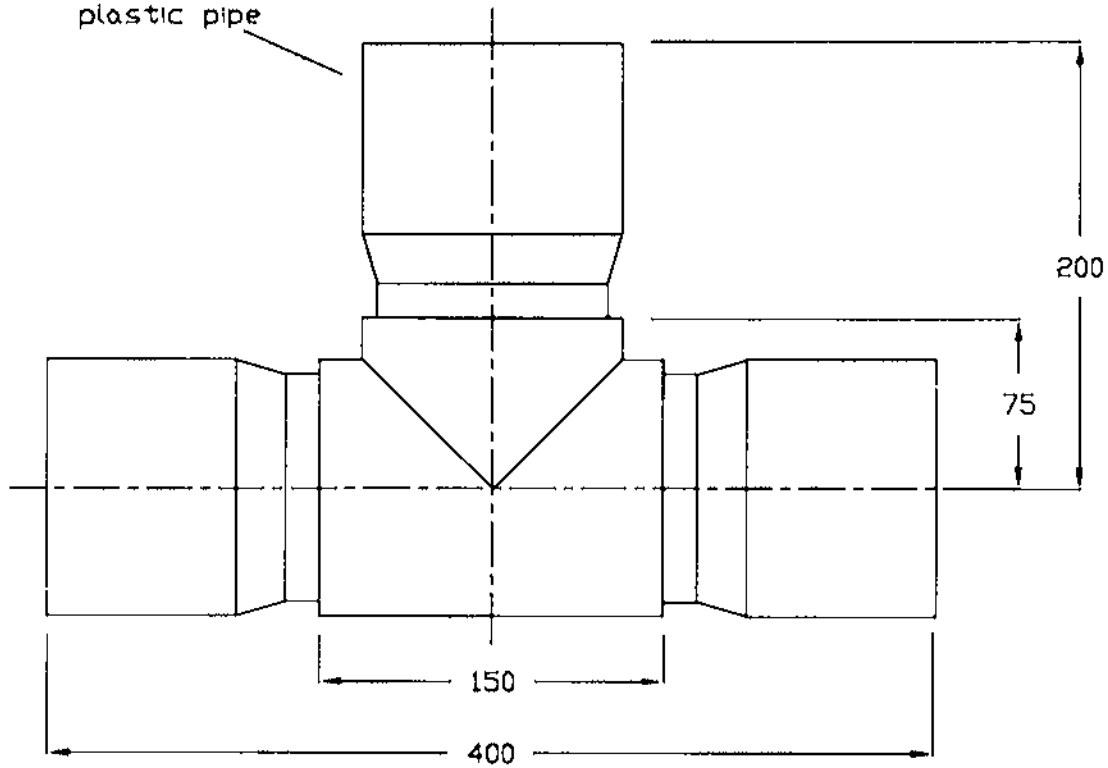
File both PARTS A & B until a reasonable fit is obtained. Glue together both parts using a mixture of PVC glue and pipe shavings to fill the gaps.



Wire may be wrapped around the glued parts to hold them more securely together. The T-piece is now ready to be set into a concrete base.

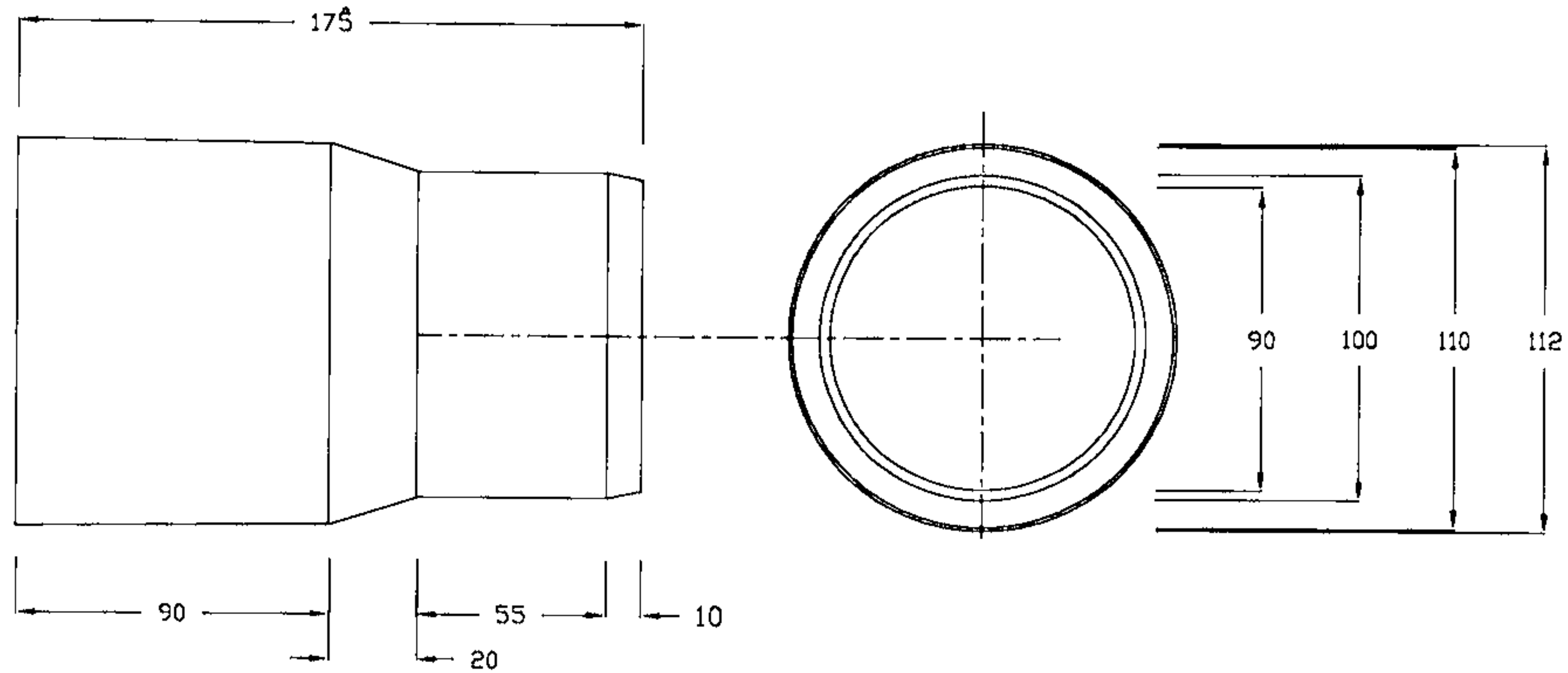
ALL DIMENSIONS IN mm

COMPLETED T-PIECE



NOTE: The drive pipe for the P90 has a diameter of 90mm. Therefore, to fit the drive pipe into this T-piece, a 100mm to 90mm adaptor should be purchased or made up using available pipe.

DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
P90 T-PIECE MANUFACTURE
Drawing No. 9 of 12 NOT TO SCALE



NOTES:

1 - This socket former is designed to be used with 110mm outside diameter plastic pipe.
2 - This former may be made from steel, aluminium or wood.
3 - The former is used to to make both the pump T-piece and End cap.

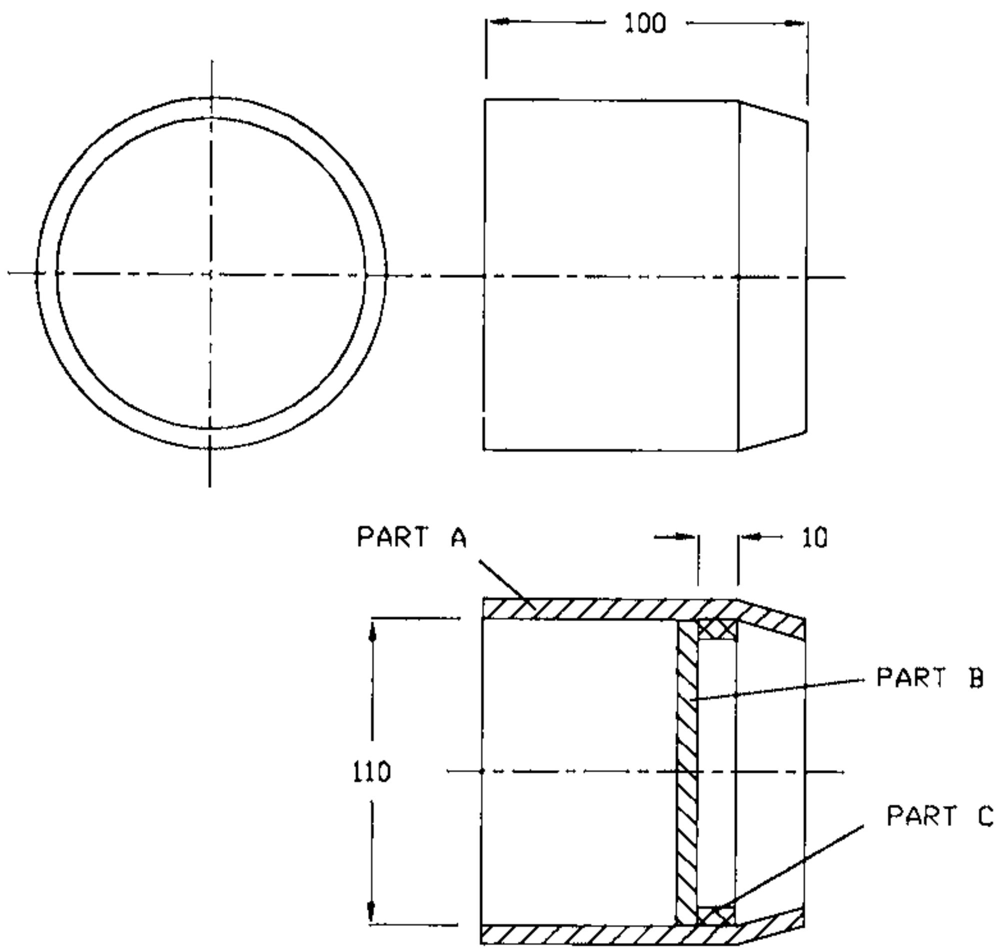
INSTRUCTIONS FOR USE

1 - The first step is to heat up the plastic pipe to make it soft and workable. This may be done by immersing the pipe in hot oil at about 130 C or by rotating the pipe slowly over a heat source.
2 - When the pipe is soft enough push it quickly and firmly over the former.
3 - Let the pipe cool and harden before removing it from the former.

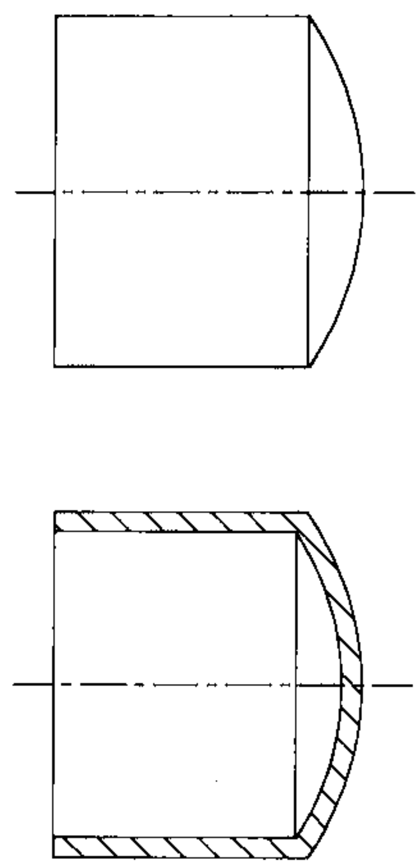
ALL DIMENSIONS IN mm

DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
P90 SOCKET FORMER Drawing No. 10 of 12 NOT TO SCALE

MAKING AN END CAP



STANDARD END CAP



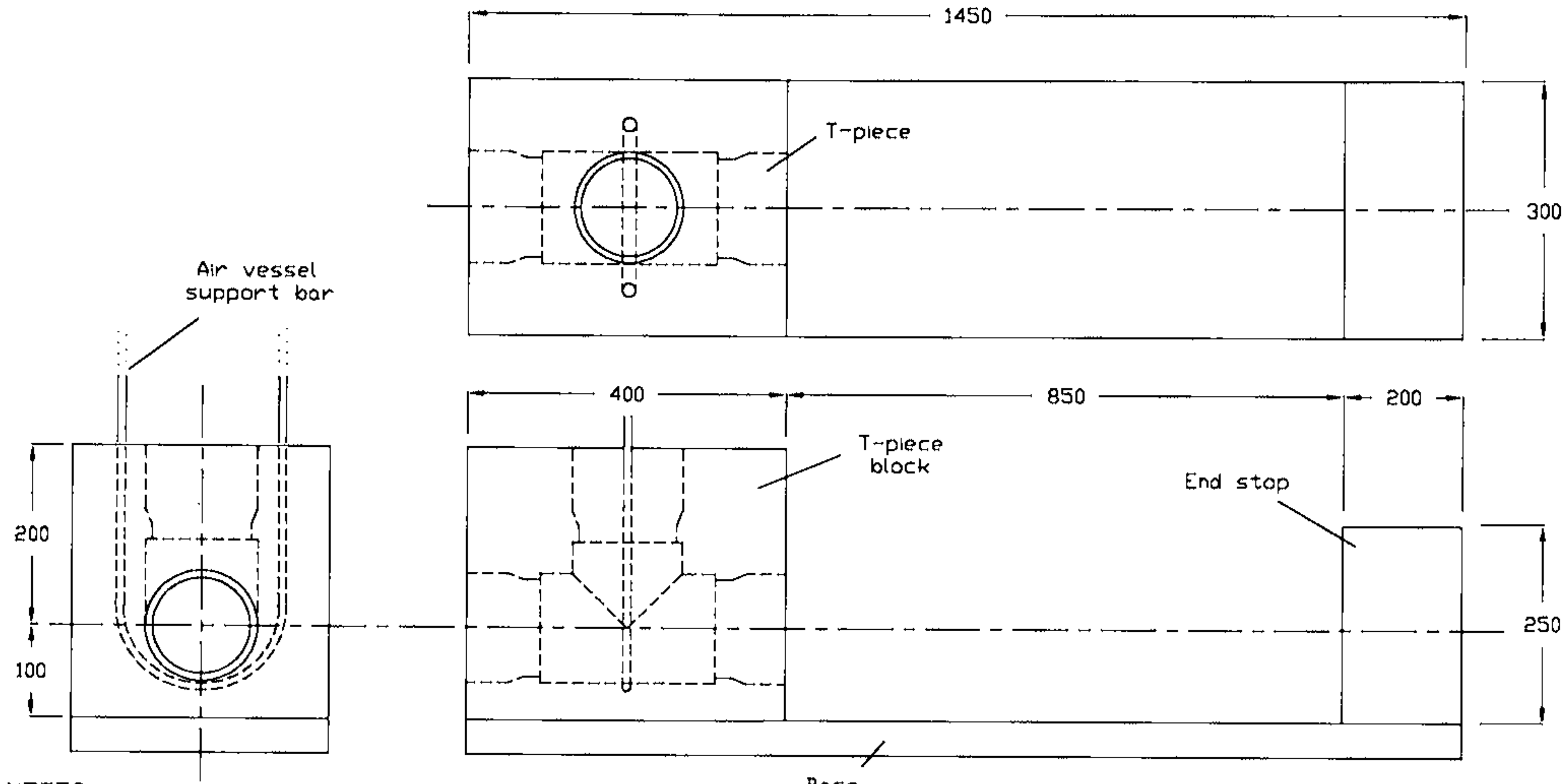
NOTE: The End cap may be purchased commercially or made as shown.

MANUFACTURE INSTRUCTIONS

1 - Cut a 100mm length of high pressure plastic pipe and heat it as per the socket former instructions.
2 - Push the softened pipe quickly and firmly over the socket former to shape PART A.
3 - PART B, the end cap plug, and PART C, the reinforcing strip are made by the same methods used to make the Air vessel.
4 - When PART A has cooled, use a good plastic adhesive to glue in the reinforcing strip followed by the end cap disc. When dry the end is ready for use.

ALL DIMENSIONS IN mm

DEVELOPMENT TECHNOLOGY UNIT University of Warwick, Coventry, UK
P90 IMPULSE VALVE END CAP
Drawing No. 11 of 12 NOT TO SCALE



NOTES:

1 - The height and length of the T-piece block will be shorter when a standard T-piece is used. If a standard T-piece is used make sure the bottom of the T-piece is about 50mm above the top of the base.

2 - The depth of the base should be at least 75mm. The depth may be greater if a more solid foundation is needed.

3 - Use steel reinforcing in the base, T-piece block and end stop to join the three sections together and to reinforce each section. A local builder will know how to do this.

ALL DIMENSIONS IN mm

DEVELOPMENT TECHNOLOGY UNIT
University of Warwick, Coventry, UK

P90 REINFORCED CONCRETE
BASE, T-PIECE BLOCK
& END STOP

Drawing No. 12 of 12   NOT TO SCALE

Hydraulic Ram Pump
Research Programme

NEW DEVELOPMENTS IN HYDRAULIC RAM PUMPING

Technical Release 13

1996

Development Technology Unit
School of Engineering, University of Warwick

This technical release has been written more for ram pump enthusiasts, researchers and manufacturers than for installers and users. It describes the main current trends in system and pump design.

1 GENERAL TRENDS

The ram pump is a 'mature' technology. Over the last two centuries pump designs have stabilised and many variations to the basic configuration (of drive pipe, pump, pump house and delivery pipe) have been tried. One might think that no further significant change was likely in the ram pump itself or in the system in which it is used. However there are changes occurring in both pumping needs and in materials.

Before the invention of petrol engines or the arrival of electricity on farms, the ram pump was in many locations the only feasible way of lifting water from streams or springs to neighbouring hillsides. In consequence a high cost was tolerated; strong but expensive pumps made from cast steel, gunmetal and brass were used. Today there are more alternatives, so that ram pumping can only hold its 'market share' in water supply for humans and for cattle by becoming cheaper and simpler.

All over the world water is getting scarcer and dirtier. In consequence ideal sites for rarn pumping - where a large flow of clean water drops steeply - are becoming fewer. Quite often the water requires cleaning if it is to be used for domestic purposes. There are various possible responses to this problem of polluted drive flow. One is to filter the delivery flow. A second is to use an indirect ram pump that permits falling dirty water to power the raising of clean water from a nearby source. A third is to concentrate on applications like cattle watering and irrigation where water quality is less important.

Filtering and disinfection are well understood, and the technical options for applying them are increasing in number. The availability of only one or two watts of electricity, say from a small photo-voltaic panel, now enables chemical or ultra-violet sterilisation to be performed at a household or village scale. Adding such processes to a ram pumping system may require other design adjustments, for example those to permit delivery flow only in day light hours.

Indirect pumping is a technique known for a hundred years or more. Indirect pumps are still manufactured but they are complex and hence costly. They have more wearing parts than normal ram pumps and they require a source of clean water close to the dirtier flow that drives them. One might argue that to require such elaboration in system installation and maintenance is to head in the wrong direction. Field experience suggests that the use of ram pump technology is already severely limited by people thinking it is 'too complicated'.

It is the authors' experience, mostly in an African context, that even after a 3 weeks' training course many water technicians do not have the confidence to survey, design and install a ram pump system. The design rules seem complex and they fear making any mistake that might cause a system to fail. Yet systems do occasionally fail - through wear and corrosion, insufficient drive flow or flood damage, siltation or blockage, theft or malicious damage. It is not possible to build a perfect system.

With petrol-engine pumping at its simplest, the user carries the pump to site, drops a suction hose into the water source, rolls out the delivery hose and starts the pump. With electric powered pumping using mains, photo-voltaics or transported batteries, the procedure is a little more complex. Ram pumping is more complex again. There has therefore been a growing interest in simplifying the technology, especially in order to serve irrigation operated by peasant farmers.

2 SIMPLER PUMPS

A pump normally comprises an adjustable impulse valve, a (non-return) delivery valve, a pressure vessel to smooth out the pulsating delivery flow and an anchorage or cradle. Where 'free' air is the buffering medium in the pressure vessel (which must be vertical to work properly), a third ('snifter') valve is needed to replenish this air.

Simplifications can take a number of forms, but the main ones of current interest are

- removing the mechanism to adjust the drive flow,
- replacing 'free' air by 'contained' air,
- simplifying the anchorage of the pump and its attachment to drive pipe and delivery.

Removing the tuning mechanism of course removes all the benefits of tuning, namely the ability to adjust the pump to match the drive flow locally available. Under some circumstances, especially when only a small fraction of stream flow is needed, there is no great merit in being able to tune. Where a manufacturer produces a range of pumps it is normal for each step up in size to correspond to a two or three fold increase in maximum drive flow. An untuned pump is effectively permanently set to its maximum (or 'rated') drive flow. Thus using such pumps singly will restrict the drive flow, and hence delivery, to one of a few widely spaced values. If however several (say three) identical pumps, or two pumps of different size, are used in parallel, it is usually possible to get within 25% of any ideal drive flow. In fact there are four distinct alternatives to on-site tuning for matching pumps to available flow, two of them applicable when the system is installed and two when it is in use.

During *installation* the drive flow capacity of a system can be roughly selected by choosing the right number and size of pumps to be run in parallel. Alternatively pump(s) can be used that are 'preset' to a particular drive flow. This lower level of adjustability not only simplifies pump design (e.g. it can be provided by having two or three different weights of impulse valve), but removes the possibility that the user completely mistunes his pump. Such mistuning through operator ignorance is quite common in high technology systems as well as in the simple ram pump ones we are discussing here.

During *operation*, there may be a need to respond to a fall in available stream flow. If pumps are not tunable this can only be done by reducing the number of pumps in operation or by running them all intermittently. Using the 'three same size pumps' or 'two different size pumps' arrangement recommended above, it is possible to follow any changes in stream flow by changing the number of pumps in use. Intermittent operation by contrast can be used with only a single pump but requires much more operator activity and also a reservoir capable of storing at least 2 hours drive flow. In practice intermittent operation, where the user turns on the pump when the reservoir is full and off when it is empty, is very rare. It could become more common where small-farm irrigation is the pumping application. Technically it should also be possible to use a self-priming siphon to achieve intermittent operation without human intervention: the authors know of no example of this being done.

Given the desirability of having more than one pump running in parallel for reliability reasons, the relative rarity of requiring very close matching of system drive flow to stream flow (often extremely variable) and the likelihood of mistuning by inexperienced operators - we may expect to see more simple pumps that are untunable or are tuned ('preset') only during manufacture.

Using "contained " air to buffer the pulsations in delivery flow has real advantages over using a conventional air vessel. By "contained" air or "air packet" we mean air in a bladder or closed-cell foam. Normal commercial pressure-surge limiters in water pipelines use diaphragms to separate the air from the water, however such diaphragms are difficult to make and to seal and are therefore expensive. By contrast closed-cell foam such as bubble-wrap has already been used in a number of small ram pumps. The advantages of substituting air packets for the free air of a conventional pressure vessel are several. The containing chamber need no longer be vertical, air cannot be lost through tiny holes in welds or fittings, the snifter valve is no longer necessary, the pump can be operated under water. Disadvantages are the possible fatigue failure of the air-containment materials, slow loss of air through the walls of bladders or foam and the significant reduction in air volume at start up.

Consider a conventional air vessel of volume 10 litres in a pump delivering to 90 meters. Initially, before start-up, the air is at atmospheric pressure (1 bar). At start-up the absolute pressure rises rapidly to 10 bar (9 bar 'gauge') as the air is warmed and compressed. It then cools until its volume is about 1 1itre, namely one tenth of its initial value: the air vessel is now nearly full of water. Over a period of hours however the air is replenished via the snifter valve to its original volume of 10 litres. The pump may run rather noisily until this has taken place.

If however a closed air packet replaces the conventional free air, there is no replenishment mechanism, so throughout the run time it remains at 1 litre. It therefore is necessary to provide an air packet whose initial volume is equal to:

V_{init} = air volume required in operation (V_{op}) $\times$ delivery pressure in bars absolute.

(Note that 10 meters delivery head corresponds to 2 bars absolute, 20 meters to 3 bars etc.)

Recent research and experimentation suggest that the air volume in operation (V_{op}) can safely be as little as twice the volume of water delivered per cycle. [The pump efficiency does not fall significantly compared with when V_{op} is large, and the overpressure of about 30% is usually also tolerable from a fatigue point of view.] An irrigation pump may only lift to 20 meters, so initial air volume V_{init}, is only 3 times V_{op}, whereas a domestic supply pump may lift to 80 meters (V_{init} = 9 times V_{op}) or higher. We would therefore expect this air compression problem to be more severe with high-lift pumps. However as the delivery head is increased (while the drive head and drive flow are kept constant) the volume delivered per cycle goes down. The combination of these effects means that for a given size of pump, the appropriate initial air packet size does not vary much with delivery head. In practical terms, the minimum initial packet size relates to pump size roughly as shown in Table 1:

This table indicates an initial air packet volume, and therefore vessel size, equivalent to 1 m of drive pipe (or less length of a larger diameter) should be sufficient: this is a tolerable size.

With an air packet, pump design can be simplified to essentially an impulse valve followed by a packet-enclosing horizontal tube entered via the non-return delivery valve. This results in a fairly compact design of pump that can be placed under water to maximise drive head and to reduce noise. Although there are some particular problems that can arise when operating under water – for example sucking debris in through the impulse valve, increased vulnerability of flood damages and difficulty of access for tuning – in many situations the advantages outweigh the disadvantages.

As materials further improve we may expect more ram pumps to incorporate air packets or even a diaphragm instead of traditional air vessels.

Simplifying the pump attachment is a particular requirement for irrigation use where ram pumps and even drive pipes may be removed at night and will certainly need to be removed at the end of the dry season. The shock forces on pumps when they are in use are large, so any anchorage has to be sturdy. Already it is usual to bolt pumps onto a permanent (i.e. concreted-in) cradle. There is now interest in providing clip-on arrangements both between pump and cradle and between pump and drive pipe, rather than using nuts, bolts or wedges.

3 NEW MATERIALS, LOWER COSTS AND HIGHER PERFORMANCE

Materials For long-life pumps, traditional construction materials are largely suitable. By contrast, new materials have particularly found their place in cheap ram pumps of modest but adequate performance. During the last twenty years metal piping has largely been superseded by plastic, especially PVC, ABS and HDPE. It is therefore

Table 1

Drive pipe size (ID)	mm	25	50	75	100
Assumed driveflow	litres/min	25	100	250	500
Packet size (for *drive* head of 2 meters)	litres	0.15	0.60	1.50	3
of 6 meters)	litres	0.45	1.80	4.50	9
Volume of I meter of drive pipe for comparison	*litres*	*0.5*	*2.0*	*4.4*	*8*

tempting to use these rust-proof and easily worked materials for constructing ram pump bodies. Unfortunately the poor stiffness, fatigue strength and sunlight resistance of plastics poses problems.

The water-hammer effect that underlies ram pump operation is dissipated in very elastic, or worse energy absorbing, materials. For this reason we try to avoid accumulation of air in drive pipes and we look for a high level of wall stiffness in them. The maximum height a ram pump can deliver to is approximately $h_{max}=\nu\ C_{dp}/g$. Where ν is the maximum water velocity in the drive pipe and C_{dp} is the velocity of sound in that water. It can be shown that for an infinitely stiff pipe, C_{dp}/g is about 140 meters height per meter/second, in a steel pipe it is typically 120 but in a plastic pipe it is only about 30.

[The formula normally used is:

$$C_{dp} = C\sqrt{\frac{1}{1+\frac{DG}{tE}}}$$

Where C is the velocity of sound in water, D and t the diameter and thickness of the plastic pipe, G the stiffness of water and E the stiffness of the plastic.]

This effect shows itself in a plastic system being only able to deliver to about 30% the height of an all-steel system. For really high head deliveries steel drive pipe is essential. For delivery height under 50 meters, plastic drive pipe is adequate

All materials show 'fatigue' in that a loading that they can tolerate easily if it is applied only a few times may cause failure if applied millions of times. In a ram pumping system, the pump and drive pipe experience between 15 million and 100 million pressure pulses per year, so fatigue failure is a real danger. For plastic drive pipes it is usually sufficient to select a pipe pressure rating of 3 times the delivery pressure. For plastic pumps, fatigue failure is so likely that either they are metal reinforced or they are restricted to use with very low delivery heads. Apparently no one is making pumps out of glass reinforced plastics (GRP) despite this material having suitable stiffness and fatigue performance.

Injection moulded plastics are used in centrifugal pumps and hand pumps. They could be used for ram pumps too, but the small production runs do not at present justify the high tooling costs. A few experimental pump bodies have even been made of concrete whose inertia may act as a substitute for strength in the face of sudden forces. The material is cheap, though heavy, but the problems of getting really high densities and of sealing the joints between concrete sections have apparently defeated concrete pump designers.

Certainly the use of simple plastics in small or low-lift ram pumps is now well established alongside that of metals for higher lifts. It seems unlikely that more complex materials or processes will be soon employed to make these devices.

Lower costs come from use of fewer, cheaper or 'easier' materials, from mass production and from design simplifications. Understanding of ram pumps is better than in the past and this had led to a few design changes leading to lower costs.

Mass production of complete pumps is constrained by small markets, while attempts to assemble the pumps from mass-produced fittings have not generally led to either high performance or to much lower costs. Fittings are not cheap if used in any number: pumps made from them are generally clumsy and have too many parts.

For fabricated steel pumps the now ready availability of square section tubing offers simplification of design and assembly compared with traditional round tubing. Square tubing is not efficient at containing high pressures but this is not normally a problem at all but the highest delivery heads. Fabrication is better suited to some pump-using countries than employing iron casting, machining, forging or threaded connectors. Welded joints can be opened again if necessary by cutting them out with an angle grinder

Probably the greatest need for cost cutting is in irrigation applications. If a siphon drive pipe could be developed (requiring the pump to be submerged), the installation costs of irrigation pumps could be reduced substantially. Some effort is being applied to designing essentially portable systems for use with dams as low as 500 millimeters, where the pump and its drive pipe can be quickly disconnected from anchorage and dam respectively.

High performance takes various forms, such as higher efficiency, higher delivery head, quieter operation and greater durability. It seems that little theory was used in the past when designing either pumps, or complete systems. Today ram pumps have something of a fascination for analysts so that there are several publications that aid high-performance design. For example the main sources of inefficiency are well documented and it is not hard to devise an economical system with an overall efficiency as high as 70%.

In Nepal, the Andes, Rwanda and elsewhere there is some need for pumps that lift as high as 200 meters, well beyond the limit of normal machines. The procedures and materials for achieving very high heads are known, but so far the market for such pumps has been too small to cover the costs of fully developing them. DCS, Butwal in the Himalayas have reached 180 meters lift with some reliability.

Quiet operation has been traditionally achieved by placing pumps and drive pipes underground. For years some pumps (for example the Blake's machines) have used rubber impulse valves in otherwise metal systems to reduce noise. The move towards plastic drive pipes may lower efficiency a little, but it beneficially converts high-frequency 'clanging' into less intrusive low-frequency 'thumping'.

Only in the area of durability can one find no significant improvement. Perhaps the lifespan of cheap pumps has increased a little from its former low level, but it is still far below that achievable with traditional 'over-designed' machines.

DTU

Ram Pump Programme

DTU S2 PUMP

TECHNICAL
14
RELEASE

DTU TECHNICAL RELEASE NO.14 : THE DTU S2 PUMP

INTRODUCTION

This Technical Note is in four parts.

The first part (1 page) is a 'Stop Press' UPDATE containing amendments (dated November 1998) to the original which was written in 1995.

The second part (4 pages) is a summary of the pump's design and performance, suitable for copying, laminating and posting in the pump house.

The third part (20 pages numbered 1-20) is a detailed description of how to use the drawings to make an S2 pump. It should be read in conjunction with the amendments mentioned above.

The fourth part is a set of 16 drawings, of which Nos. 11a, 13, 14 and 14a have been added as amendments and offer alternatives to earlier drawings.

The 'Status' of this pump is that several dozen have been made, mostly in Africa, but that none has been run continuously for years on end. The pump is not a 'very-long-life' design although, in normal applications and incorporating the amendments in the UPDATE, continuous use should require only occasional replacement of worn parts. The Drawings deliberately do not precisely define the materials to be used, because the design is supposed to be somewhat adaptable to what is available. If all the metal parts are of mild steel, corrosion will cause some problems - especially affecting nuts and bolts and the impulse valve stem. If possible these parts should be made of stainless steel.

It is not the intention of the Development Technology Unit to undertake further development of this pump. However the Unit's Director would be happy to receive any comments from the field about its performance or any suggestions for further 'UPDATES'.

Development Technology Unit, School of Engineering, Warwick University,
Coventry, CV4 7AL, UK
tel +44 1203 523122, fax 418922, email at dtu@eng.warwick.ac.uk,
web at http://www.eng.warwick.ac.uk/dtu

TR14 UPDATE - November 1998

Since this Technical Release was written in 1995, based mainly on field experience in Zimbabwe and Zaire, the DTU has obtained further experience with this S2 model of ram pump. The following modifications derive in particular from observations during extended testing in Staffordshire, England during Summer 1998. The modifications are expressed as variants on the production procedure of the (original) pages that follow. Extra drawings have been added (Dwgs. 11A, 13, 14 and 14A).

1) [*Problem of steel washer cutting into delivery valve rubber disc*]
   Refer to page 14 - Making the S2 pump delivery valve.

Fit a rubber washer, made from the same material as the valve disc and of the same diameter as the steel washer, between the steel washer and delivery valve disc.

2) [*Wear and tilting wobble of impulse valve stem*]

Refer to page 12 - Making the S2 pump impulse valve - and to old Drgs Nos.9, 11a & 12 and new Drawing No.11a.

Fit a second (upper) stage to the impulse valve guide. This requires that a longer valve stem and longer guide bolts are made/used than those shown in drawing number 12. The plain section of the valve stem is 50 mm longer (i.e. 100 mm overall length), the bolt shank is 50 mm longer (i.e. 130 mm long).

If a *new* two stage guide is to be used then the assembly should be welded together *before* the holes are drilled. However if the second stage of the guide is fitted *as an addition to/modification of* an existing impulse valve, then (to ensure correct location and operation of the valve itself) any welding should only be undertaken *after* the impulse valve is fully assembled and bolted to the pump body, as detailed on pages 17 to 18.

3) [*Problem of delivery valve gasket blowing out midway between the clamping bolts*]

Refer to old Drawing No.3 & new Drawing No.13

New gasket size, internal diameter = 95 mm, outside diameter = 165 mm. Using the internal diameter of the gasket shown in Dwg.13, which is a little smaller than that of the flange in Drg. 3, ensures that it protrudes slightly inside the flange. This wider gasket is primarily required between the air vessel and delivery valve plate, but can also be used between delivery plate and pump body and in the impulse valve.

4) [*Problem of rusting bolts*]

Thoroughly grease all bolts prior to assembly and use nuts or washer/nut combinations that extend to the end of the bolt threads, leaving almost no thread exposed. Of course if stainless steel bolts and nuts are available and affordable, they should be used in preference to plain steel.

5) [*Problems in aligning pump with drive pipe*]

If possible, the drive pipe, pump and cradle should be assembled in situ within the shuttering for the concrete base *prior* to any concrete being poured

6) [*Problems with marking out delivery valve plate for drilling*]

Refer to old Dwg. Nos. 5, 6 & 7 and new Drgs. Nos. 14 & 14a.

An alternative layout of holes in the delivery valve plate is to have them in parallel staggered rows as shown in Drawing number 14. The delivery valve rubber disc now flexes as a butterfly's wings and has to be retained by a bar or rod held in position by two bolts (rather than the original single central bolt) - see Drawing 14A. Slightly inferior pump output may be experienced (due to more back-leakage before the delivery valve recloses) but this layout is easier to mark out and make, so there is less chance of having too thin a wall between adjacent holes. As before, care should be taken to not so tighten the bar-retaining bolts that the valve rubber disc no longer lies flat on the valve plate. The nuts on these bolts should if possible be self-locking or held on with a locking compound such as 'Locktight'. The alternative of using a second locknut on each bolt is not so good.

7) [*Problems with silting up of intake pool*]

Ram pumps driven by water drawn from immediately below springs, or from large reservoirs, have little problem with silting. Pumps drawing water from behind small dams (e.g. 400mm high) on streams may experience major silt problems, with the 'pool' silting up by as much as 300mm after a single storm. Drawing all drive water through silt will almost certainly overload the silt-settling capacity of the drive tank. There is no easy way of automatically keeping such a pool silt-free and such locations are therefore not suitable for ram pumps unless someone is on site to apply frequent manual desilting procedures to both pool and drive tank.

DTU S2

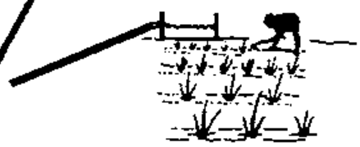
hydraulic ram pump

The name "S2" stands for a Steel pump with a drive pipe up to 2" in diameter.
The S2 replaces the DTU M8 pump

The DTU S2 hydraulic ram pump is a steel machine, using a 1.5" or 2" diameter galvanised drive pipe, that can lift water up to a height of 100 meters. It was designed for village water supply but may also be used for irrigation, and is being used successfully in many African countries. The pump has been designed to be made in small workshops with welding equipment and a pillar drill. A lathe can be useful but is not essential.

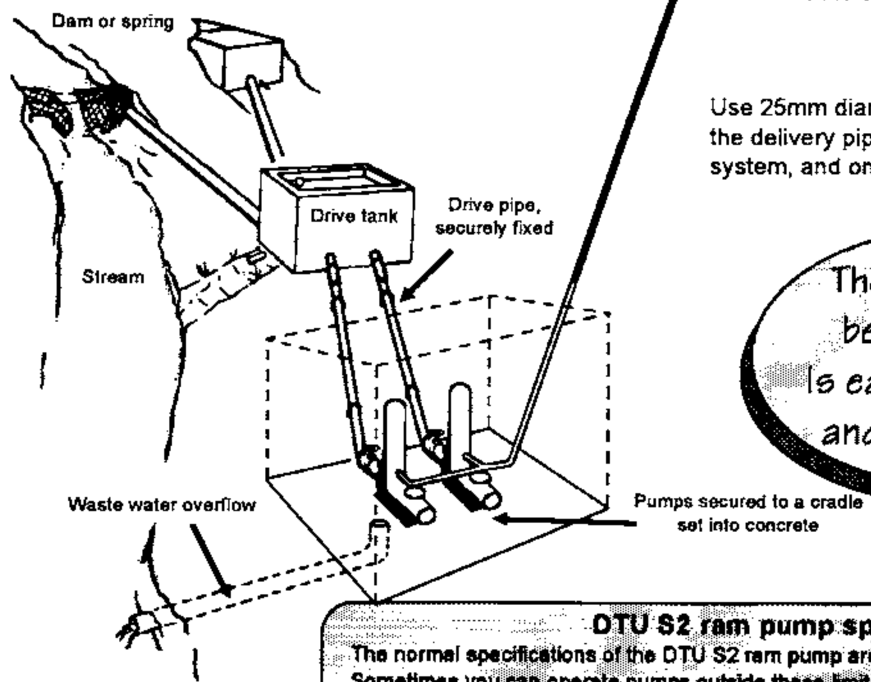
A ram pump is powered by falling water. Water from a stream or spring is diverted and dropped through a drive pipe into the pump. The power of the falling water is used to pump some of the water where it is wanted. The amount of power in the falling water limits how high you can pump, and how much water you can pump. Generally, the more water you drop and the further you let it fall, the more power there will be.

In areas where the water source flow varies greatly during the year, more than one pump can be installed, all sharing the same delivery pipe as shown in the drawing below.



Distribution system, for domestic or irrigation use. A tank is always recommended.

Delivery pipe, rising all the way along its length (no ups and downs). The pipe should be buried where possible and protected if it has to be above ground.



Use 25mm diameter plastic pressure pipe for the delivery pipe if there is one pump in the system, and one size larger if there are two.

The DTU S2 can be locally made
Is easy to maintain
and cheap to run!

DTU S2 ram pump specifications

The normal specifications of the DTU S2 ram pump are given here.
Sometimes you can operate pumps outside these limits, but they may not work well.

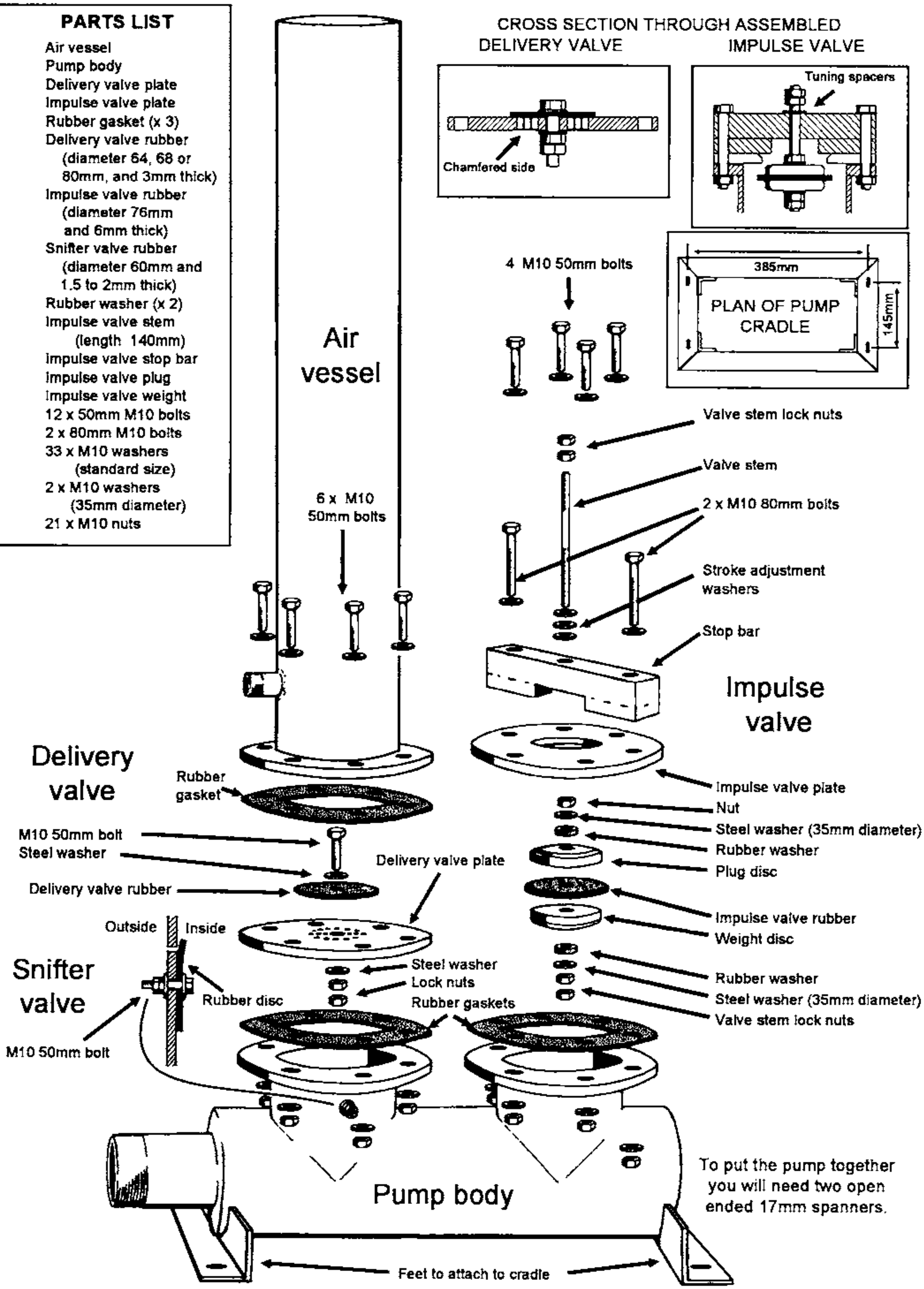
drive head range	—	2 to 15 meters
drive flow range	—	40 to 120 liters a minute
drive pipe material	—	Galvanised iron
drive pipe diameter	—	1.5" for flows from 40 to 60 liters a minute
drive pipe diameter	—	2" for flows from 60 to 120 liters a minue
delivery head range	—	up to 100 meters
typical delivery range	—	1 to 25 liters a minute
delivery pipe diameter	—	25mm

DTU S2 PUMP: USER INSTRUCTIONS

AN EXPLODED VIEW OF THE DTU S2 PUMP

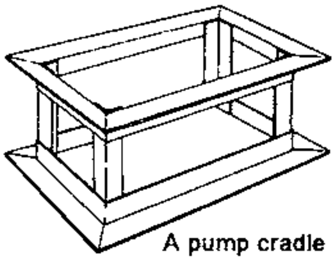
PARTS LIST

- Air vessel
- Pump body
- Delivery valve plate
- Impulse valve plate
- Rubber gasket (x 3)
- Delivery valve rubber (diameter 64, 68 or 80mm, and 3mm thick)
- Impulse valve rubber (diameter 76mm and 6mm thick)
- Snifter valve rubber (diameter 60mm and 1.5 to 2mm thick)
- Rubber washer (x 2)
- Impulse valve stem (length 140mm)
- Impulse valve stop bar
- Impulse valve plug
- Impulse valve weight
- 12 x 50mm M10 bolts
- 2 x 80mm M10 bolts
- 33 x M10 washers (standard size)
- 2 x M10 washers (35mm diameter)
- 21 x M10 nuts



To put the pump together you will need two open ended 17mm spanners.

Installation notes



A pump cradle

The DTU S2 pump should be installed in a properly designed system. To prevent vibration causing breakages, it should be firmly bolted to a steel frame (called a pump cradle) that is half buried in a concrete base. The cradle is usually made from 40 x 40mm angle iron and will vary in size depending on the number of pumps installed. Hole locations for just one pump are shown on the previous page.

All pipes in the system should be supported firmly, and buried where possible. The drive and delivery tanks should be constructed on good foundations by experienced tradesmen. Pipe joints to the drive tank should allow the pipes to move slightly without damaging the tank walls or leaking badly.

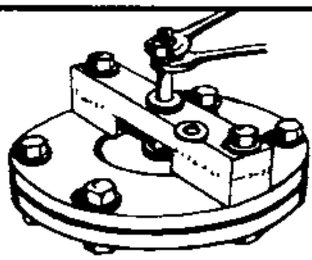
Starting and stopping the ram pump

Although ram pumps often start very easily, they can be awkward the first time they are run.
To start the pump:

1. Make sure that any valve fitted on the delivery pipe is open and then open the drive pipe valve. Water will flow out of the open impulse valve until it suddenly shuts. If it reopens automatically, the pump should continue to run on its own. If it does not, you must prime the delivery system as described in Step 2 alongside.
2. Push down on the top of the impulse valve stem with your foot to reopen it (wear strong boots). Again, water will flow out of the open impulse valve until it suddenly shuts, then push down immediately to re-open the valve. Keep helping the valve to re- open until it will do so by itself.

To stop the pump, hold the impulse valve stem up to close it or shut the valve at the bottom of the drive pipe.

Tuning for best performance



The DTU S2 can be tuned to adjust performance. This is done by changing the up and down movement of the impulse valve, which is usually set to around 15mm. Tuning is usually done to achieve either the maximum delivery flow or the most efficient use of the drive water available.

Maximum delivery

When there is plenty of drive water available, the pump can be tuned to deliver as much water as possible. To do this, remove all washers from the impulse valve stem so that the valve has as much up and down movement as possible (about 20mm).

WARNING: - this also puts the pump parts under greater stress and makes them wear more quickly.

Low drive flow

If the pump uses more water than is available it will soon stop. If this happens it must be tuned to use less. The impulse valve should be tuned down to use 90-95% of the water available from the source. To tune the pump down, add washers onto the impulse valve stem so that the valve has less up and down movement. The shorter the stroke, the smaller the amount of drive flow needed, and the less water is delivered. The minimum stroke length is around 10mm.

Routine maintenance

While the pump is running normally, a visit should be made once a week to check that bolts are tight and that there are no leaks. Once a month an inspection of the whole system should be carried out. It is also recommended that a log book is kept to record the checks and repairs that have been made.

Monthly maintenance check list (without stopping the pump):

1. Inspect all the joints to check for leaks.
2. Check if there is sufficient air in the air vessel. This can be done by listening carefully to the pump. If there is insufficient air in the air vessel, the pump will be much louder than usual. This means that the snifter valve is probably blocked and will need to be cleared. If a bleed screw is fitted, when opening it, air or a small amount of water followed by a rush of air means the air level is OK. If only water emerges, then again the snifter valve will have to be checked for blockage.
3. Clean any filters installed in the system.
4. Remove excess silt or debree from tanks or from behind the intake dam or weir if necessary.
5. Walk along all pipes looking for damage. Also, inspect the tanks for leaks, particularly at pipe joints.

Pump repair

If the pump stops or it delivers less water than usual, it may require adjustment or repair.

Look at the pump and if there is no obvious fault start it again if you can. Watch the pump and listen for irregular pumping or unusual noises. A worn impulse valve, for example, is usually obvious because water squirts through when the valve is closed. Some parts of your ram pump may need occasional replacement. the frequency of this will depend on how hard the pump is working and on the cleanliness of the drive water.

Tools you will need:

2 x 17mm ring/open end spanners to disassemble and assemble the pump

2 x Adjustable wrenches - to loosen a union joint on the delivery pipe (if fitted)

Taking the pump apart

Depending on the fault it may be necessary to disassemble the impulse valve and/or the air vessel. Before attempting to take apart the pump:

1. Make sure that the drive pipe valve is closed and the impulse valve is open. This will allow you to work on the impulse valve ONLY.
2. Depressurise the air vessel.

WARNING - Before attempting to remove the air vessel, always release the pressure in it slowly. An ideal system will have a gate valve or one-way valve and a union fitted between the air vessel and the bottom of the delivery pipe and the optional bleed screw fitted to the air vessel. With the pump stopped, close the gate valve in the delivery pipe to stop it draining back. If a one-way valve is fitted it will close automatically. Then loosen the bleed screw to release the pressure in the air vessel. If none of the above are fitted, the only other way to release the pressure in the air vessel is to loosen each of the air vessel flange bolts one turn at a time until the water and air escapes through the join at the flange. You will certainly get wet this way.

Checks

1. Check the delivery valve rubber for wear and blockage of the valve holes. Check that the lock nuts on the valve bolt are tight.
2. Check that the snifter valve is in good condition.
3. Remove the impulse valve and check the impulse valve rubber and the rubber washers for wear. Check that the nuts on the valve stem are tight and check for excessive wear of the stem. Replace parts if necessary.
4. Check the pump body is firmly bolted down, then reassemble the pump, ensuring that all bolts are greased.

Putting the pump back together

Assembly of the pump is shown in the exploded view drawing, but the following important points need to be kept in mind:

1. Assembling the delivery valve
   Put together the delivery valve plate, the rubber and the bolt. Make sure the side of the plate with the chamfered holes is on the opposite side to the rubber, and that the rubber is on top.
   Screw on the first nut until it is finger tight and then undo it by one turn. Care must be taken not to overtighten the bolt and nuts as this will affect the performance of the valve. Next, screw on the other nut and tighten it up against the first. Use the spanners to tighten them firmly together. This will lock them together, and also allow a small up-and-down movement of the bolt and rubber.
2. Assembling the snifter valve
   Put the 'shaped' bolt and washer together, feed the bolt through the valve rubber, then push this through the pump body. Make sure that the shaped curve of the bolt head and washer align with the curvature of the body.
   Screw on the first nut until it is only finger tight. If the nut is on too tight the rubber will curl away from the pump body and will need to be slackened off slightly. Then screw on the second nut and tighten the two nuts firmly together using the spanners. Then check that the rubber has not distorted. If it has, slacken the nuts half a turn and tighten the outer one again.
3. Assembling the air vessel/delivery valve/rubber gaskets
   Align the delivery valve, air vessel, pump body and rubber gasket mounting holes and feed through the bolts. Make sure the delivery valve is the correct way up (the valve rubber facing upwards) and then tighten the nuts by hand. Use the spanners to tighten each nut and bolt a little at a time, working around the flange. This will draw the assembly together evenly.
4. Assembling the impulse valve
   The first parts to assemble are the valve stem, discs and rubbers. Screw on a nut to the longer threaded end of the stem up to the end of the thread. Push a steel washer on up to the nut, then add a rubber washer. Follow this with the valve plug disc, with the chamfered side towards the nut. Slide the valve rubber up against this, then the weight disk with the chamfered side facing away from the rubber. Follow this with another rubber washer and a steel washer. Screw a nut up them until it is finger tight. Thread on another nut and use the spanners to tighten the nuts together. This part of the assembly is sometimes known as the valve plug.
   Hold the impulse valve plate and the valve plug together, with the chamfered side of the plate opposite to the side against which the valve rubber presses. Slide the stop bar onto the top of the stem and thread a nut loosely on the stem.
   Align the valve assembly, pump body and rubber gasket and feed through the flange bolts. The two longer bolts feed through the cross-bar as well.
   Thread on the six nuts by hand, then use the spanners to tighten the four shorter bolts that hold down the valve plate. Again care must be taken to ensure that these nuts are tightened evenly. The next step is to make sure the closed valve plug is centred in the valve plate hole before tightening down the two remaining nuts that secure the stop bar. To check the alignment, open and close the valve manually turning the valve plug to make sure it does not catch on the hole in the valve plate.

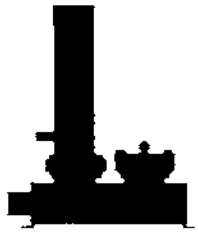
Now you only need to set the stroke length of the valve for the pump to be ready for use.

Spare parts to keep on the site

- impulse, delivery and snifter valve rubbers
- an impulse valve stem
- a few spare M10 nuts, bolts and washers

The Development Technology Unit (DTU)
Department of Engineering
University of Warwick,
Coventry CV4 7AL UK
Tel: 44 (0)1203 523122
FAX: 44 (0)1203 418922

Making a DTU S2 ram pump



When you have decided to make a pump, the first thing you must do is make photocopies of the DTU S2 ram pump design drawings in Technical Release number 14b. The copies can be used in the workshop and it will not matter if they get dirty or are damaged. Some of the drawings can also be used as templates.

It will be useful to have the copies of the design drawings beside you as you work through this Technical Release. The pump's normal operating ranges are on the first drawing.

The manufacture process is presented under the following headings.

- **The parts of the pump**
  This introduces the main parts of the DTU S2 pump.

- **Tools required**
  The DTU S2 pump was designed to be made in small workshops with limited tools. This section describes the tools recommended. We have assumed that you are a workshop technician who already knows how to use the tools.

- **Guides and Templates**
  Simple guides and templates are useful, so we have included descriptions of guides and templates, and suggest ways of making them.

- **Materials needed to make a DTU S2 ram pump**
  A "shopping list" of the materials that you will need is given on page 6. This can be photocopied and taken with you when you buy materials.

- **A step by step guide to manufacture**
  The process of manufacture is described in detail under this heading. It begins with making the pump body and air vessel, and goes on to cover the valves and the pump cradle.

- **Assembly and testing**
  Putting the valves together properly is very important, so the process of assembly is described. The description includes some suggested ways of checking that they work properly.

- **Spare parts**
  This explains which spare parts to supply with a pump. It points out which parts you should expect to wear out or fail.

- **Optional addition**
  The addition of a "bleed" screw in the air vessel can make it easier to find out what is wrong with a pump when it does not perform properly. It can also make the pump easier to maintain. The "bleed" screw is described under this heading.

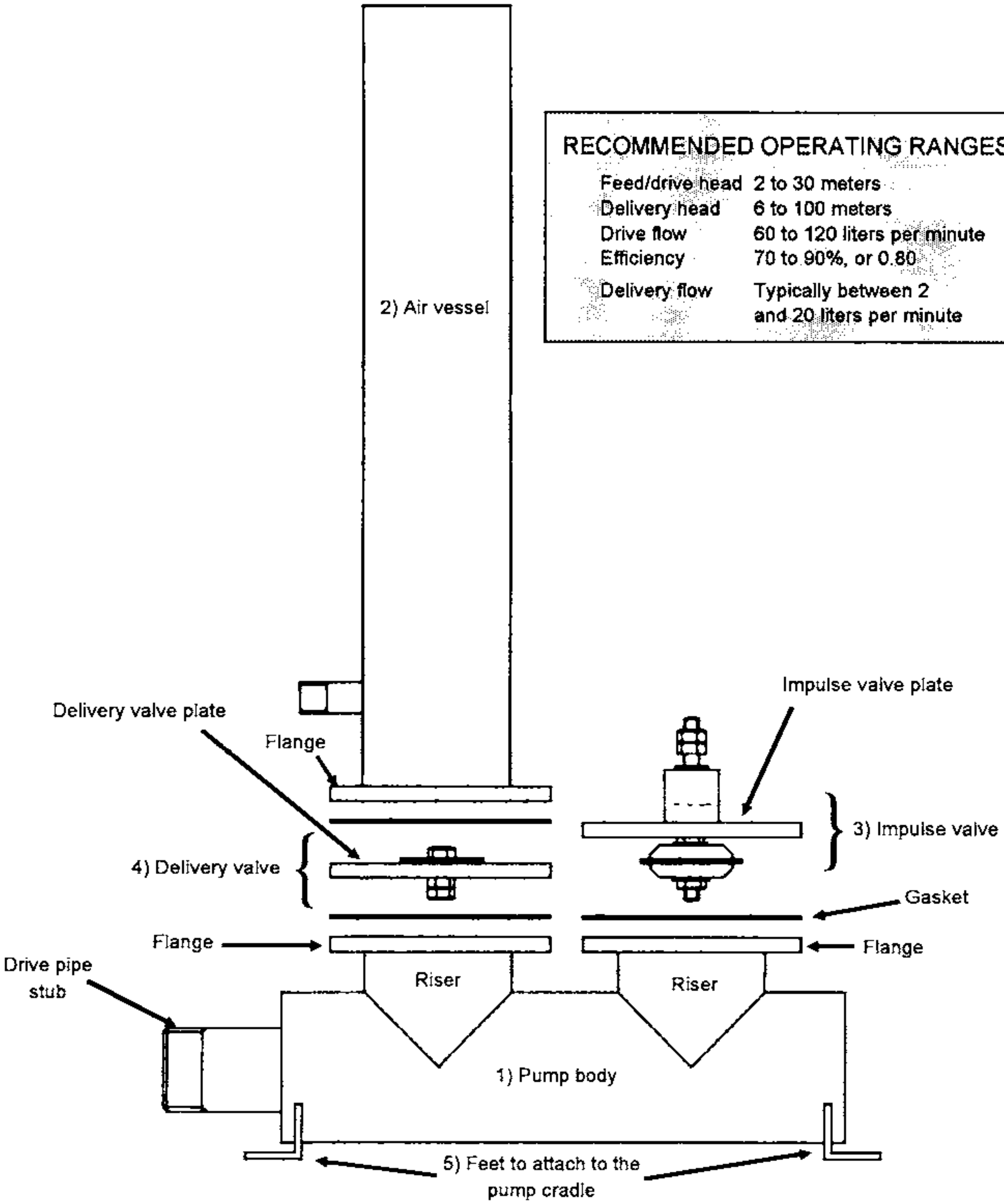
The parts of the pump

The picture here shows the main parts of a DTU S2 ram pump. There are five, apart from the risers and flanges. The pump's recommended operating ranges are given in the box.

1 The pump body
2 The air vessel
3 The impulse valve
4 The delivery valve
5 Feet to attach the pump to the cradle.

RECOMMENDED OPERATING RANGES:

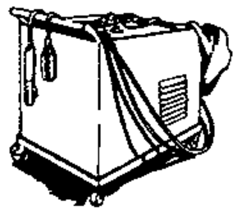
Feed/drive head	2 to 30 meters
Delivery head	6 to 100 meters
Drive flow	60 to 120 liters per minute
Efficiency	70 to 90%, or 0.80
Delivery flow	Typically between 2 and 20 liters per minute



Tools required

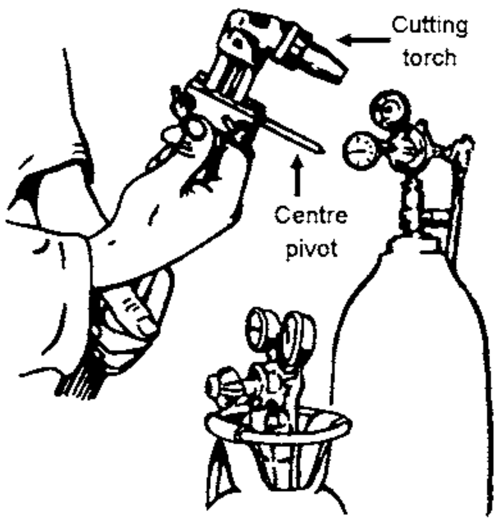
DTU S2 ram pumps can be made in a workshop with welding facilities, a drill, a hacksaw and files. It is hard work making them with as few tools as this. It is also hard to make the pumps to a high enough standard. Cutting and filing the parts by hand will take a very long time and it may not be possible to make the parts accurately. Do not plan to make more than one pump unless you have access to some other power tools.

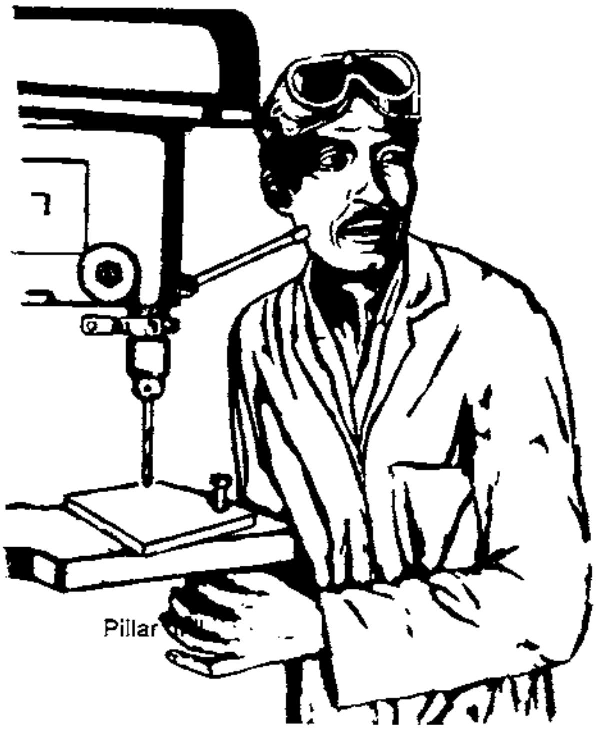
Most small workshops do not have a lathe, but many do have a gas cutting torch. Many also have a bench-grinder and a hand-held angle grinder. The step-by-step manufacturing guide that follows assumes that the pump is being made using a gas cutting torch, an arc-welder, a pillar-drill and grinding machines. Some parts of this guide will be useful to people who are making the pumps using other tools. A lathe can be useful for making parts of the impulse valve when one is available.

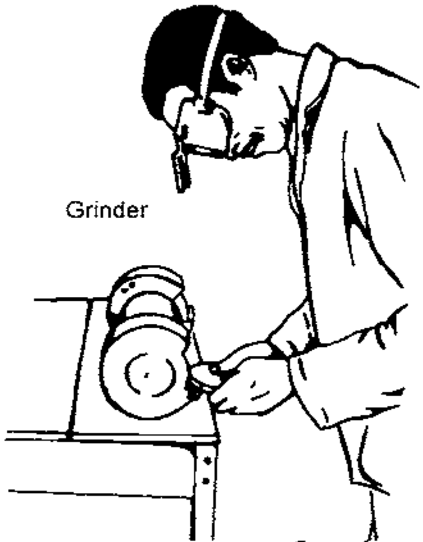


Arc welder

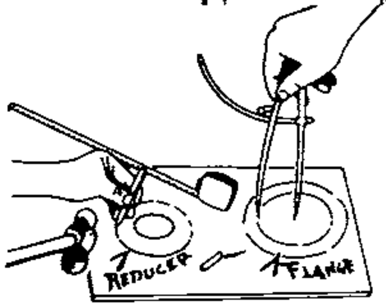
To cut circles accurately with a cutting torch, a centre pivot can be bought or made. The one shown here was made in the workshop where it is used.







You will also need chalk, a scriber, a measuring tape, dividers, a centre punch, files, a 10mm hole punch, an M10 tap and die, drill bits, two 17mm spanners, goggles, gloves, PTFE tape, a setsquare, a feeler-gauge and a hammer.

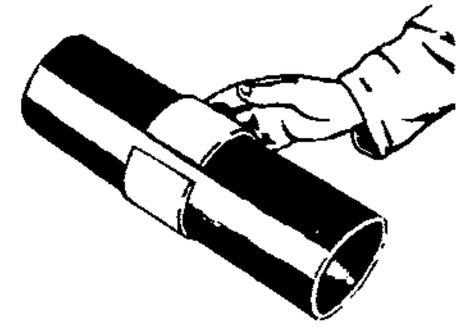


Guides and templates

When you are planning to make a number of pumps, it is worth making a few guides or templates or guides to help when marking up, cutting and drilling.

Guides for marking out the pump body

Use a short piece of PVC pipe of the same size as the pipe to be marked. Cut the PVC pipe along its length and push it over the steel pipe. Hold it in place and use it as a guide to score a line around the pipe.



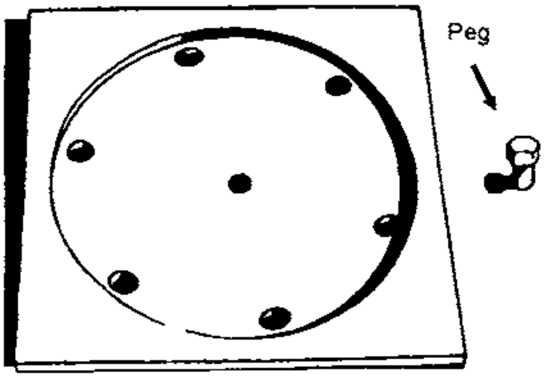
A "V" notch template like the one shown on the left can be made using thin plate (1mm) or PVC pipe. This is a guide for marking up the notches cut into the pump body and for the risers that fit into the notches.

When you have scored the line in the right place you can make it easier to see by rubbing chalk over it. Scored lines can be very hard to see through the protective goggles used when cutting. Use a centre punch to mark dots along the scored line. Use the punch about every 5mm.

It is very important to mark up clearly. If you do not, you may not cut a straight line with the torch and so you may have to start again.

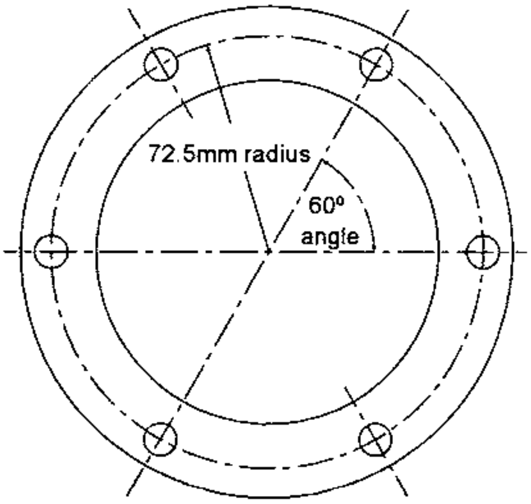
Flange and valve plate template

Design drawings number 3, 5, 6, 7, and 10 can be copied and used as templates. If you want to make a lot of pumps it is probably worth making a a steel template for the flanges. If you do not have access to a lathe, consider having the template made for you in another workshop.

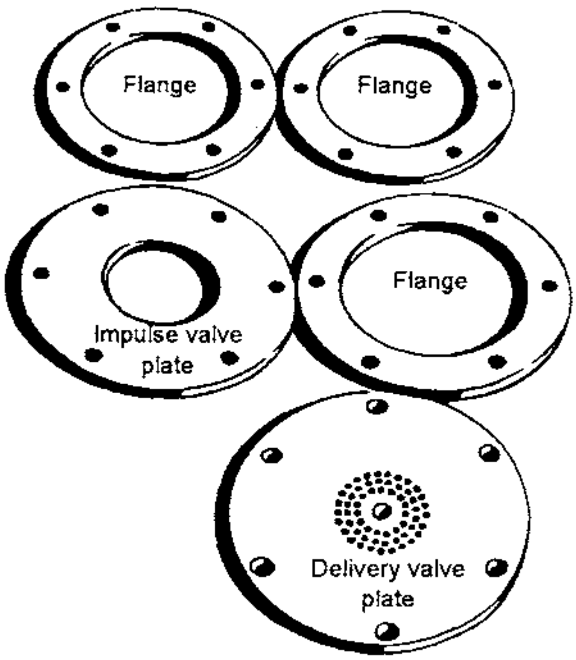


The template can be recessed with a lathe so that the flange drops into it when it is the right size. The example shown is recessed and has a ring of large holes around the outside. The example shown also provides a pattern for the bolt holes on the flanges and both valve plates.

When the first hole in the outer ring has been drilled a "peg" made by cutting a bolt is put into the hole to prevent the disc turning inside the template.



A simple template like this can be made using thin plate



You must drill the flange holes equally spaced and at the correct radius. If you do not, the flanges may not bolt together, or may only bolt together one way around.

Some people simplify the making of flanges by stacking them and drilling them together. Although this will always mean that the holes line up, it can mean that only the holes that were underneath each other when they were drilled line up. When the pump is put together is it important that the holes align whatever way around the parts are turned. Either take great care marking up each flange and be sure that the centre-punch marks are perfectly positioned before drilling, or use a template.

Materials needed to make a DTU S2 ram pump

STEEL	Size	No. of	Length/area	Notes	✔
STEEL PIPE	4" (105mm internal diameter)	1	1350mm	The outside diameter of 4" pipe is 115mm. The wall thickness should be 5mm.	
STEEL PLATE	10 or 12mm	1	330 x 500mm	Do not use plate less than 10mm thick.	
STEEL BAR	25 x 25mm	1	165mm	If this is hard to find, make some by welding together a stack of thinner 25mm wide bars.	
STEEL BAR	25 x 15mm	1	165mm		
STAINLESS OR MILD STEEL ROD	10mm	1	280mm	If this is not available, mild steel reinforcing bar will do. This includes enough to make a spare impulse valve stem to supply with the pump.	
RUBBER	Size	No. of	Area	Notes	
GASKET RUBBER (inner tube)	Car or small truck	1	825 x 170mm	Make sure that it has not perished. This includes enough to make two spare flange gaskets to supply with the pump.	
IMPULSE VALVE RUBBER	6mm	1	152 x76mm	Offcuts of conveyor belt and shoe sole material have been used in the past . This includes enough to supply a spare of each with the pump.	
DELIVERY VALVE RUBBER	3mm	1	160 x 80mm		
NUTS & BOLTS	Size	No. of	Length	Notes	
BOLTS	M10	1	30mm	These must be stainless steel or galvanised. The 4 x 40mm bolts, nuts and washers to hold the pump to the cradle are included. Extra nuts and washers are needed for the valve stem and are included in the totals. The totals also include a few spares that you should supply with the pump.	
BOLTS	M10	10	40mm		
BOLTS	M10	8	50mm		
BOLTS	M10	3	80mm		
NUTS	M10	28			
WASHERS	M10	40			
CONSUMABLES				Notes	
WELDING RODS	The amount you use will vary according to your skills.			Select rods to give good penetration on the 5mm walls of steel pipe and on 10mm steel plate.	
GAS				For the cutting torch.	
PRIMER (PAINT)				Either have the parts of the pump galvanised or paint it. In most cases it is easiest to paint it	
ENAMEL PAINT					
THINNERS					

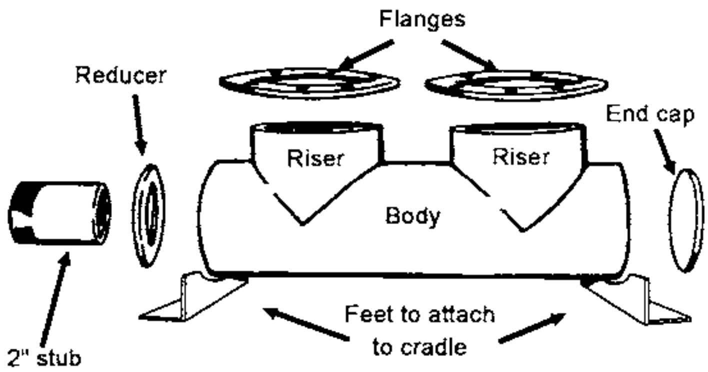
✎ Photocopy this table and tick off the materials as you get them so that you know you have everything you need before you start.

A step by step guide to manufacture

Before starting, decide which delivery valve template you will use. This will depend on the delivery head at the site where the pump is to be installed. If you do not know where the pump will be installed, use drawing number 5, which is the standard design.

Cutting the pump parts

All the cutting for this pump can be done with a cutting torch. To save grinding and to minimise scrap, the metal cut from the centre of the flanges is used as the ends of the pump body. The ends are called the "Reducer" and the "End cap". The pump body is made from steel pipe with an internal diameter of 105mm (4" pipe). The length shown on the S2 design drawings is 365mm.

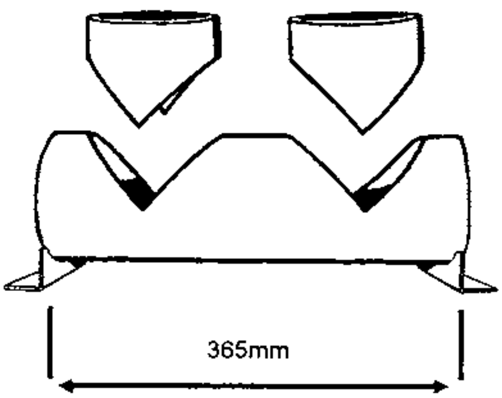


The discs of steel that fall out when the flanges are cut will fit inside the pump body, but are not big enough to cover the ends. So the ends are welded about 5mm inside the body. This should leave enough overlap to weld neatly to.

The feet are cut from 40 x 40mm angle iron. A curve that fits the pump body is cut out of one side of the angle iron.

The body is cut from a length of pipe by using the PVC pattern to score a line around the pipe. Mark it with chalk if it does not show up clearly, then go over the line with a centre punch to make sure it can be seen when cutting.

Skilled use of the cutting torch can result in the cut parts falling apart with clean edges. Even then, they will probably need to be tided up a little with a file or a grindwheel.



It is necessary to have a clean cutting nozzle on the torch, and to set the gas pressures appropriately.

If the pipe being used is galvanised, it can help to grind away the galvanise from the area to be cut. It is not worth using galvanised pipe for the pump body unless it is easy to get because a lot of the galvanised protection burns away by the time the pump is finished.

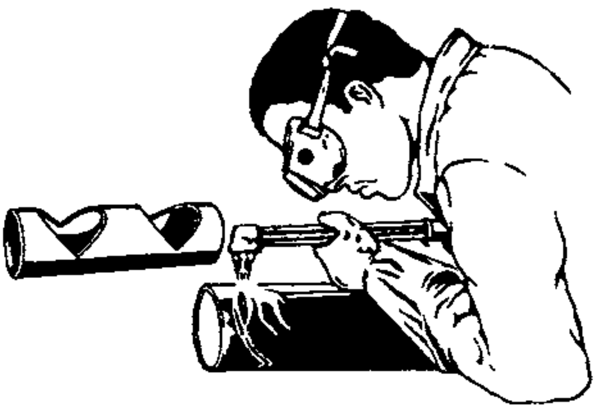
☠ *The fumes given off by burning Zinc galvanising are poisonous and should not be breathed.*

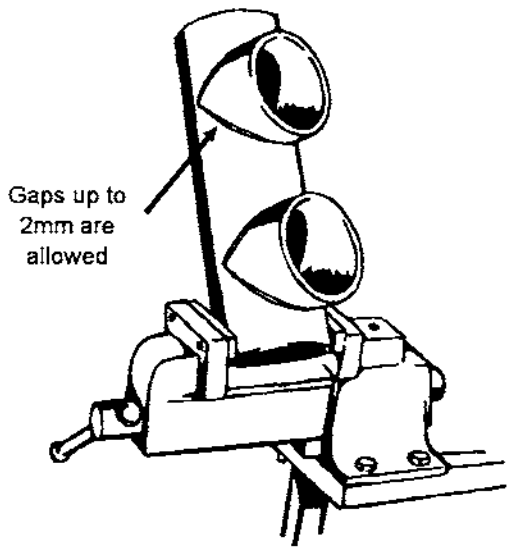
Cutting the S2 pump risers

The risers are made from the same pipe as the pump body. They are welded into "V" notches cut into the pump body. Both the risers and the notches can be marked up using the "V" notch pattern or template. At its shortest point, each riser should be no less than 30mm high. At its longest point it should be 87.5mm high.

Remember to go over the scored lines with a centre punch to make them easier to see.

Use an angle grinder or a bench-grinder to clean up the cuts so that the parts fit together properly. A gap of up to 2mm can be filled with weld.

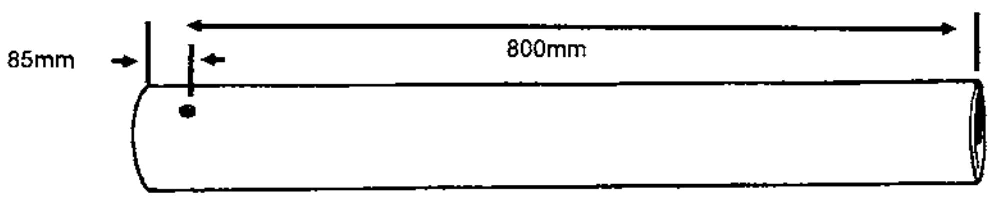




Place the risers in the pump body and grind them to fit. It can be useful to mark the risers and the notches, then grind each riser to fit a particular notch.

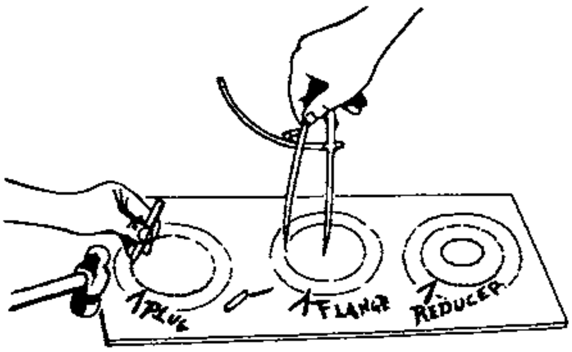
Cutting the S2 air vessel body

The air vessel body is cut from the same 105mm (4") pipe as the pump body. The length should be 800mm. This size will always be big enough, so do not change it. A ¾" hole should be cut (or drilled) with its centre 85mm from one end of the pipe. This is where the delivery pipe stub is attached.

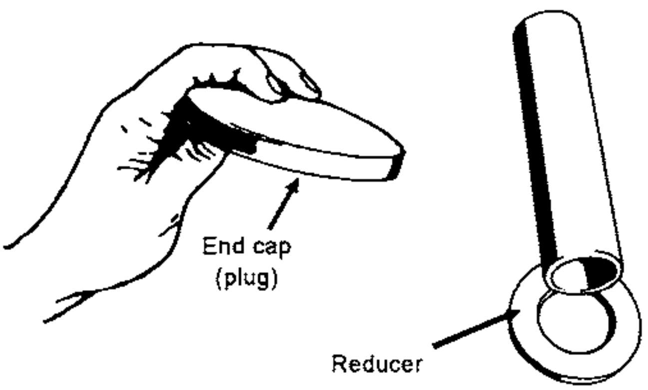


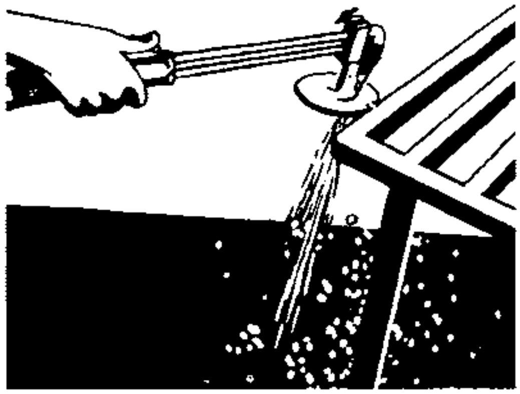
Cutting the S2 pump flanges and plates

The S2 pump body has two flanges and two pump body ends. The ends are called the "End cap" and the "Reducer". The "End cap" is the inner circle cut from a flange. The "Reducer" is the inner circle cut from a flange with another hole cut in the middle of it. The "Reducer" reduces the pipe size from the 4" pump body to the 2" drive pipe, so the inside hole is cut to take a stub of 2" pipe. The S2 air vessel has one flange and one end cap.



Mark up the flanges carefully, and cut the outer circle first. It can be useful to tack-weld the edge of the first circle to the workbench to make it easy to cut the second circle.

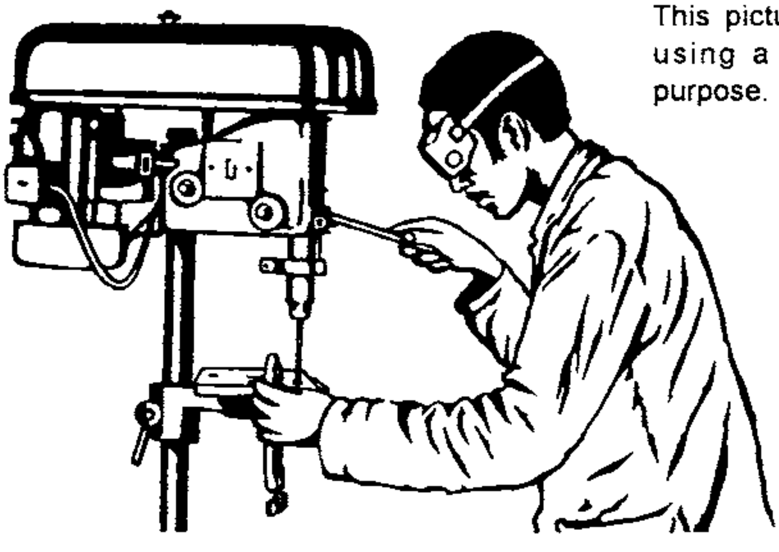




Drilling the flanges and valve plates

It is wise to make a template before drilling the flanges. Use a photocopy of design drawing number 3 as a template. Put it over the disc to be drilled, then use a centre punch to mark the hole centres through the paper template. To make the template last longer, glue it to thin cardboard before you use it.

This picture shows a flange being drilled using a steel template made for the purpose.

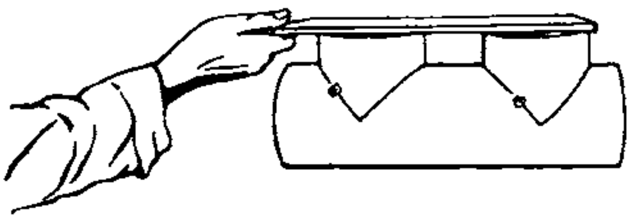


Notice that the holes are drilled before the flanges are welded to the pump body.

Welding the S2 pump body

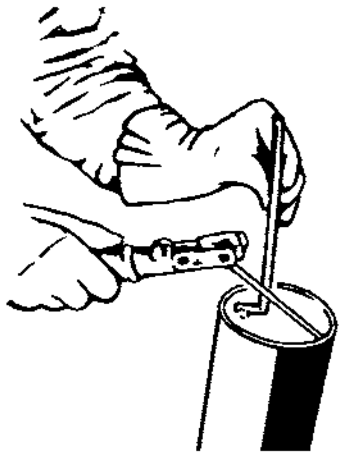
When the parts of the pump body and air vessel have all been cut they must be welded together. The following pictures show a suggested order in which to weld the parts. This order can be changed to suit the person doing the welding. It is important that the welder be skilled because the welds must be strong and not leak under pressure.

[1] Tack-weld the risers onto the body, making sure that their tops are level.

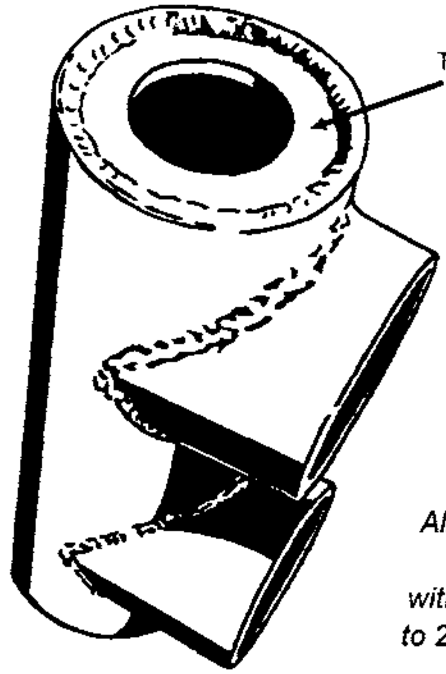


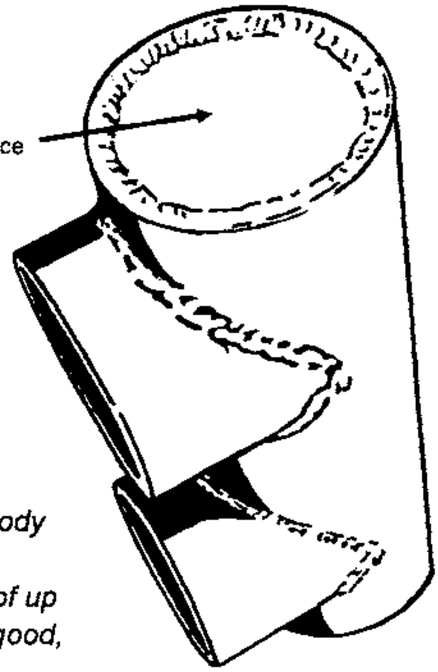
[2] Weld the risers all the way around on the outside of the body.

[3] Weld the end cap and the reducer 5mm inside the ends of the pump body.

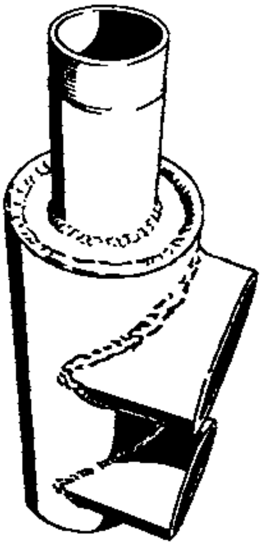


If the ends are a loose fit, tack a rod to them and hold them in place while tack-welding them. Then weld all the way around.



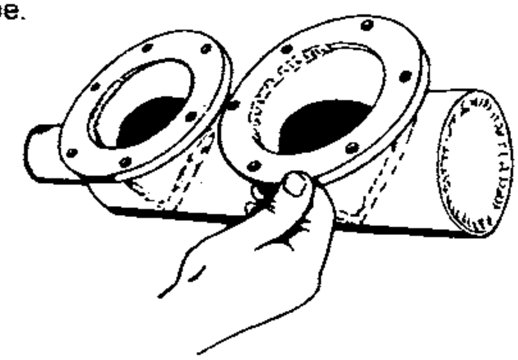


All welds on the pump body must be capable of withstanding a pressure of up to 25 bar. Be sure to get good, even penetration.

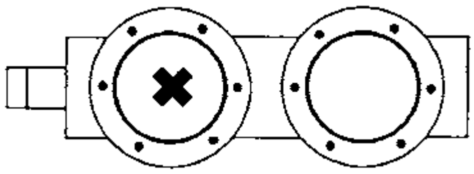


[4] Weld the stub of the 2" drive pipe into the hole in the reducer. Cut a thread on one end of the stub before cutting it from a length of pipe.

[5] Tack-weld the flanges onto the risers and make sure that the flanges are level, then weld all the way around.

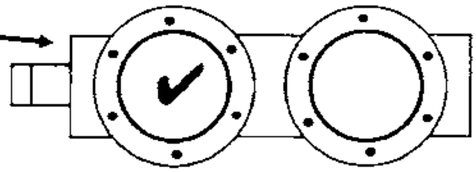


Make sure that the flange holes are not in line with the pump body.

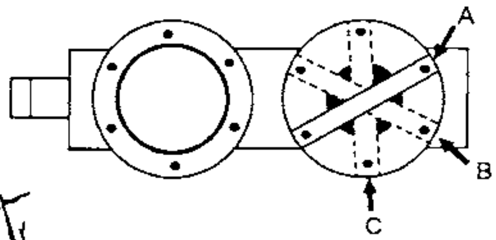


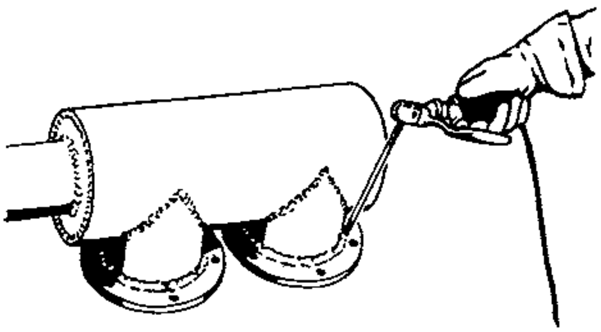
Flange hole alignment

They should be offset like this.

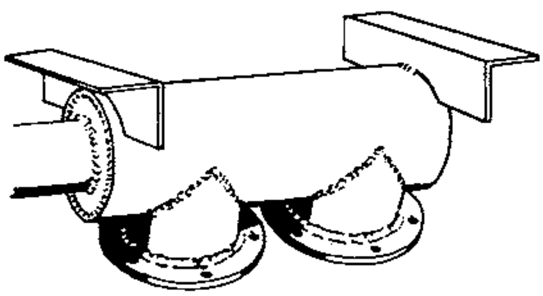


The impulse valve stop bar can later be bolted to the flange in any of the three possible positions, A, B, or C.





[6] Turn the pump body over and weld around the flanges on the other side.



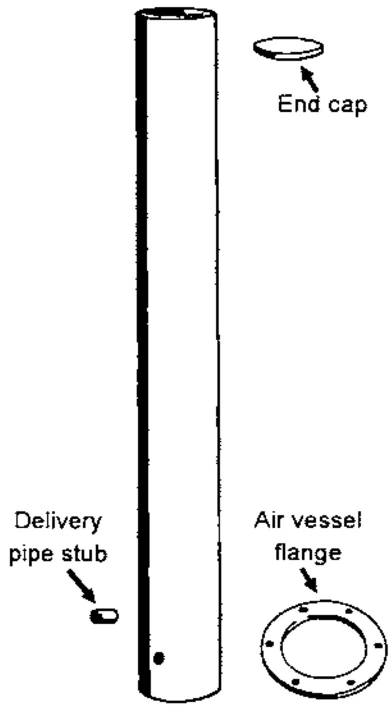
[7] Tack-weld the angle iron feet to the bottom of the pump body, 5mm in from each end. Make sure that they are both level, then weld them on both sides.

Welding the S2 pump air vessel

8 Weld the air vessel End cap in place, 5mm below the top of the air vessel.

9 Weld the air vessel flange in place, being sure to get it level. Weld it inside and outside.

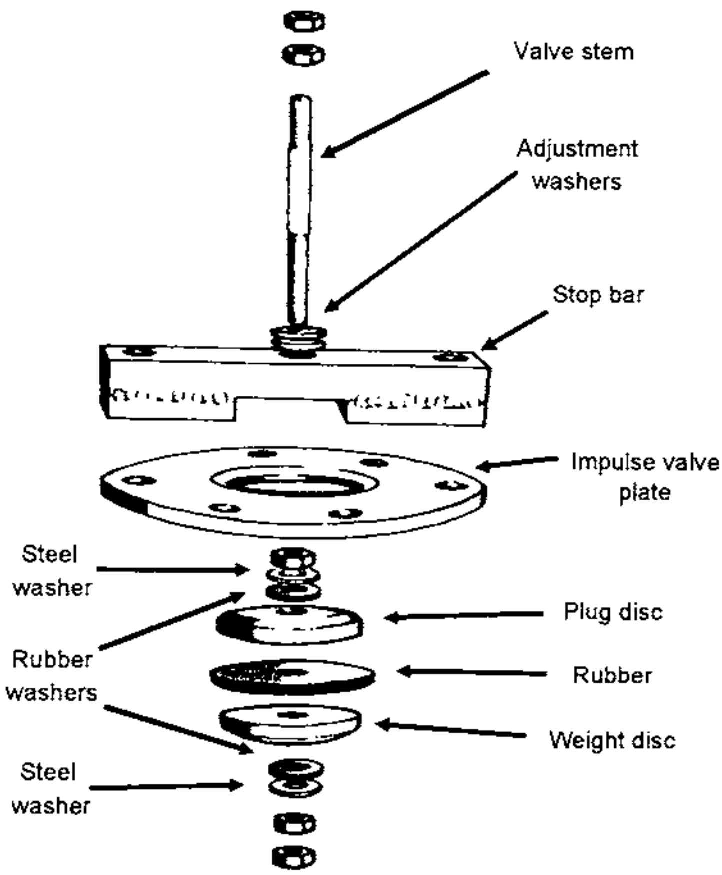
10 Weld the delivery pipe stub into the hole in the air vessel. Cut a thread onto the stub before you cut it from the end of a length of pipe.

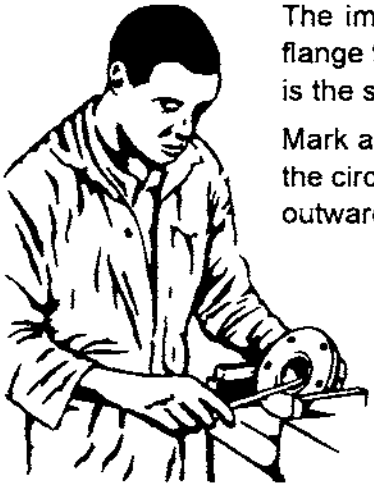


Making the S2 pump impulse valve

The valves are the most important parts of these pumps. They must be made accurately or the pump will not be as efficient as it should be and may be unreliable. If they are made carelessly, the pump may not work at all.

The valve stem rises and falls as the pump works. When it falls it pushes the plug down to allow water to flow through the central hole in the impulse valve plate. When it rises it pulls the plug up to almost block the hole, and the rubber makes a final seal.

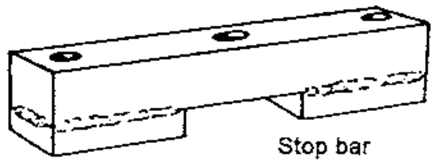




The impulse valve plate is a disc of 10 or 12mm steel plate. Use the flange template to drill holes around the outside. The outside of the disc is the same size as a flange (165mm diameter).

Mark a circle with a radius of 33mm from the centre of the disc and cut the circle out. Use a file to chamfer one side of that hole so that it slopes outward at 45°. The chamfer should be 5mm deep.

The stop bar is a 165mm length of 25mm square steel, with two 58.5mm lengths of 15 x 25mm bar welded to it. The bar is drilled to fit on top of the impulse valve plate.



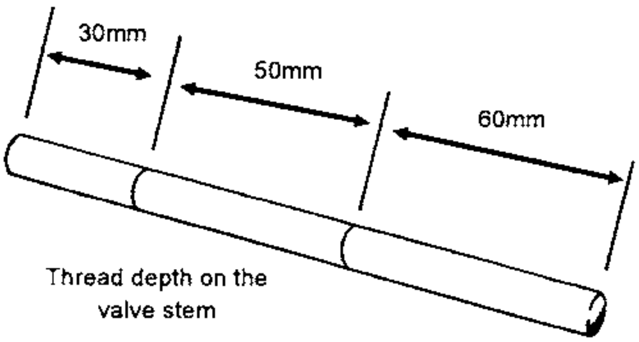
Stop bar

It is important that the holes are drilled upright through the bar, so use a pillar drill when you can.

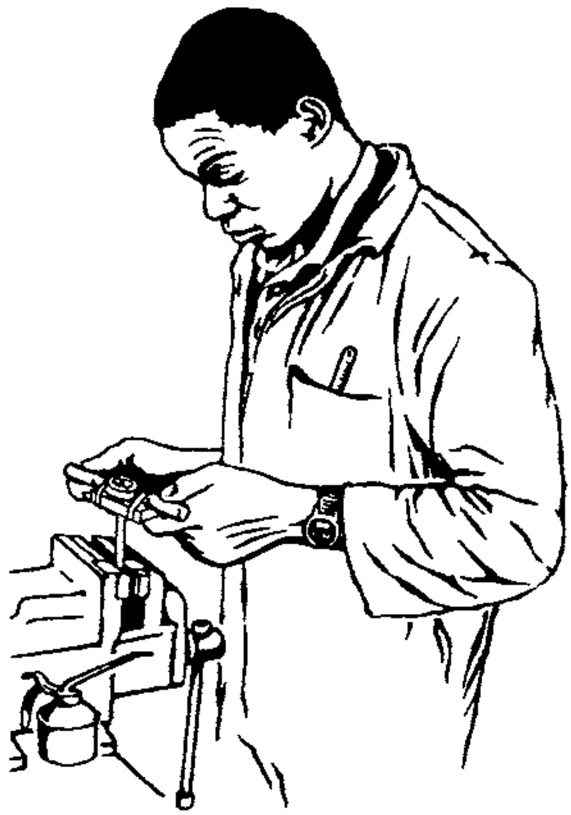
Lathe alternative

Drill the holes in the stop bar and cut the threads on the valve stem with a lathe when you can.

People prefer their pumps to look good, so it is often worth grinding away any excess weld on the stop bar using a grindwheel. In the drawing alongside, a hand-held grinder is held in a vice to make a temporary bench-grinder. If you do this, make sure that the grinder is held firmly.



Thread depth on the valve stem



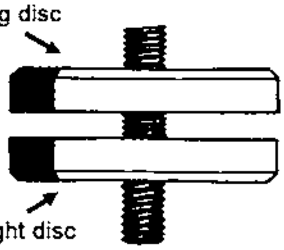
The valve stem is 140mm of 10mm stainless or mild steel bar. If plain bar is not available, reinforcing bar can be used, but it may not last as long. The bar must be straight. A slightly bent bar may break quickly when the pump is being used.

The stem must be threaded at each end. The depth of the thread is greater at the bottom than the top.

The impulse valve plug and weight discs are identical. When they are put onto the valve stem, the second is put on upside down.

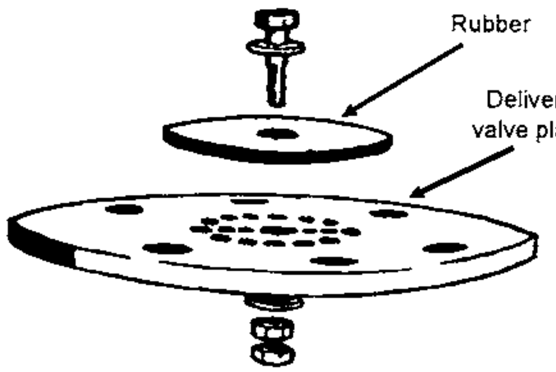
Cut two 60mm disks from the same 10 or 12mm steel plate that you used for the Impulse valve plate, then drill a hole of 11mm diameter in the centre of each. It is important that the holes are straight.

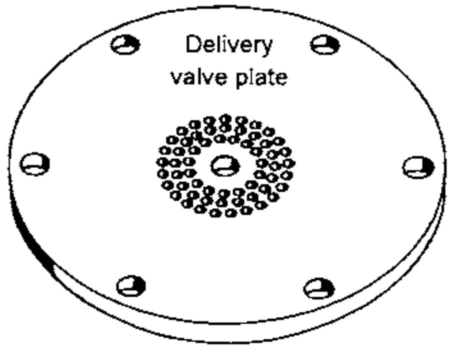
One edge of each valve disc must be chamfered with a 45° angle. The chamfer should only be 3mm deep, leaving 7mm of the edge straight.

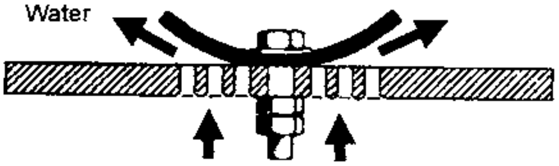


Making the S2 pump delivery valve

This is a simple flap valve. The delivery valve rubber is pushed up when the pump reaches delivery pressure, and flaps closed when the pressure drops. When the rubber is bent up, water flows through the small holes into the air vessel and into the delivery pipe.



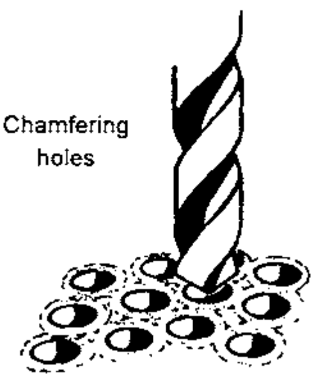




CROSS-SECTION THROUGH A DELIVERY VALVE WITH THE RUBBER OPEN

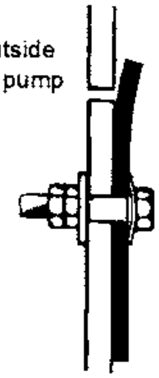
The delivery valve plate is cut from the same 10 or 12mm plate used for the flanges. You should have already chosen which of the design drawings numbers 5, 6 and 7 to use as a template. Punch through the template to mark out the holes. The central rings of holes allow water to pass through when the valve is open. To increase the flow through these holes, chamfer them using a larger drill bit (6 - 8mm) on one side of the valve plate.

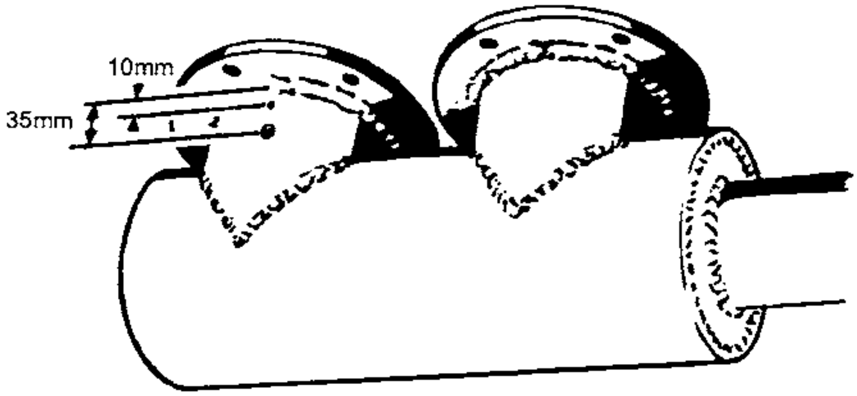
The chamfered side must be underneath when the valve is put together.



Making the S2 pump snifter valve

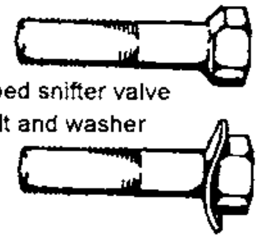
The snifter valve is a small flap valve in the riser furthest away from the drive pipe stub. When the pump is running, it allows a little air into the pump body. The air is pumped through the delivery valve with the water and keeps the level of air in the air vessel constant.





Drill a 1 or 2mm diameter hole in the riser 10mm below the bottom of the flange, (or 20mm below the top of the flange, which is the same thing). Drill a 10mm diameter hole 25mm below the first hole. The 1 or 2mm hole is the hole that air will come through.

The rubber flap inside the pump body must sit flat against the side of the riser. Because the riser is curved, you must file away parts of the bolt head and bend a washer as shown in the drawing. Then, when the bolt is tightened, the rubber flap will be able to sit flat. Be careful not to tighten the bolt enough to distort the rubber. Usually it is enough to tighten the nut with your fingers, then lock another against it using spanners.



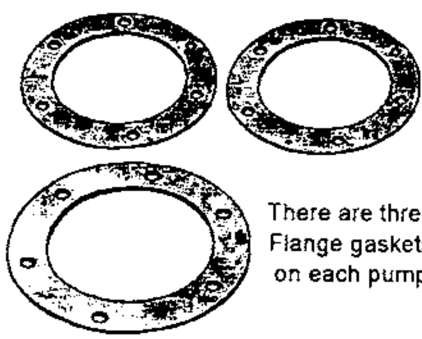
Shaped snifter valve bolt and washer

It is very important for the air in the air vessel to remain at a constant level, so it is essential that the snifter valve is fitted and working properly. If it does not work properly, the pump will be less efficient, and may even be damaged.

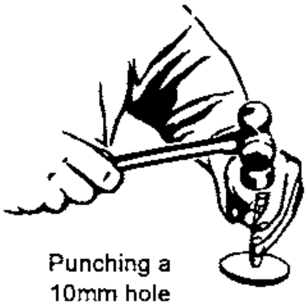
Rubber parts

You will need valve rubber and flange gasket rubber to complete an S2 pump.

Flange gaskets are usually made using old inner tube rubber. Make sure that it is not so old that it has started to perish, and try to select an area where it is more or less the same thickness. On some tubes, especially those made for big wheels, the rubber can be twice as thick in some places as others. Using the flange template as a guide, cut the big circles using a sharp knife. The bolt holes should be punched out using a 10mm hole punch. This leaves clean holes on which the threads of the bolts do not catch.



There are three Flange gaskets on each pump


Punching a 10mm hole


Delivery valve rubber


Impulse valve rubber

Inner tube rubber can also be used for the snifter valve rubber disc.

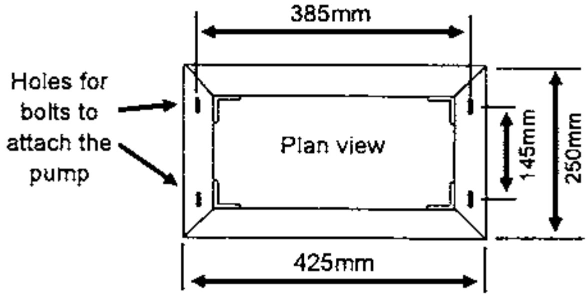
The large impulse valve rubber must be stiff but the delivery valve rubber has to bend easily. The impulse valve rubber should be about 6mm thick. Thicker rubber will probably be all right, but do not use thinner rubber. The delivery valve rubber should be about 3mm thick. Delivery valve rubber can also be used for the small impulse valve rubber washers above and below the impulse valve discs. We have used rubber made for shoe repairs and offcuts of conveyor belt when we cannot buy rubber sheet easily.

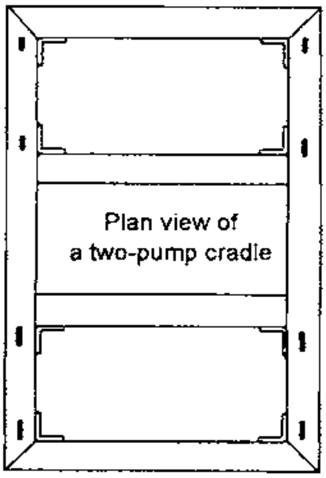
The pump cradle

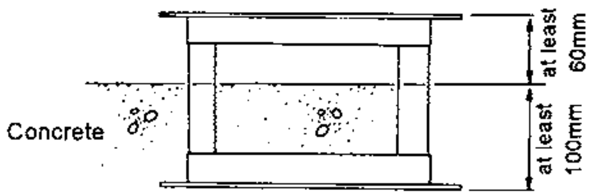
You should not make and sell a pump without including a pump cradle. If a ram pump is not securely fixed during use, it will break, and may break the connecting pipework too.

It is important that you use steel strong enough to hold the pump firmly, and that the cradle is buried in a strong concrete base. The pump tries to vibrate when operating and will soon shake itself loose from an insecure base.

Use 40mm angle-iron, about 5mm thick, to make the cradle. It can be extended to support two pumps when needed.







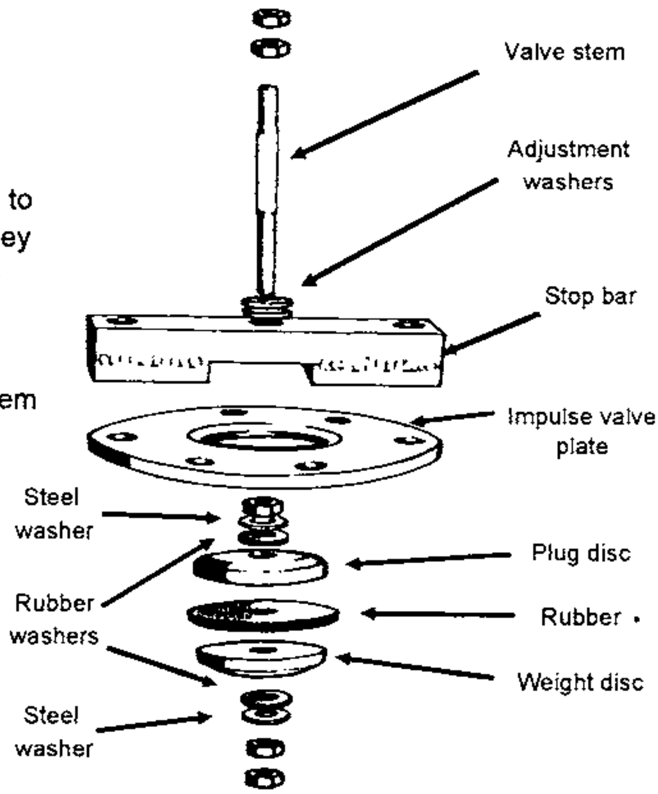
SECTION THROUGH A CRADLE INSTALLED ON A CONCRETE BASE

The pump is attached to the cradle by the angle iron feet that you have welded to the bottom of the body. Notice that the holes are slotted so that the pump can be moved from side to side slightly to allow for small mistakes in drive pipe alignment. The cradle must stand up from the concrete far enough to make it easy to reach the fixing nuts.

Assembly and testing

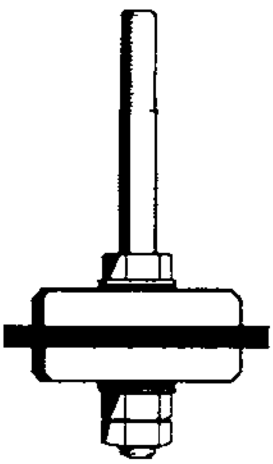
Putting the pump together properly is very important. The process of assembling the valves is described here, including some simple ways of testing them. Paint all the metal parts you have made before you assemble them for the final time. You may like to assemble them loosely before painting to check that everything is OK. To make the pump last longer, it should be painted inside and out with a metal primer, then a good enamel paint.

Assembling the impulse valve



The impulse valve design drawings, numbers 9, 19, 11 and 12, are also templates. Start by holding each part against the drawing of it on the template to check that the dimensions are right. If they are not right, either adjust them or make them again.

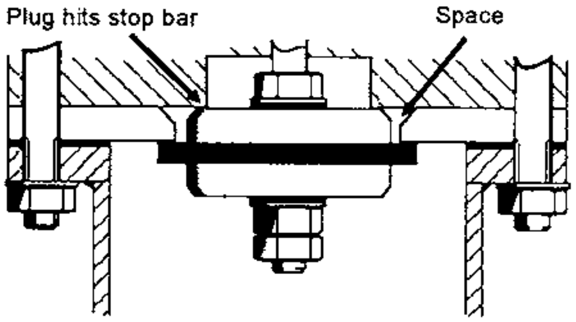
When the parts are all the right size, thread a nut onto the end of the valve stem that has a longer thread. This will be the bottom of the valve stem. Screw the nut to the end of the thread and use a spanner to lock it there. Then push a steel washer and a rubber washer up to the nut. Put the plug disk onto the stem, with the chamfered edge towards the nut and washers already there. Then push the impulse valve rubber over the stem up to the plug disc. Put the weight disc over the stem with the chamfered edge facing away from the valve rubber. Then push a rubber washer and a steel washer up to the weight disc. Thread on a nut up to the washer. Thread it on finger tight. If you use a spanner, do not tighten it so that it squashes the rubber washers. Hold the nut with a spanner and use another spanner to lock a second nut against it. The valve stem should look like the one in the drawing below.

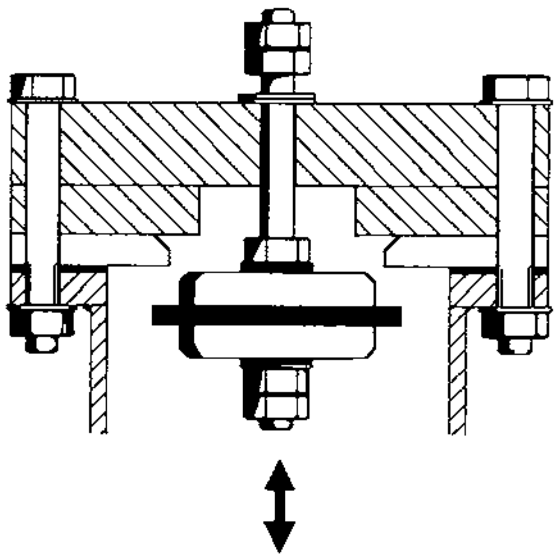


Push the stem up through the Impulse valve plate and through the stop bar. Drop two washers over it and thread on two nuts. Lock them lightly against each other. They should be adjusted when the pump is installed, so do not over-tighten them.

When the valve has been assembled, bolt it to the pump with a rubber gasket between the impulse valve plate and the flange on the pump. The impulse valve goes on the riser furthest away from the drive pipe stub. Tighten all the bolts on the flange except the two holding the stop bar in place. Put nuts on these but leave them finger tight.

Check that the impulse valve can move freely up and down without the plug disc catching on the sides of the hole in the Impulse valve plate. If it does catch, push the stop bar away from the side that catches. The stop bar has a small amount of movement between the bar and the threads of the bolts holding it.

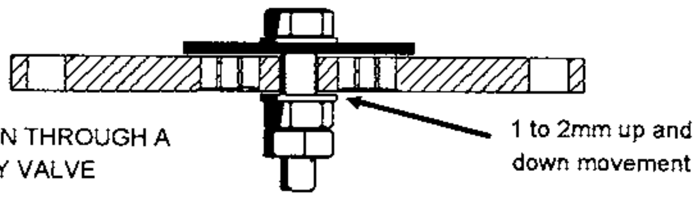




There should be a space all the way around between the side of the plug disc and the Impulse valve plate. If the pump has been made exactly to the measurements, the space will be 3mm. If it is less than 1mm, you should take the valve apart and check the parts against the templates. If the plug disc is too large you may be able to make it smaller. If the hole in it is not in the centre, you will probably have to make another. If the hole in the impulse valve plate is too small or not round, you may be able to file it out. Do not file it so that the hole is bigger than 66mm as shown in design drawing number 10. The plug disk must also hit the stop bar all the way around. Turn the plug disk to make sure that this is so.

Assembling the delivery valve

The delivery valve must be assembled so that the nuts are tight on the bolt, but the bolt has some up and down movement. This is shown on the drawing below. When the bolt head is pressed down against the washer and rubber on top of the delivery valve plate, there should be a gap on the underside of 1 to 2mm between the plate and the washer.

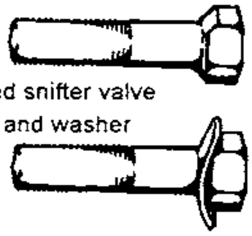


CROSS SECTION THROUGH A DELIVERY VALVE

To assemble the valve put a washer over a 40mm M10 bolt and push it through the delivery valve rubber. Make sure that the rubber moves freely on the bolt. It the hole in the centre was cut out with a hole punch, it should move freely up and down. If the rubber distorts when it is pushed over the bolt, you must make the hole in the middle bigger. This can sometimes be done with a round file or a stick with emery paper wrapped around it.

Check that there are no rough edges on the holes in the delivery valve plate then push the bolt through it. The chamfer on the delivery valve holes should be on the underside of the plate, away from the bolt head. Push on a washer and thread on a nut, leaving 1 to 2mm up and down movement in the bolt. Use a feeler-gauge if you have one, or do up the nut with your fingers until it starts to get tight, then unscrew it one full turn. Use spanners to lock a second nut against the first, then check that there is still enough up and down movement. It should be at least 1mm and not more than 2mm.

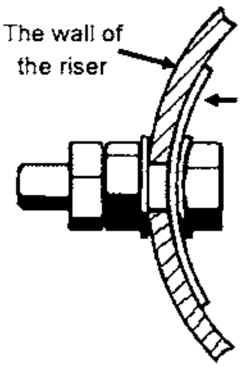
Assembling the snifter valve



Shaped snifter valve bolt and washer

The snifter valve rubber must sit flat against the inside curve of the riser below the delivery valve. To make this happen you must shape a nut and washer as shown on page 15.

Make sure that there are no rough edges on the inside of the two holes you have drilled through the riser on the pump body. Even a small burr can stop the valve working properly. Also make sure that the holes have not been blocked by paint.



CROSS SECTION of the snifter valve assembly from above

To assemble the valve, push the rubber over the bolt up against the shaped washer. Make sure that the rubber moves freely on the bolt. It the hole in the centre was cut out with a hole punch, it should move freely up and down. If the rubber distorts when it is pushed over the bolt, you must make the hole in the middle bigger. Then push the bolt through the riser on the pump body. Make sure that the rubber disc covers the small snifting hole in the riser. Be sure to use a disc of rubber rather than a smaller flap. If the disc turns it will still cover the snifting hole.

Push a washer over the thread outside the pump body and thread a nut onto it. Holding the head inside the pump, tighten the nut half a turn with spanner. Check that the rubber disc has not distorted. If it has, slacken the nut off until the rubber disc sits flat against the riser. When the disk sits flat and there is no play in the nut and bolt, use spanners to lock a second nut against the first.

Spare parts

It is important that you make the pump parts carefully to match the dimensions given on the design drawings. Then, if a part needs to be replaced, it can be made from the drawings and taken to the pump. If the parts are made carelessly, the pump may have to be brought to the workshop so that you can make a part that fits.

The DTU S2 ram pump costs much less than any commercial pump we have seen. It is made using materials and tools that are widely available. The disadvantage of this is that some parts will wear out or fail more quickly than they do on expensive pumps. The parts that have failed in the past are the rubber discs and the valve stem. You should make spares of these and supply them with the pump. A valve stem made from stainless steel usually lasts a lot longer than a stem made from mild steel. After the pump has been taken apart a few times it may also need a new flange gasket. The head or the thread on a bolt or nut may also get damaged, so the pump should be supplied with a few extra nuts, bolts and washers of the right size.

Provide these spares with each pump:

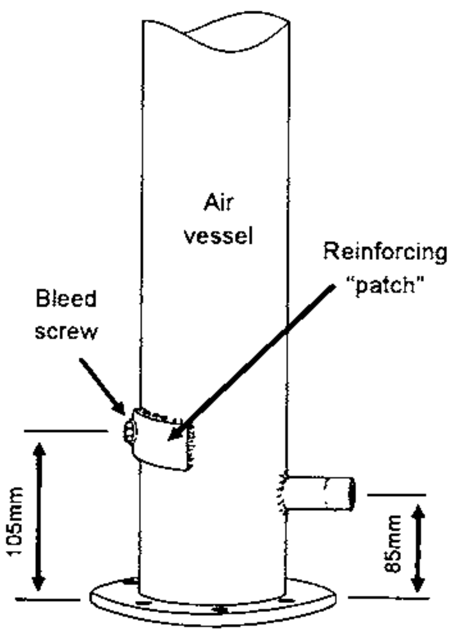
- a spare impulse valve stem;
- spare impulse and delivery valve rubbers;
- 2 spare flange gaskets.
- 1 spare 40mm M10 nut and bolt;
- 1 spare 80mm M10 nut and bolt;
- 2 spare 50mm M10 nuts and bolts;
- 4 spare M10 nuts and 6 spare washers.

If the impulse valve discs are not made carefully with their holes in the centre, the valve plug disk may not turn freely in the hole in the valve plate. It may not strike the stop bar on both sides all the time when the valve plug is turned through 360°. This may also happen if the hole in the middle of the stop bar is not drilled absolutely vertically through it. If you find these faults, you must make the parts again.

Optional addition

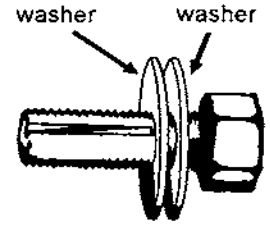
The addition of a "bleed" screw in the air vessel can make it easier to find out what is wrong with a pump when it does not perform properly. It can also make the pump easier to maintain.

Many commercially available pumps have a bleed screw on the air vessel. The screw is opened to check that there is enough air in the air vessel, and to let out the water when there is too much. It is also a useful way of letting the pressure out of the air vessel when you want to take it apart for maintenance.



The bleed screw should be position just above the delivery pipe stub on the other side of the air vessel.

The pressure in the air vessel can get very high, so the bleed screw and its thread must be made carefully to prevent it leaking. To make the thread longer and more secure, a "patch" cut from the same 4" pipe as the air vessel should be welded over the area first. Then you drill a hole through the patch and the wall of the air vessel. Cut a thread in the hole so that an M10 bolt will screw into it.



The thread on the bolt used should be 30mm long. Holding the head of the bolt in a vice, use a hacksaw to cut a shallow slot along the threads. Do not cut more than 1mm deeper than the bottom of the thread. When the hacksaw blade hits the bolt head, stop. The bolt should look like the one in the drawing alongside. You will need to use a steel and a fibre washer under the bolt head to make a good seal.

To bleed air or water from the air vessel, simply undo the bolt a few turns until air starts to hiss out, or water to spray out.

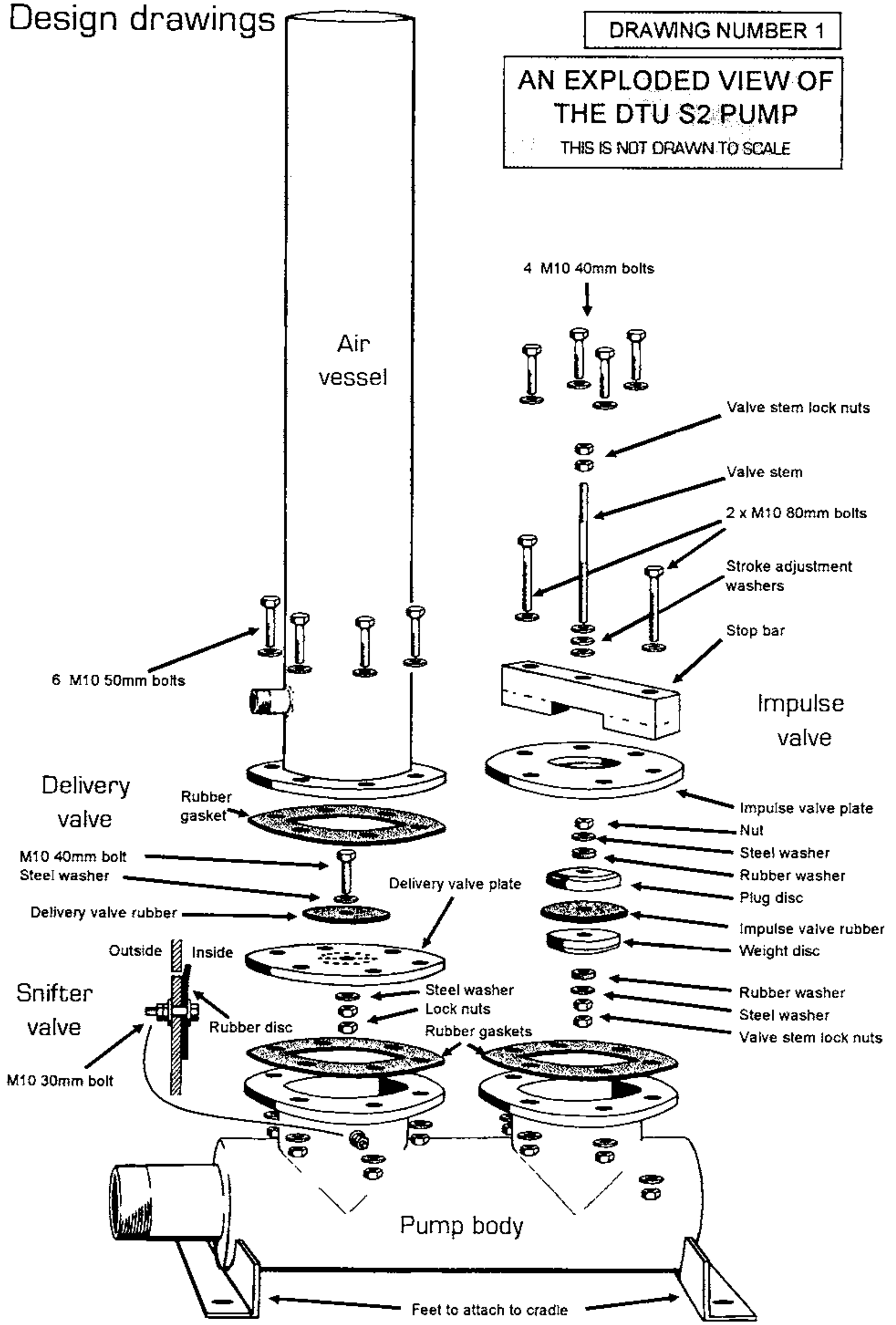
DRAWINGS

DTU S2 pump
Design drawings

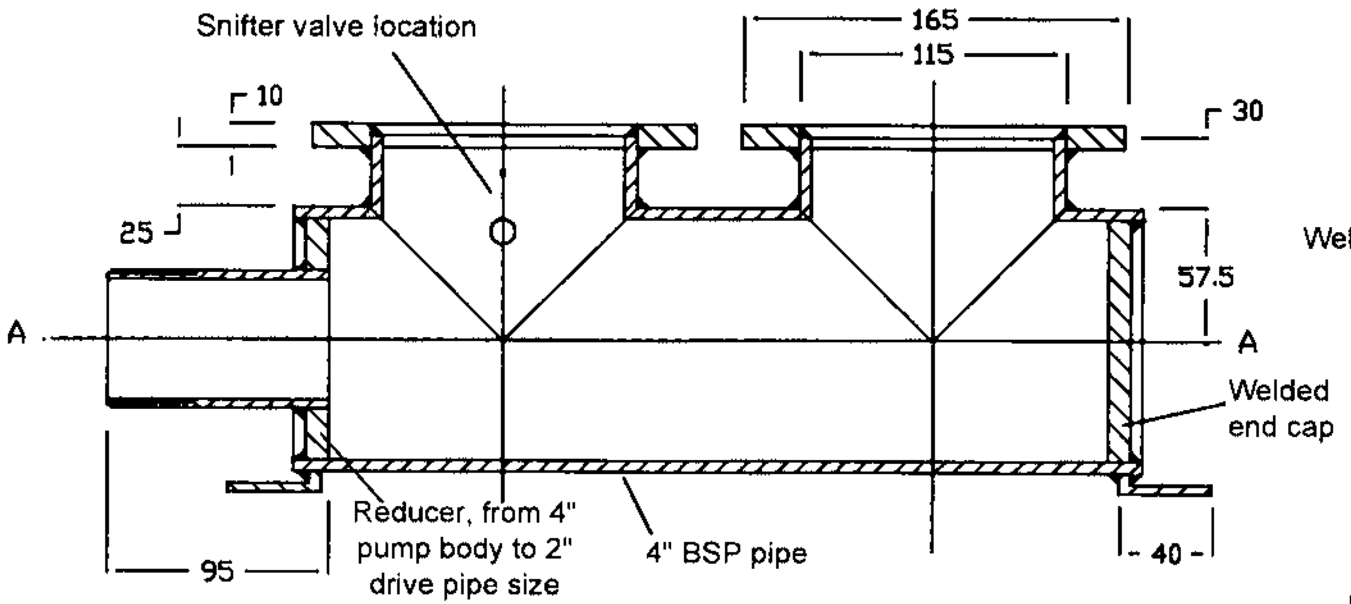
DRAWING NUMBER 1

AN EXPLODED VIEW OF THE DTU S2 PUMP

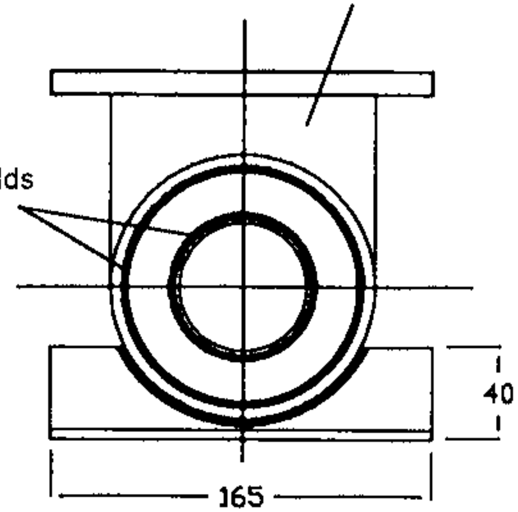
THIS IS NOT DRAWN TO SCALE



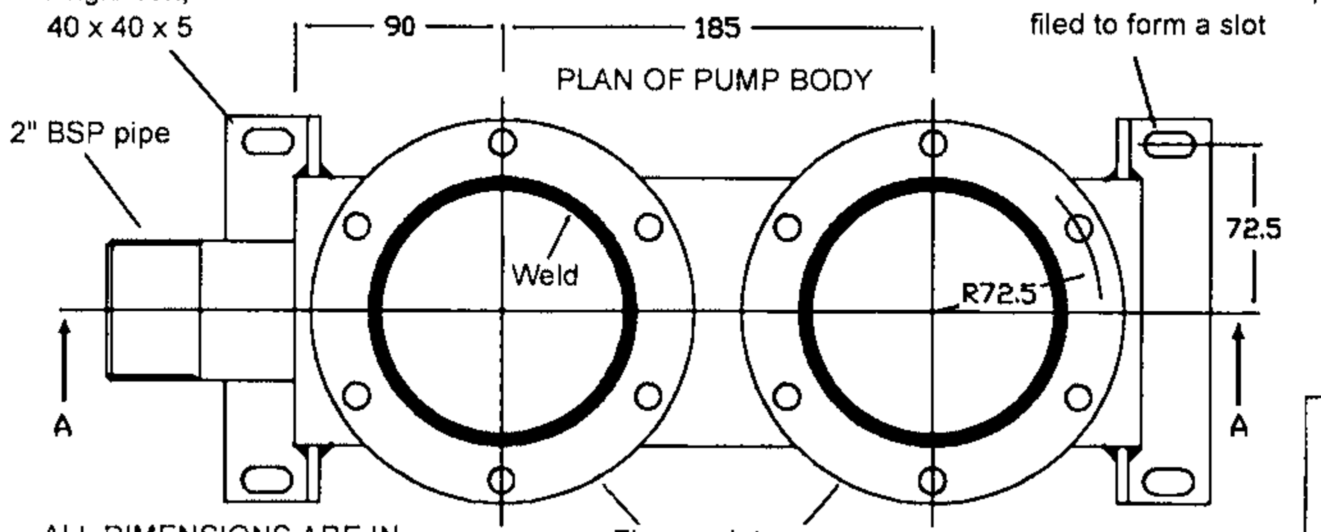
CROSS SECTION THROUGH THE MIDDLE OF THE PUMP BODY
that is from A to A on the plan drawing below



VIEW FROM THE DRIVE PIPE END



PLAN OF PUMP BODY



ALL DIMENSIONS ARE IN MILLIMETERS (mm)

MATERIALS

PIPE: the pipe used is mild steel with a 4" (115mm) outside diameter and an inside diameter of 105mm.

PLATE: the plate is mild steel either 10 or 12mm thick.

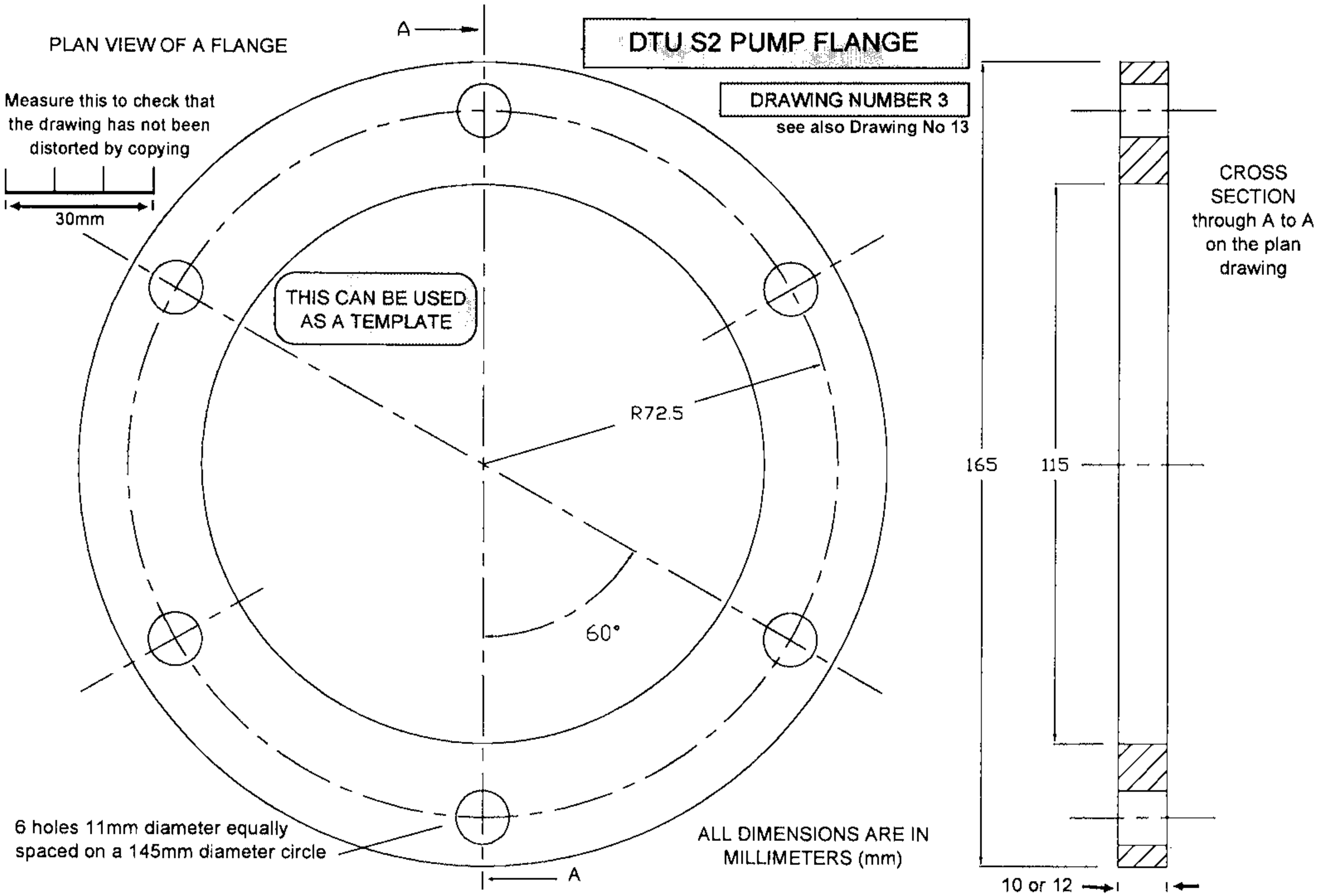
ANGLE IRON: use 40 x 40mm angle iron, about 5mm thick. Use bigger angle iron if this size is hard to get.

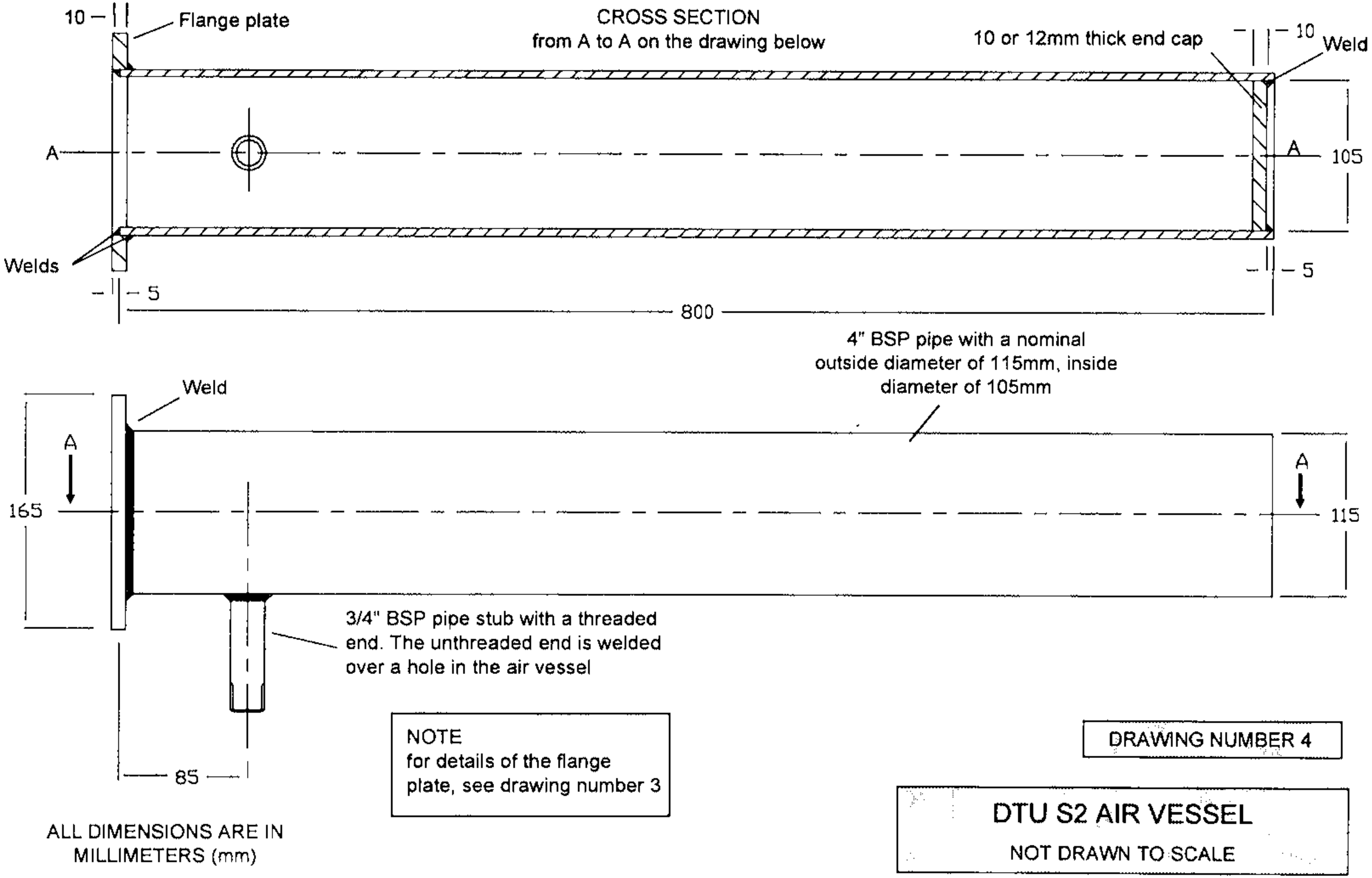
ALL THE JOINTS SHOULD BE WELDED

DRAWING NUMBER 2

DTU S2 pump body

NOT DRAWN TO SCALE


DTU S2 PUMP FLANGE
DRAWING NUMBER 3
see also Drawing No 13
PLAN VIEW OF A FLANGE
Measure this to check that the drawing has not been distorted by copying
30mm
A
A
THIS CAN BE USED AS A TEMPLATE
R72.5
60°
6 holes 11mm diameter equally spaced on a 145mm diameter circle
ALL DIMENSIONS ARE IN MILLIMETERS (mm)
CROSS SECTION through A to A on the plan drawing
165
115
10 or 12



DRAWING NUMBER 4

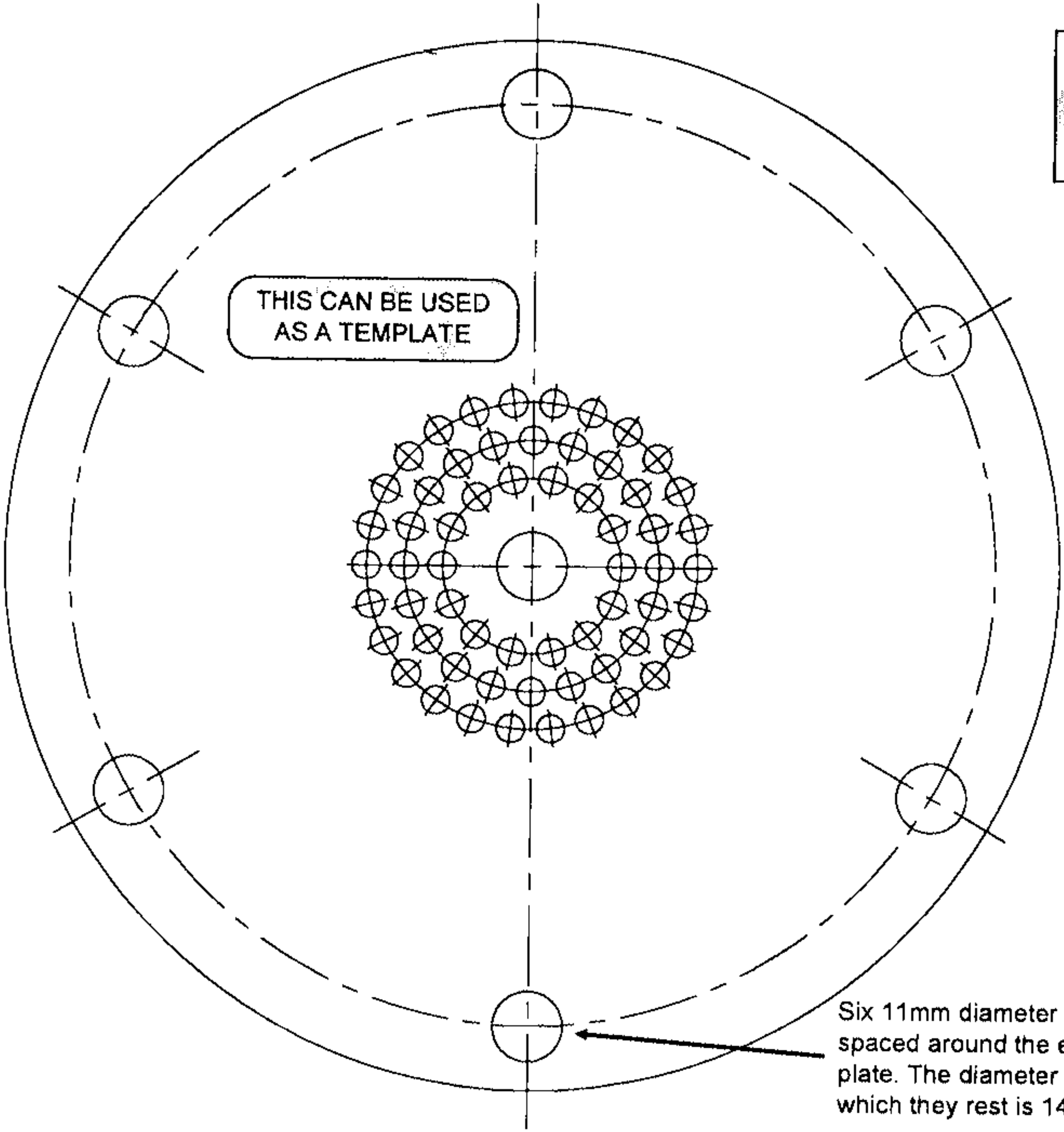
DTU S2 AIR VESSEL

NOT DRAWN TO SCALE

DELIVERY VALVE PLATE 1

Suitable for delivery heads over 75 meters
DRAWN TO SCALE: 1:1

DRAWING NUMBER 5



NOTES

The centre hole is 10.5mm in diameter.

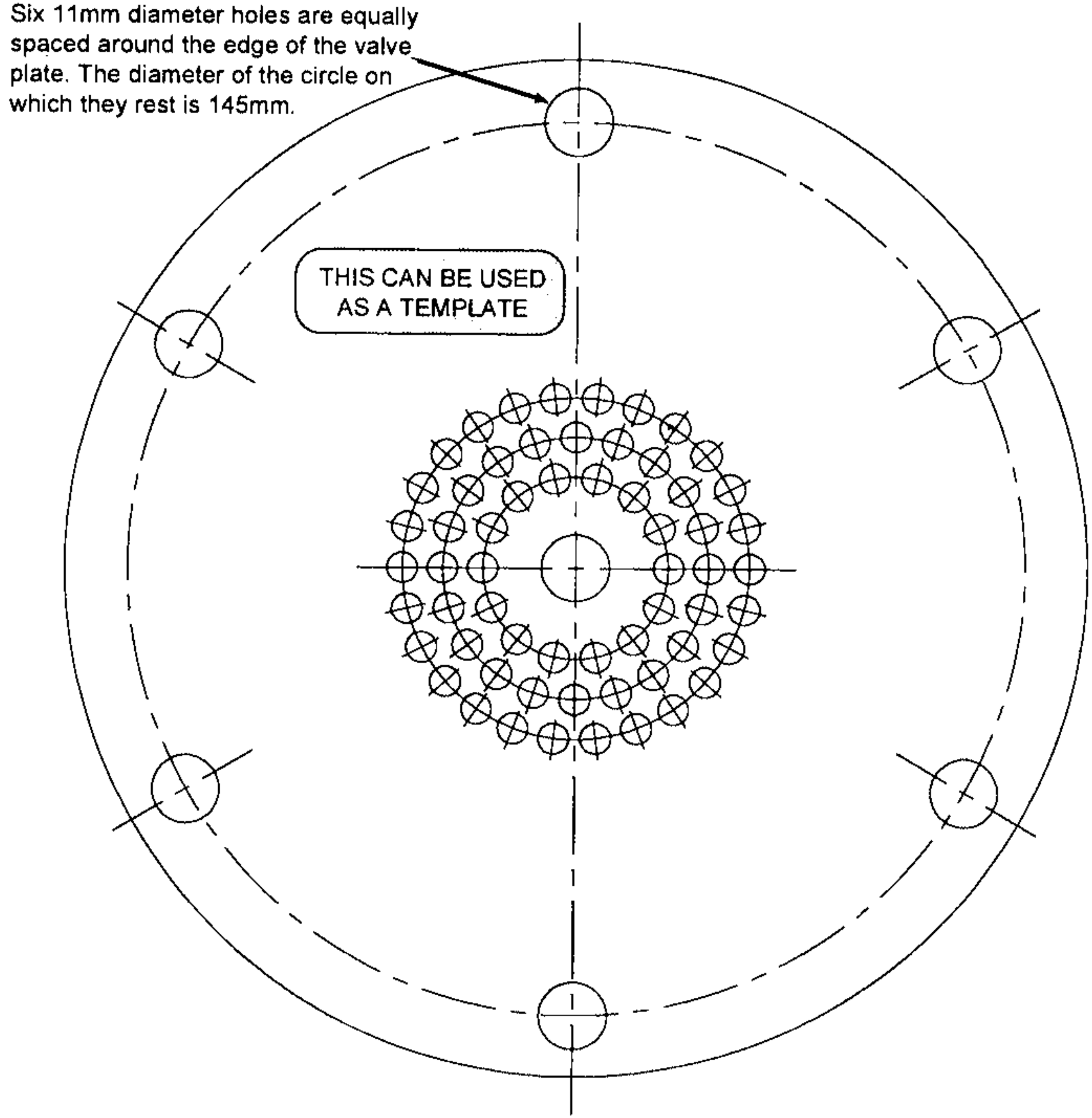
The rings of holes are 4.5mm in diameter. They are equally spaced on circles drawn from the centre with a radius of 14, 20 and 26mm. All these holes must be deburred and chamfered on one side. The chamfered side will be the underside of the delivery valve.

The valve rubber diameter is 64mm and it should be at least 3mm thick.

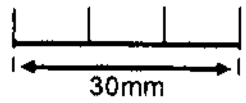
The plate should be 165mm in diameter and 10 or 12mm thick.

Six 11mm diameter holes are equally spaced around the edge of the valve plate. The diameter of the circle on which they rest is 145mm.

Measure this to check that the drawing has not been distorted by copying



Measure this to check that the drawing has not been distorted by copying



NOTES

The centre hole is 10.5mm in diameter.

The rings of holes are 5mm in diameter. They are equally spaced on circles drawn from the centre with a radius of 15, 21.5 and 28mm. All these holes must be deburred and chamfered on one side. The chamfered side will be the underside of the delivery valve.

The valve rubber diameter is 68mm and it should be at least 3mm thick.

The plate should be 165mm in diameter and 10 or 12mm thick.

DRAWING NUMBER 6

DELIVERY VALVE PLATE 2

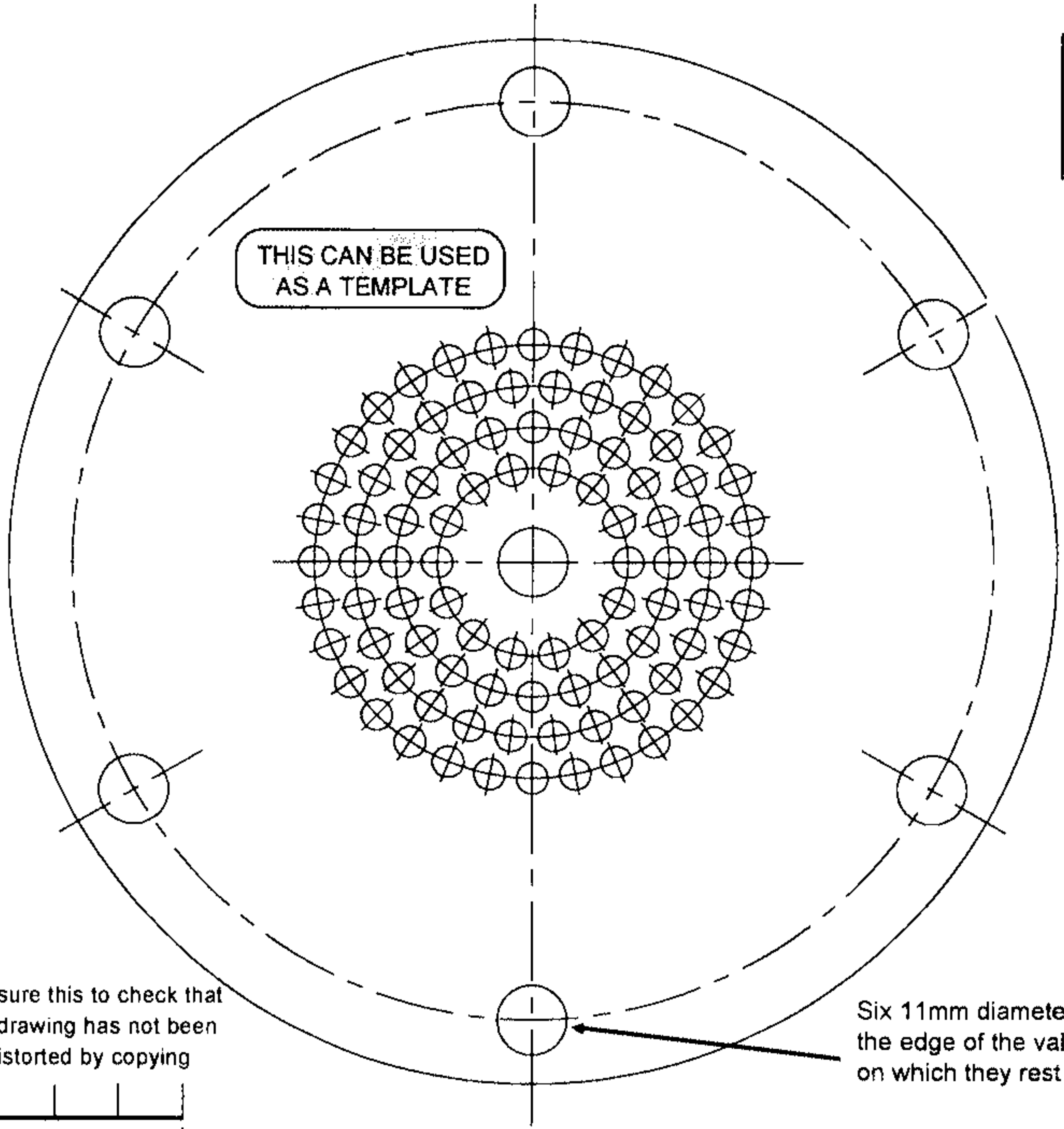
Suitable for delivery heads of 35 to 75 meters

DRAWN TO SCALE: 1:1

DELIVERY VALVE PLATE 3

Suitable for delivery heads up tp 35 meters
DRAWN TO SCALE: 1:1

DRAWING NUMBER 7



NOTES

The centre hole is 10.5mm in diameter.

The rings of holes are 5mm in diameter. They are equally spaced on circles drawn from the centre with a radius of 15, 21.5, 28 and 34.5mm. All these holes must be deburred and chamfered on one side. The chamfered side will be the underside of the delivery valve.

The valve rubber diameter is 80mm and it should be at least 3mm thick.

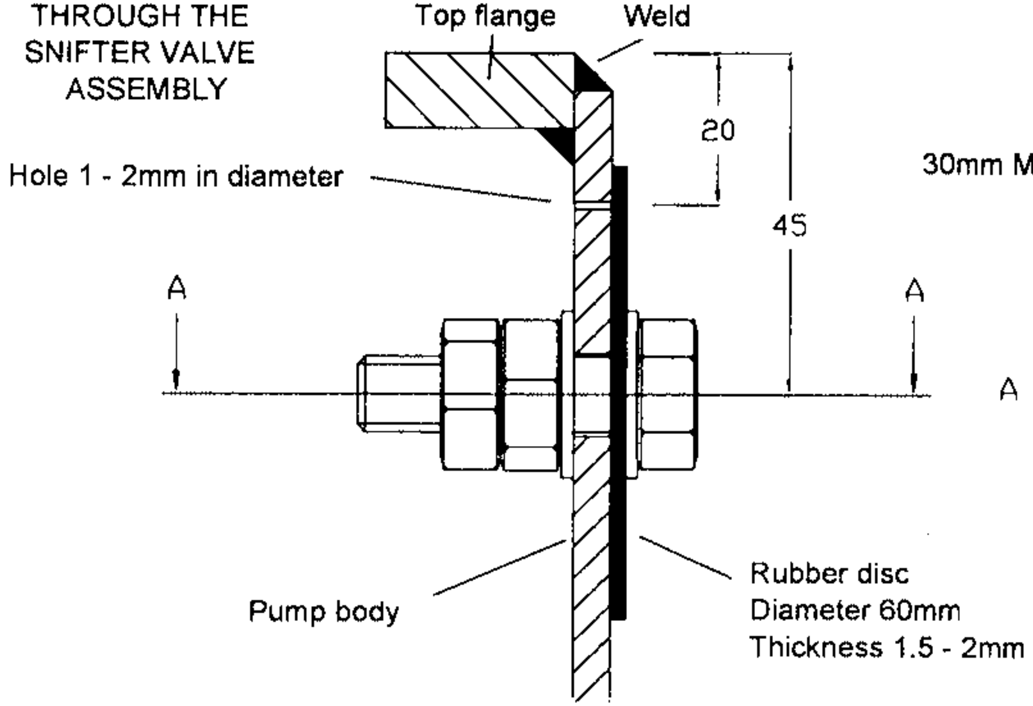
The plate should be 165mm in diameter and 10 or 12mm thick.

Six 11mm diameter holes are equally spaced around the edge of the valve plate. The diameter of the circle on which they rest is 145mm.

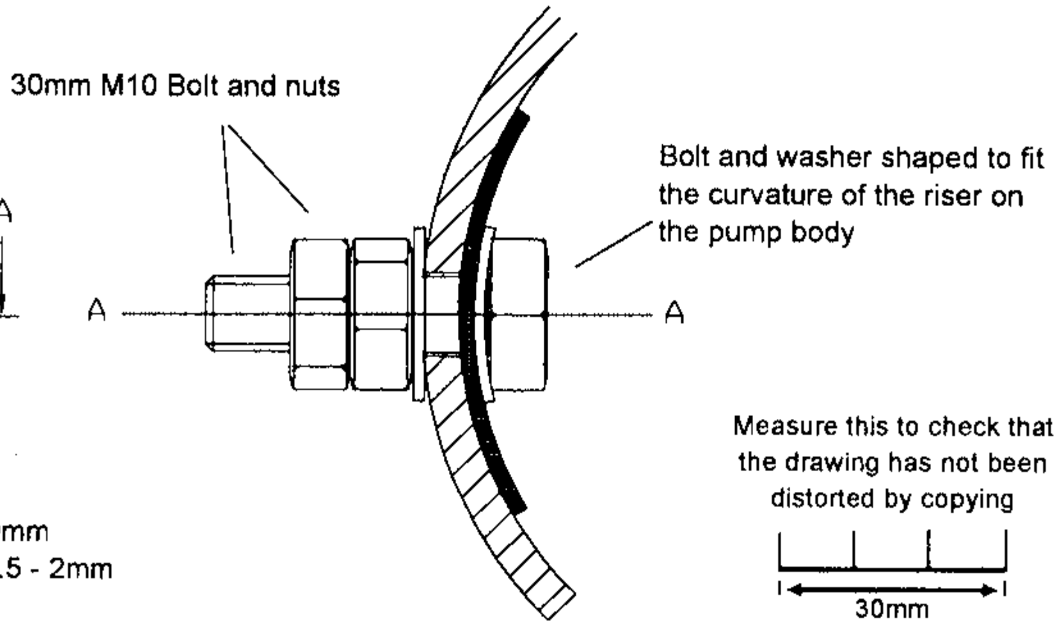
Measure this to check that the drawing has not been distorted by copying

30mm

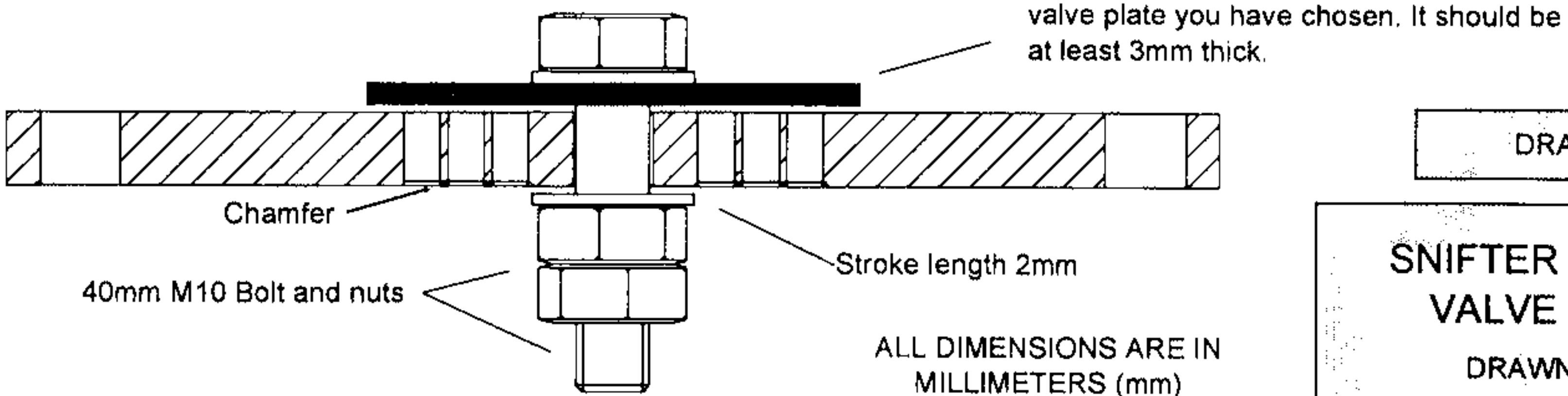
CROSS SECTION THROUGH THE SNIFTER VALVE ASSEMBLY



PLAN VIEW CROSS SECTION
that is from A to A on the assembly drawing



CROSS SECTION THROUGH THE DELIVERY VALVE ASSEMBLY



ALL DIMENSIONS ARE IN MILLIMETERS (mm)

DRAWING NUMBER 8

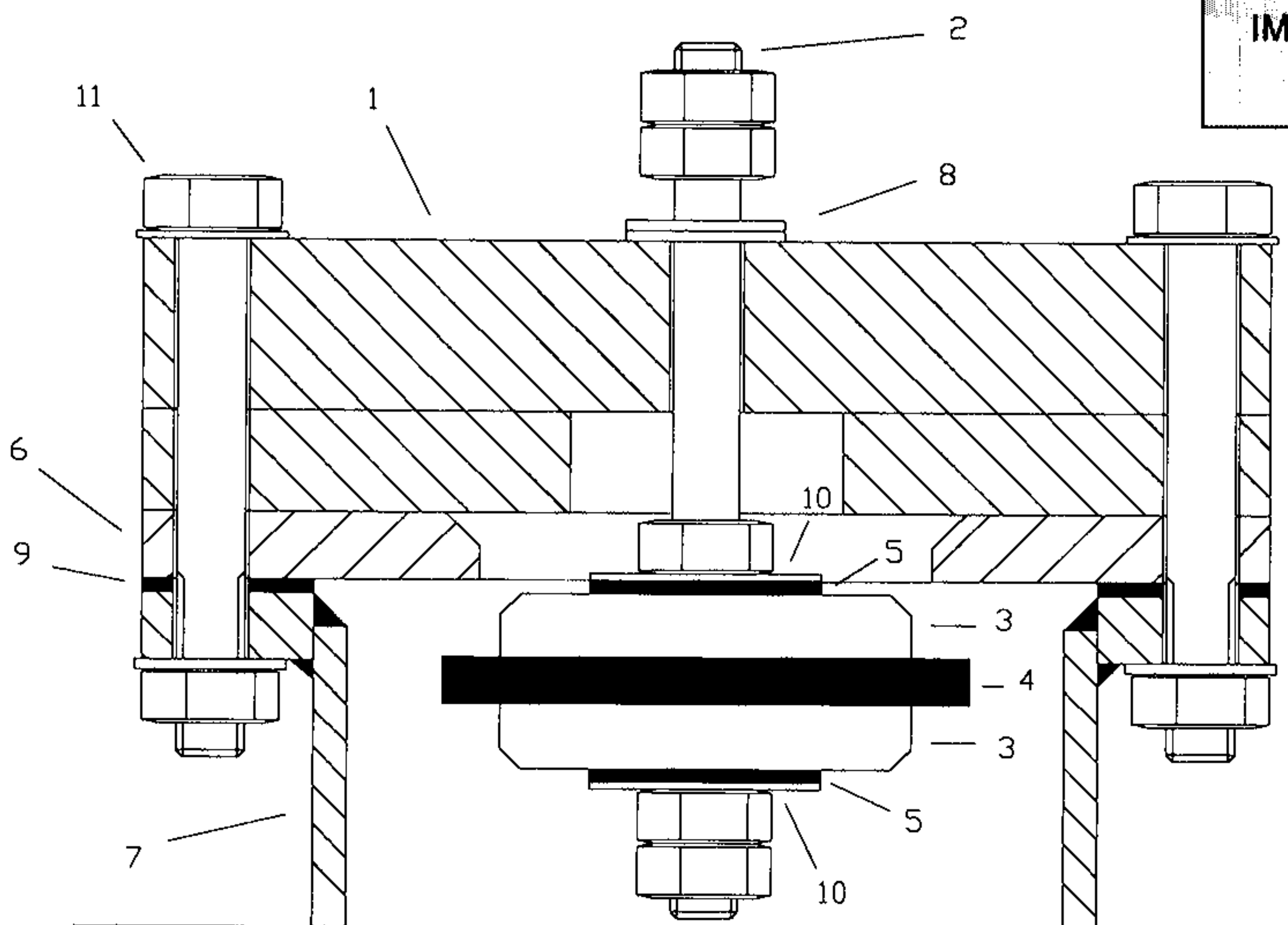
SNIFTER AND DELIVERY VALVE ASSEMBLIES

DRAWN TO SCALE: 1:1

IMPULSE VALVE ASSEMBLY

DRAWN TO SCALE: 1:1

DRAWING NUMBER 9



PARTS LIST

1 - Stop bar
2 - Valve stem
3 - Valve disc (x2)
4 - Rubber disc
5 - Rubber washer (x2)
6 - Valve plate
7 - Pump body
8 - Stroke adjustment washers
9 - Rubber gasket
10 - Large steel washers, about 35mm in diameter (x2)
11 - M10 Bolts, nuts and washers

BOLTS
M10 x 80mm (x2)
M10 x 40mm (x4)

NOTES

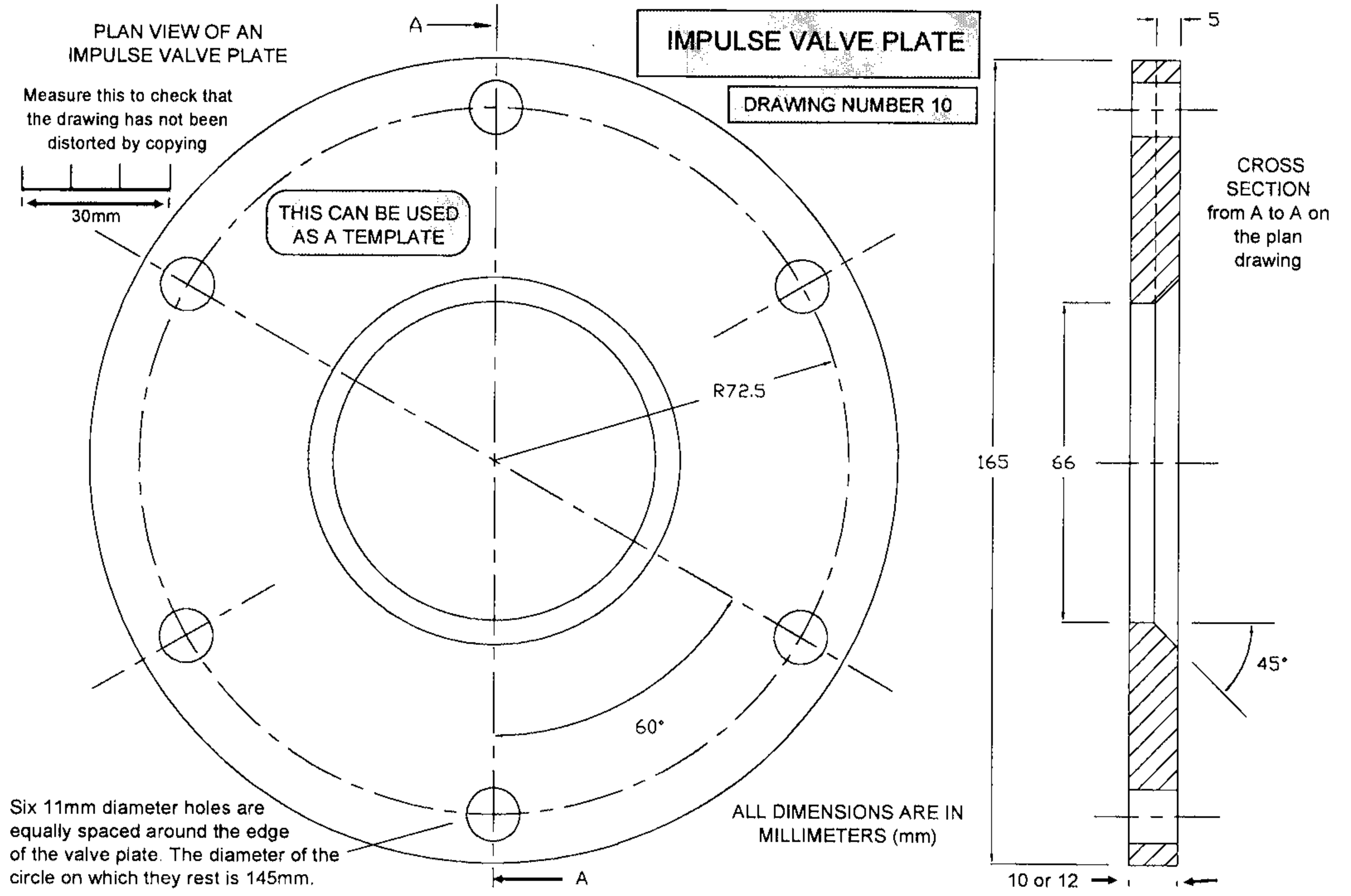
The rubber disc (4) is 76mm in diameter and 6mm thick. Do not use a larger disc.

The rubber washers (5) are made to the same diameter as the steel washers (10) and can be 1.5 to 3mm thick.

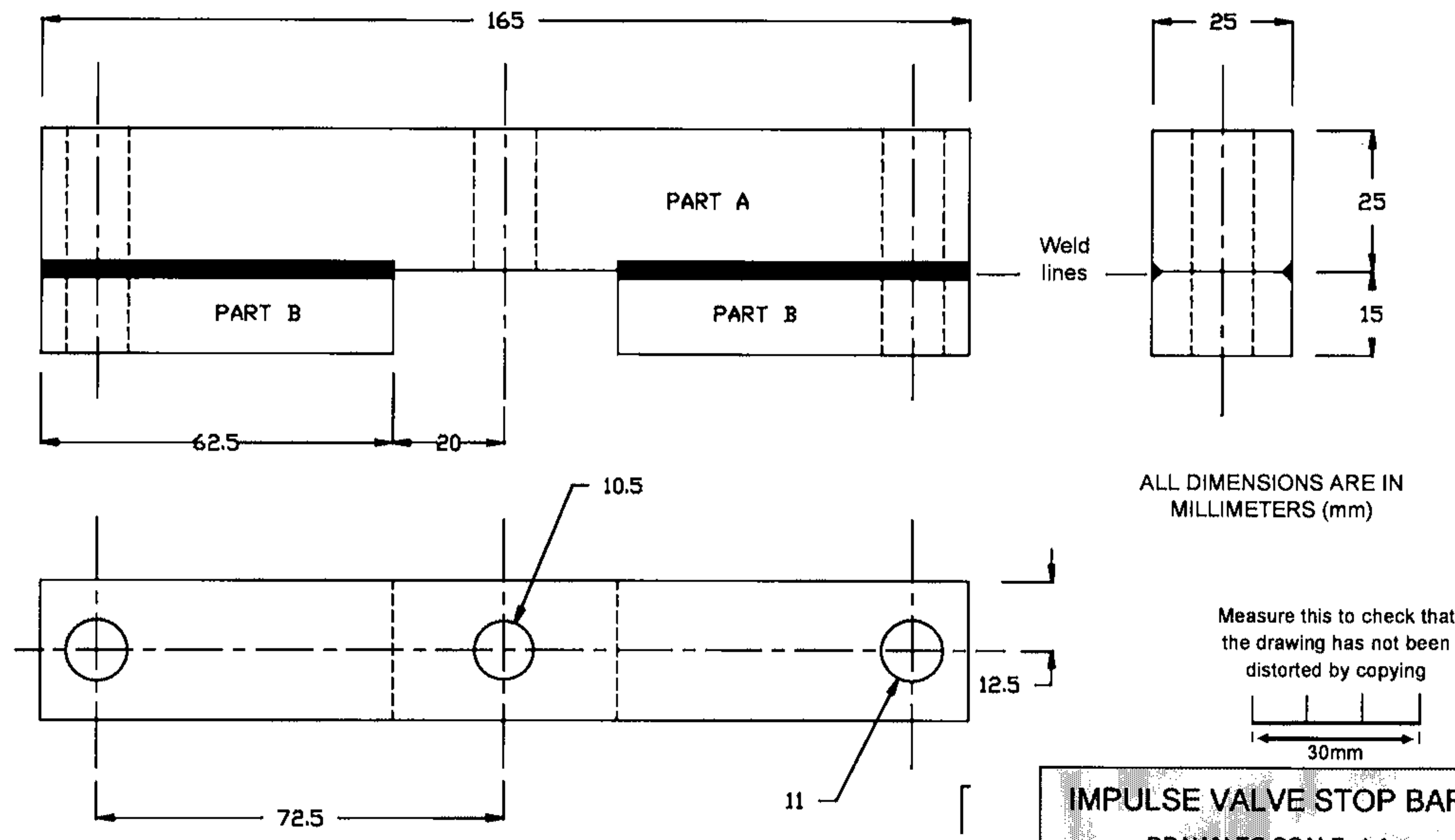
The rubber gasket (9) can be 1.5 to 3mm thick and is cut to match the pump body flange. Drawing number 3 can be used as a template.

Measure this to check that the drawing has not been distorted by copying

ALL DIMENSIONS ARE IN MILLIMETERS (mm)



IMPULSE VALVE PLATE
DRAWING NUMBER 10
PLAN VIEW OF AN IMPULSE VALVE PLATE
Measure this to check that the drawing has not been distorted by copying
30mm
THIS CAN BE USED AS A TEMPLATE
A
A
R72.5
60°
Six 11mm diameter holes are equally spaced around the edge of the valve plate. The diameter of the circle on which they rest is 145mm.
ALL DIMENSIONS ARE IN MILLIMETERS (mm)
5
CROSS SECTION from A to A on the plan drawing
165
66
45°
10 or 12



ALL DIMENSIONS ARE IN MILLIMETERS (mm)

NOTES

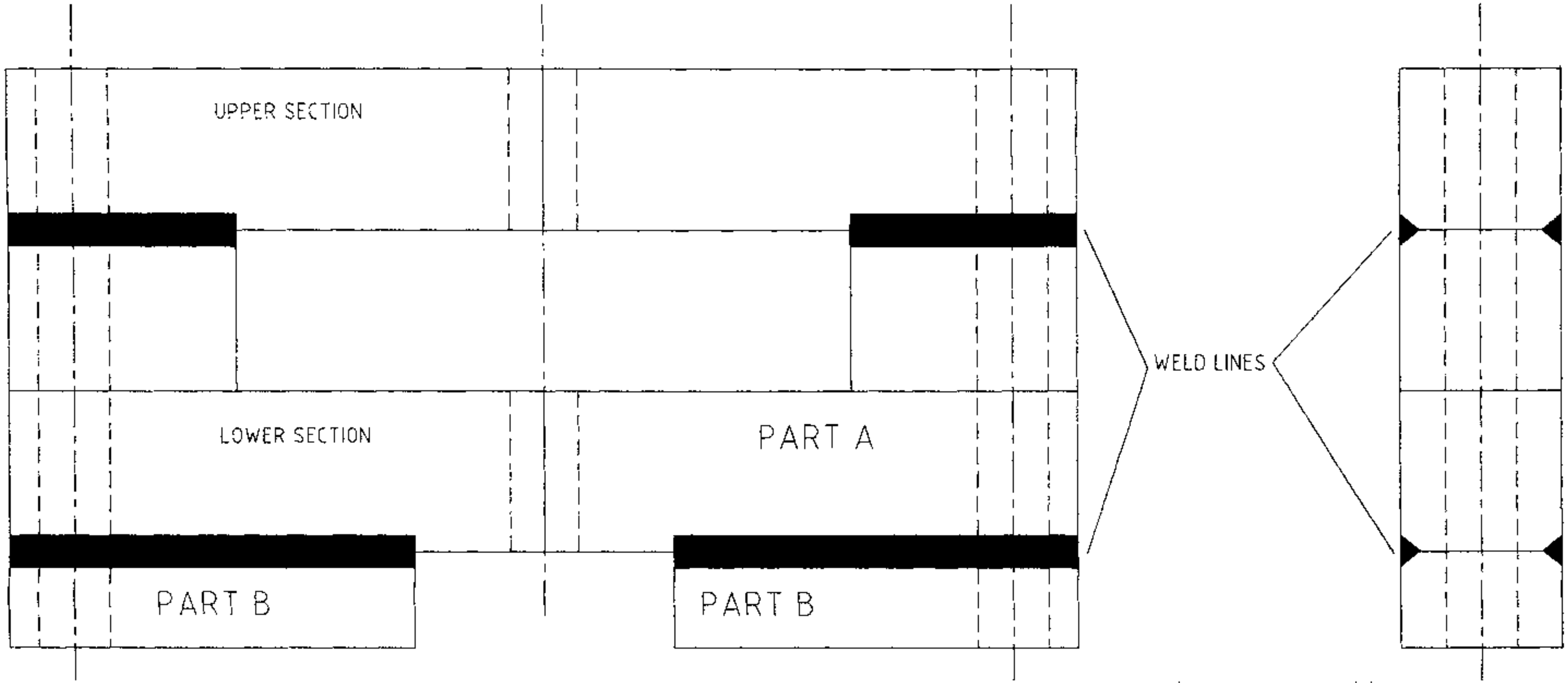
If you cannot get the right size of bar, the height of Part B must be a minimum of 15mm. Part A should be 25mm high, but sizes down to 20mm can be used. The width of Parts A and B should be 25mm but a width down to 20mm can be used.

Make the bar from mild steel. Only use stainless if it is available and you have the tools.

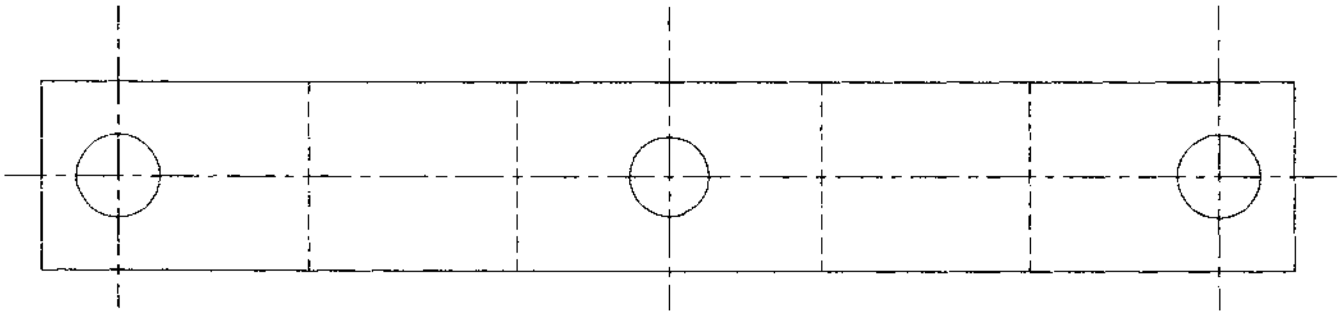
IMPULSE VALVE STOP BAR

DRAWN TO SCALE: 1:1

DRAWING NUMBER 11



Lower section as per
drawing No. 11
Upper from 25 x 25 mm
and drilled to match lower
assembly

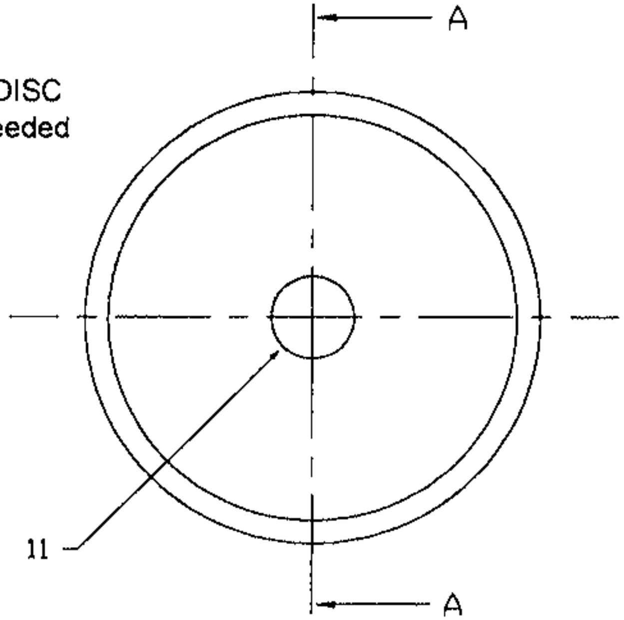


Outer Holes 11.0 mm dia. Middle Hole 10.5 mm dia.

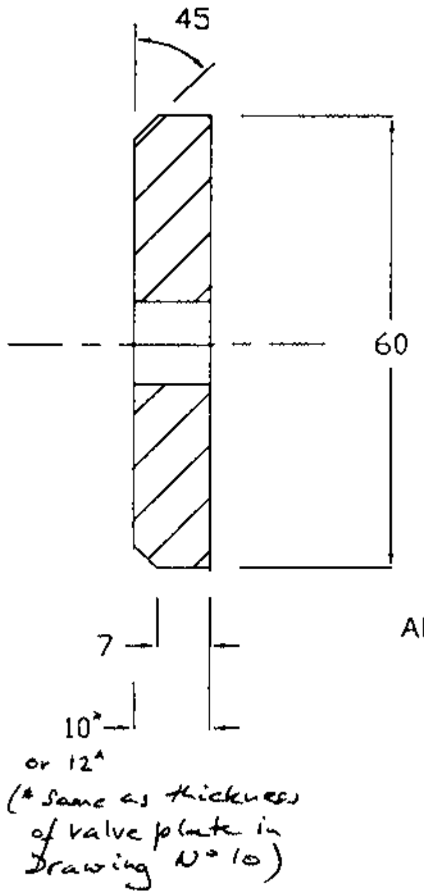
TWO STAGE IMPULSE
VALVE STOP BAR/GUIDE

DRAWING NUMBER 11a
DRAWN TO SCALE 1:1

PLAN OF AN
IMPULSE VALVE DISC
Two of these are needed
for each valve

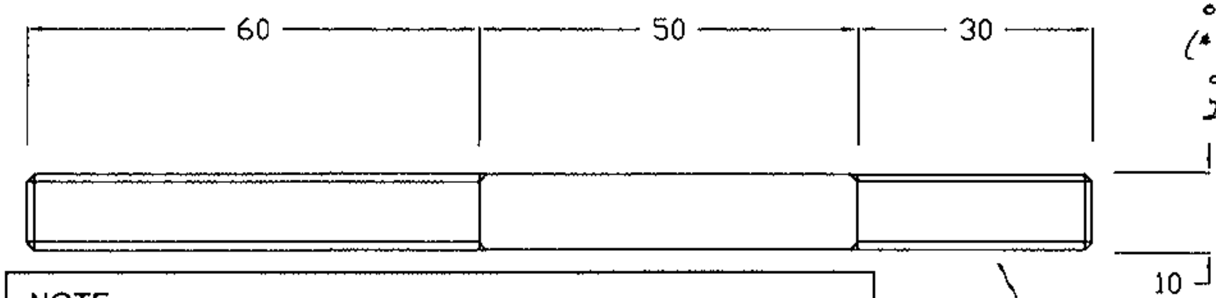


CROSS SECTION
From A to A on the
Plan drawing



ALL DIMENSIONS ARE IN
MILLIMETERS (mm)

IMPULSE VALVE STEM



Measure this to check that
the drawing has not been
distorted by copying

NOTE

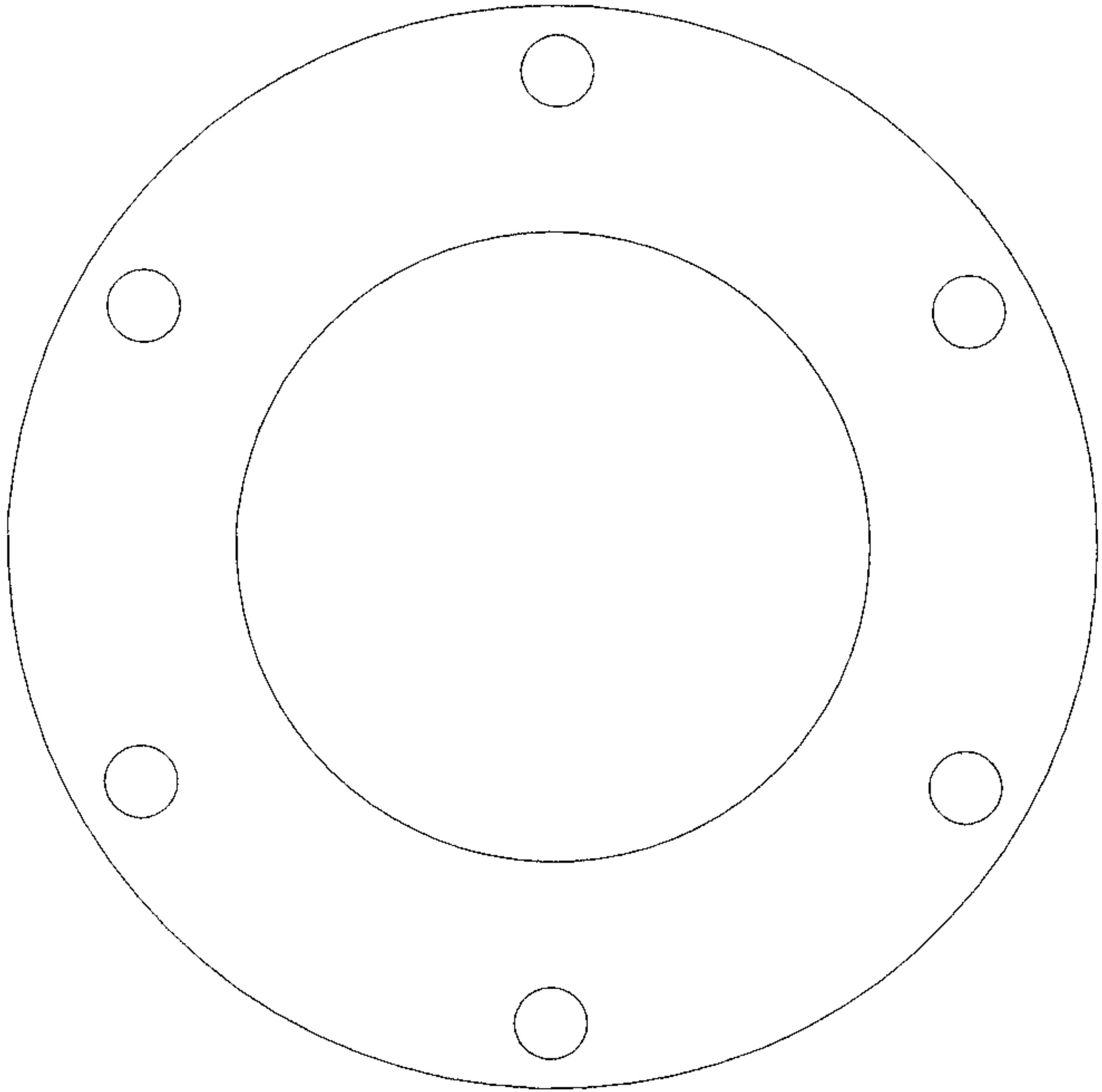
The valve stem is made using either 10mm diameter steel bar or 10mm diameter reinforcing bar. Use stainless if you can.

The thread is hand turned using an M10 x 1.5 die.

DTU S2 IMPULSE VALVE DISCS AND STEM

DRAWN TO SCALE: 1:1

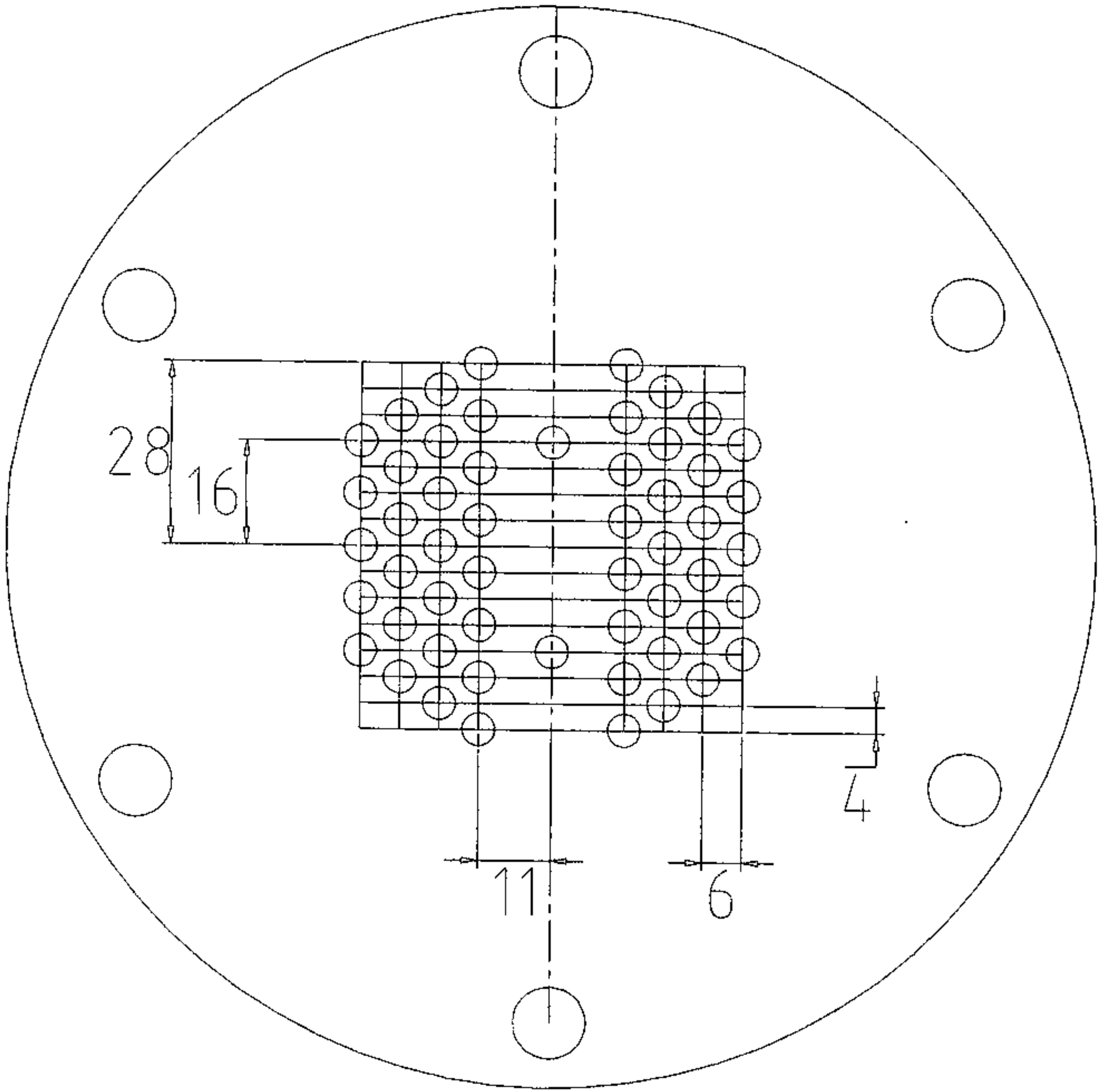
DRAWING NUMBER 12



Inner diameter 96.0 mm
Outside diameter 165 mm

DTU S2 GASKET

DRAWING NUMBER 13
DRAWN TO SCALE 1:1



Delivery Holes 52 off
4.5 mm dia. High Head
5.0 mm dia. Low Head

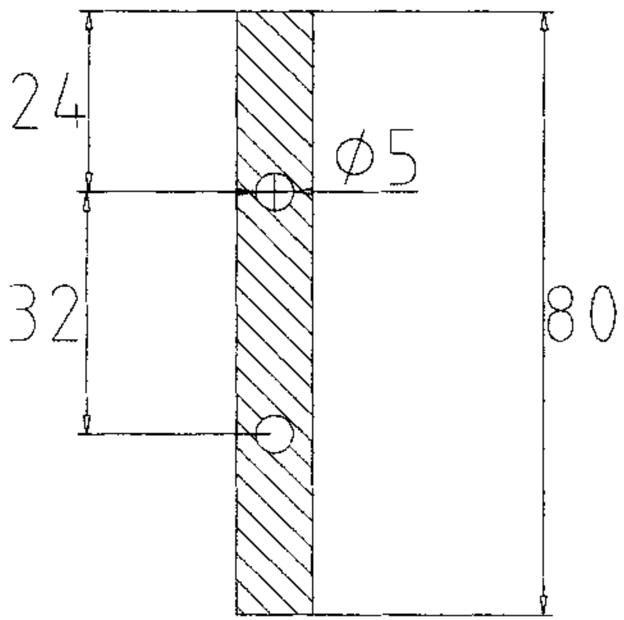
Centre Holes
5.0 mm dia.

PLATE FOR ALTERNATIVE DELIVERY VALVE

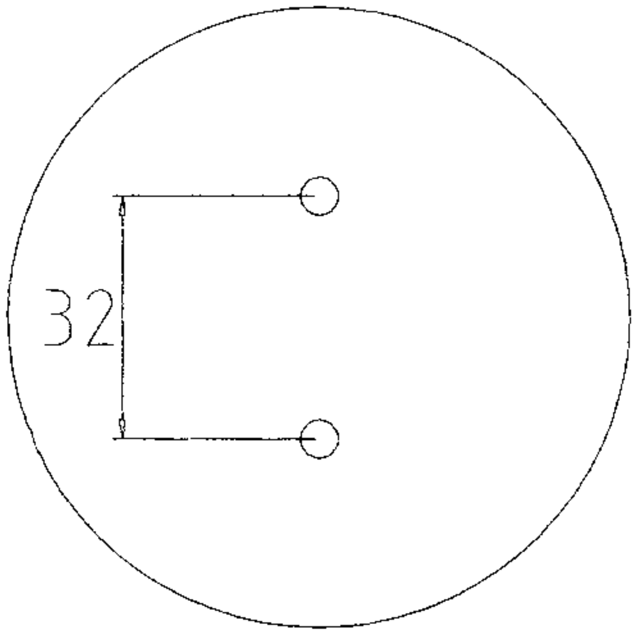
DRAWING NUMBER 14

DRAWN TO SCALE 1:1

Steel Retaining Bar
5 mm thick
10 mm wide



Delivery valve rubber
Diameter 82 mm
3 mm thick for low heads
6 mm thick for high heads



DELIVERY VALVE RUBBER AND RETAINING BAR
FOR ALTERNATIVE DELIVERY VALVE

DRAWING NUMBER 14a
DRAWN TO SCALE 1:1

DTU

Ram Pump Programme

HOW RAM PUMPS WORK

TECHNICAL
15
RELEASE

How ram pumps work

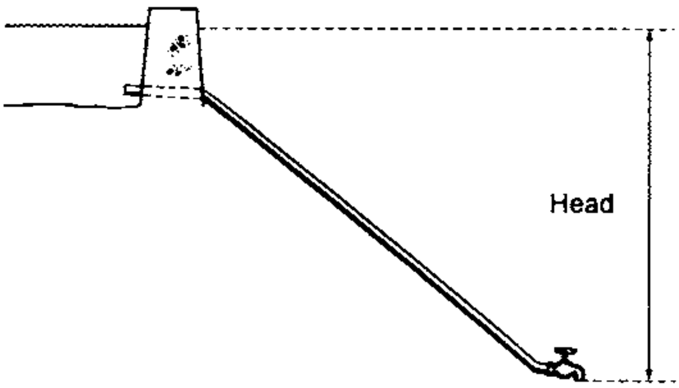
This Technical Release explains what is happening inside working ram pumps. It shows how changes in the design of a ram pump system can change its performance. It will be useful to anyone who is designing ram pump systems and to anyone who is tuning ram pumps to give the best performance.

This section begins with a few examples of simple systems and introduces the words that are used later. It also shows the way that the water flowing through pipes can be shown on a graph. Even if you are already understand terms such as *terminal velocity* in pipes, you should read all of this part so that you know the way that we are using the words.

Falling water

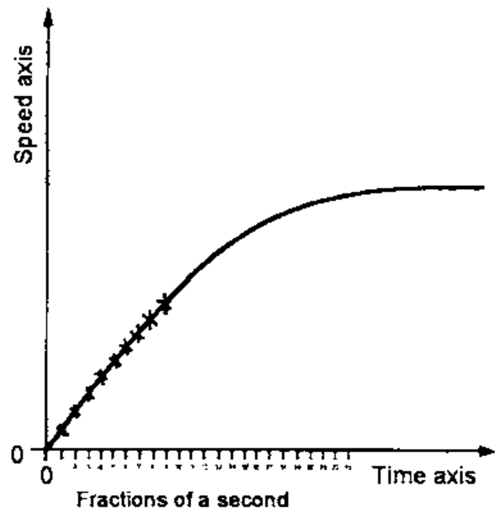
Head

In the drawing here there is a reservoir of water behind a dam. A small pipe passes through the bottom of the dam and runs down the hill. At the end of the pipe there is a tap which is closed. The height from the water level in the reservoir to the tap is called the *head*.



When the tap is opened, water starts to flow down the pipe and out through it. It flows slowly at first, but speeds up very quickly. After a short time the water reaches its maximum speed and the flow becomes steady. If the tap is left open, water will of course continue to flow steadily down the pipe from the reservoir and out through the open tap.

A graph can be drawn to show how the water coming out through the tap gets faster at first, and then flows at its maximum rate. The graph alongside shows what happens. Imagine that a stop-watch was started as soon as the tap was opened and the speed of the water in the pipe was measured every hundredth of a second. The graph has the speed of the water flowing through the pipe marked on the side axis. The time that has passed is marked along the bottom axis.

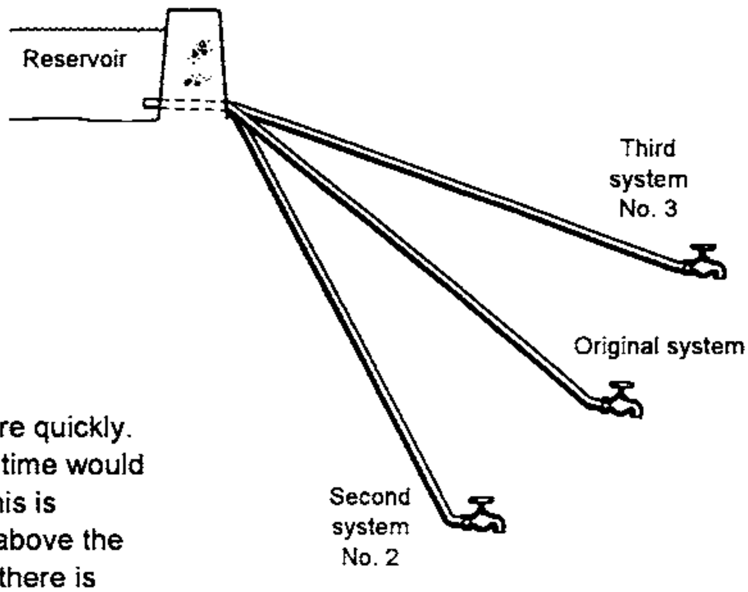


Before the tap was opened and the stop watch was started, there was no water coming through the tap. That is why the graph starts when the "Time" is zero and the "Speed" of the water in the pipe is also zero.

At first the line on the graph goes up very quickly. As the water flows it rubs against the sides of the pipe. This rubbing is usually called *friction*. As the water gets faster, the friction becomes bigger and stops the speed of the water from increasing as quickly. This is shown on the graph by the curve of the line getting flatter and flatter. After a while, the water is flowing as fast as it can and the line on the graph is flat. This shows that the water is continuing to flow out of the tap at the same speed.

A larger head

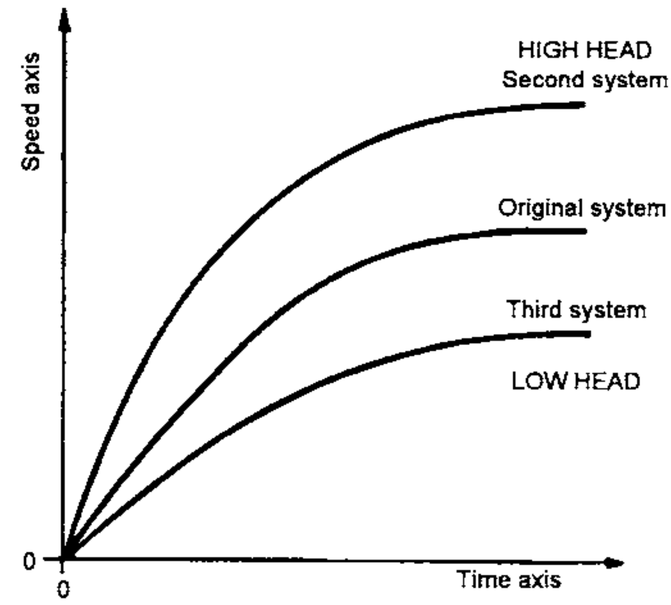
In this drawing there is a water supply with a reservoir and three pipes and taps. The pipes are labeled as the original, second and third systems. In the second system it is further downhill to the tap (the *head* is bigger than in the original system). When the tap on No.2 is opened, the water will flow down the pipe and out through the tap more quickly. The line showing speed against time would rise more steeply on a graph. This is because the reservoir is higher above the tap than it was on System 1, so there is more pressure pushing the water down the pipe and out through it.



As the water gets faster the friction between it and the wall of the pipe gets greater. The curve of the line on the graph begins to flatten out. When the water is flowing as fast as it can, the line on the graph is flat. Because the *head* in the second system is greater than the *head* in the first, the water flows faster in the second. This is shown on the graph at the top of the next page.

A smaller head

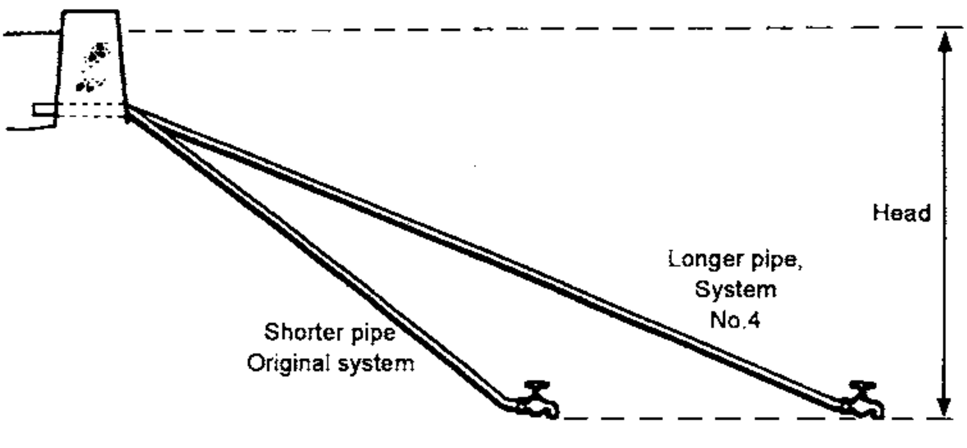
For the third system, the drop from the reservoir to the tap (the *head*) is much less than in the original system. When a graph of System number 3 is drawn, the line on the graph showing speed against time does not rise as sharply and flattens out at a lower speed. This is because there is less pressure pushing the water down the pipe and out through the tap. The speed of the water when it reaches a steady flow is the slowest of the three examples.

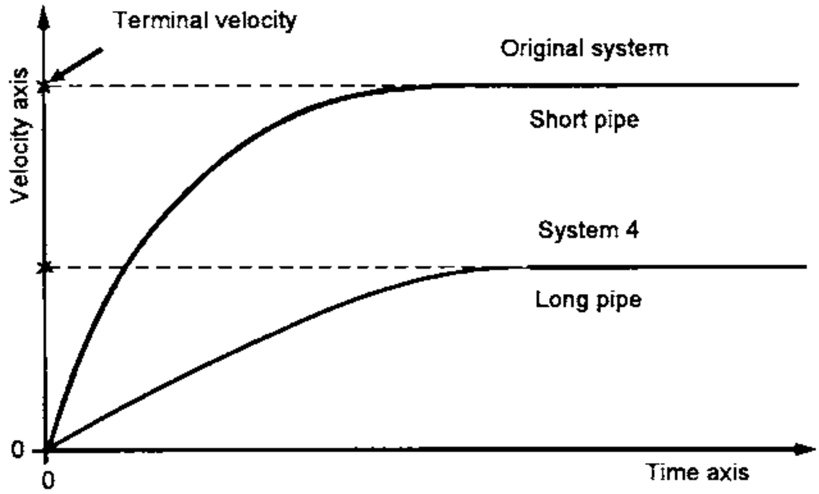


Acceleration

The rate of increase in speed is called the *acceleration*. When the tap is first opened, the speed of the water increases very quickly, so the *acceleration* is very high. As the water flows more quickly there is more friction and the line on the graph flattens out. This shows that the water is still accelerating but that the rate of acceleration is becoming lower. When the water is flowing as fast as it can, there is no more acceleration and the line on the graph is flat. This maximum speed is called the *terminal velocity*. In this case, the word *"terminal"* is used to mean *"as much as possible"*. *"Velocity"* is really just another name for *"speed"*. *"Terminal velocity"* just means *"top speed"*.

In the drawing below another pipe is installed from the reservoir. The new pipe is called System number 4. The pipe is the same diameter as the first one, but it is much longer. The height from the reservoir to the tap (the *head*) is the same. When the tap of this new system is opened, the water accelerates down the pipe and out through the tap. Because the water has to flow through a longer pipe there is more friction to slow it down, so its *terminal velocity* is not as fast.



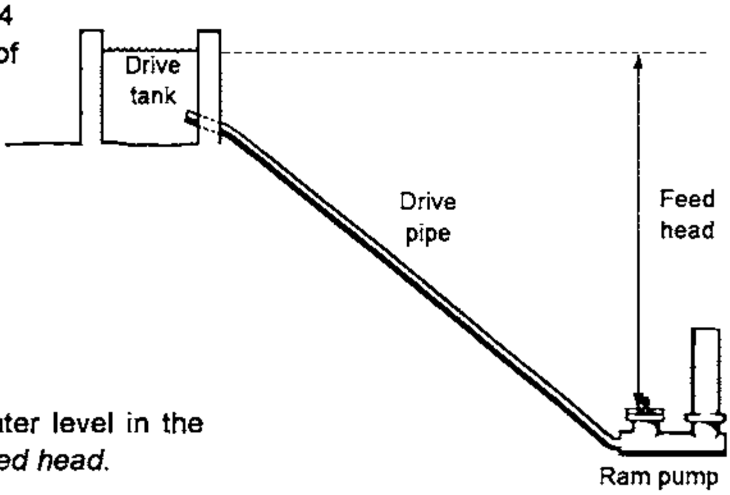


On the graph of this example above, the original system is also shown. The line on the graph flattens out at a lower speed and in a longer time.

If any one of the pipes in these systems was replaced by a pipe with a bigger internal diameter, the graph would change. This is true even if the bigger pipe were the same length and ran along the same route. When the acceleration of the water was measured again, you would find that the water reached a faster *terminal velocity*. This is because the larger pipe would have less friction. In a bigger pipe, there is more room for the water to pass through without touching the sides.

How ram pumps use the water

In the drawings of Systems 1 to 4 there was a reservoir at the top of a pipe with a tap at the bottom. From now on the reservoir is replaced by a tank and the tap on the end of the pipe is replaced by a ram pump.

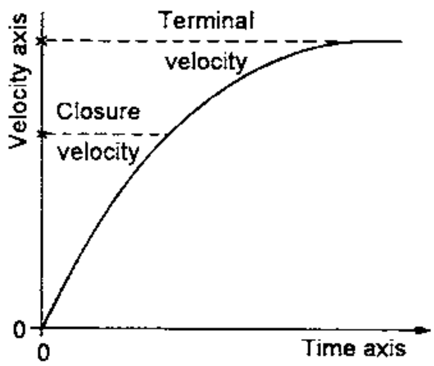


Feed head
The drop in height from the water level in the tank to the pump is called the *feed head*.

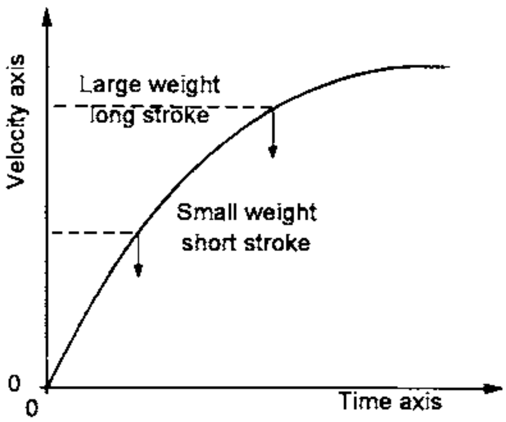
Closure velocity
When the ram pump is not working and the impulse valve is closed, there is no water coming through it. When the impulse valve is pushed down, water begins to come out through the open valve, getting faster as it accelerates down the pipe. The force of the water flowing around the impulse valve tries to lift it closed but the weight of the valve keeps it open. The force of the water trying to close the valve gets bigger and bigger.

When the force trying to close the valve becomes bigger than the weight, the valve starts to move upwards. All impulse valves will close when the drive water reaches the right velocity. What that velocity is will depend on how the impulse valve on the particular pump is tuned. The water accelerates down the pipe and out through the impulse valve until it reaches the *closure velocity* and the valve closes.

A graph can be used to show the increasing velocity in the system just as it was for the examples with the tap. If the impulse valve was held open and water allowed to flow out for a long time the graph would show it reaching a *terminal velocity* just as before. The shape of the line on this graph would depend on the feed head, the pipe diameter, the pipe length and the amount of friction through the pump. If the pump was operating normally, the impulse valve would close when the velocity reached the *closure velocity*. This point can be marked on the graph.

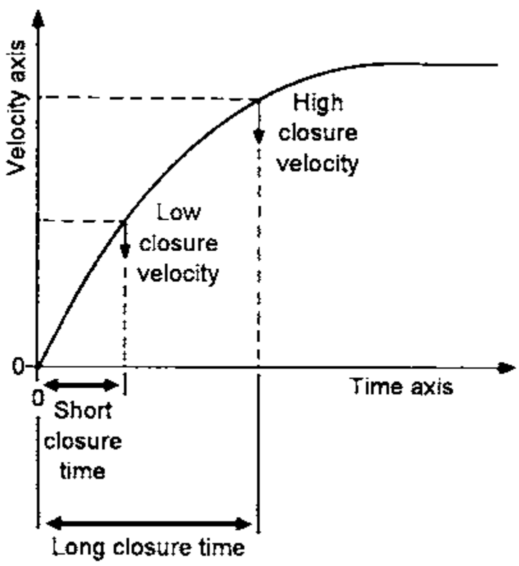


At any ram pump site the settings of the impulse valve (the stroke length and the valve weight) will decide how big the *closure velocity* is. A valve with a small weight and a short stroke has a low closure velocity. It will close when the water reaches only a low velocity. A valve with a large weight and a long stroke will need a much higher velocity to make the valve close. The *closure velocity* of the pump can be changed by making adjustments to the impulse valve — in other words, by *tuning* the pump.



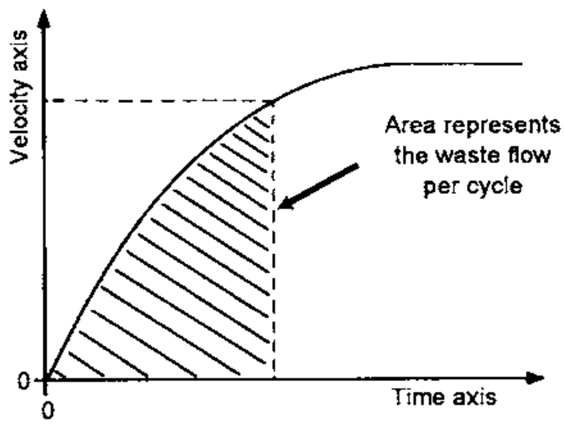
The time it takes for the water to reach the *closure velocity* can be worked out using the same graph. This is done by drawing a straight line across from the Velocity (Speed) axis at the *closure velocity* until it reaches the line on the graph. Another line is drawn from this point straight down to the bottom of the graph where it hits the Time axis. The scale on the Time axis can then be read to find out how long it takes to reach *closure velocity*.

You can see that if the pump has a high *closure velocity*, it will take a long time for the valve to close. If the stroke of the impulse valve was reduced, the velocity needed to close the valve would also be reduced and the time it would take to reach the new *closure velocity* would be less. So, reducing the length of the impulse valve stroke on a pump will make it work faster.

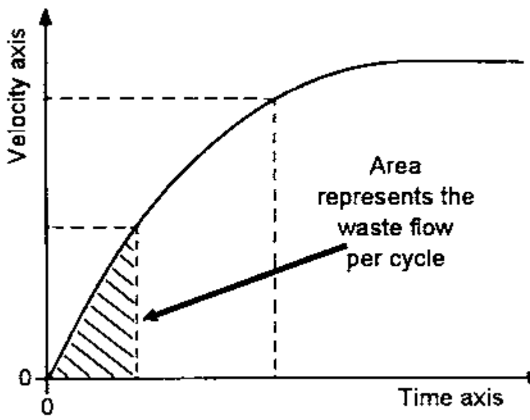


Closure velocity and waste flow

When a pump has a high *closure velocity* a lot of water will flow through the impulse valve before it shuts. This water is often called the *waste flow* for one cycle and can be represented by the area under the graph that has been shaded.

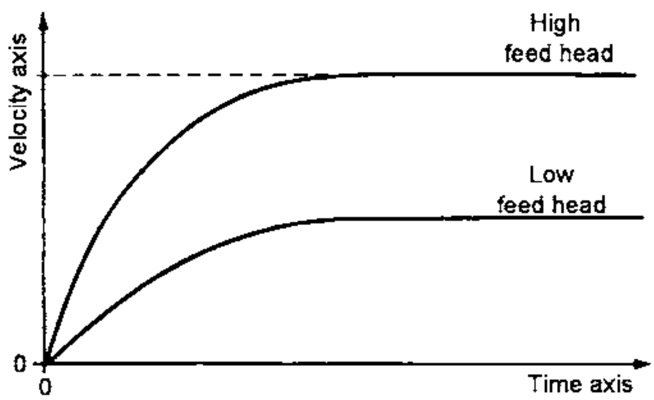


By shortening the length of the impulse valve stroke, the pump can be made to work faster and the *closure velocity* will be lower. The area representing the *waste flow* on the graph will be smaller, showing that the amount of water flowing through the pump (the *waste flow*) has gone down.

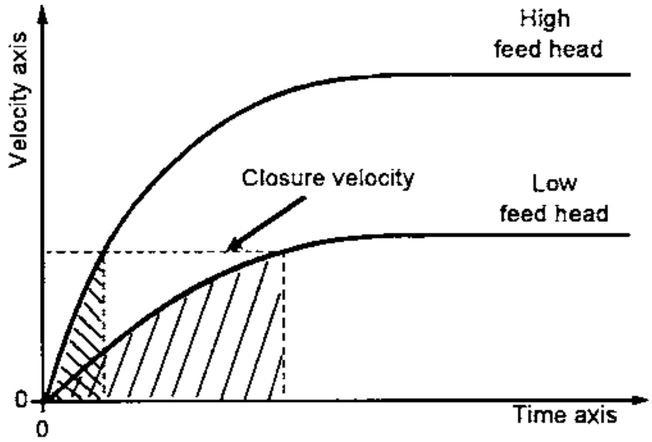


Closure velocity and feed head

You saw earlier with the reservoir, pipe and tap that when the head from the reservoir to the tap was increased, the acceleration was faster and the line on the graph rose more steeply. If the *feed head* of the ram pump system is increased, the line on the graph also rises more steeply.

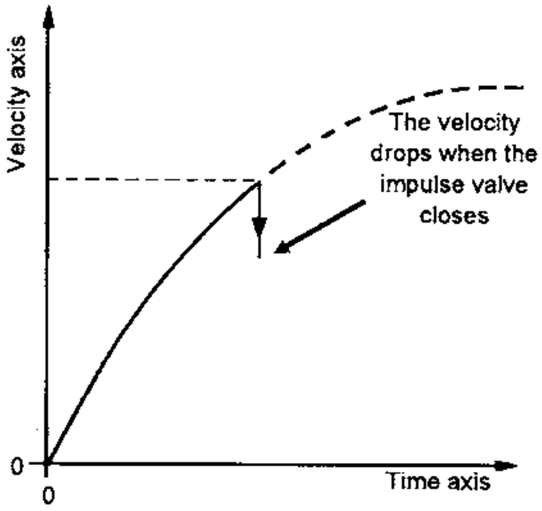


If the same ram pump system was given a different feed head, the *closure velocity* for the system would stay the same as long as the same pump was used and its tuning was not changed. The area under the line on the graph for the system with the higher feed head would be much less, showing that the waste flow was smaller. It would also take less time to reach the *closure velocity*, so the pump would work faster.



Water delivered from a ram pump

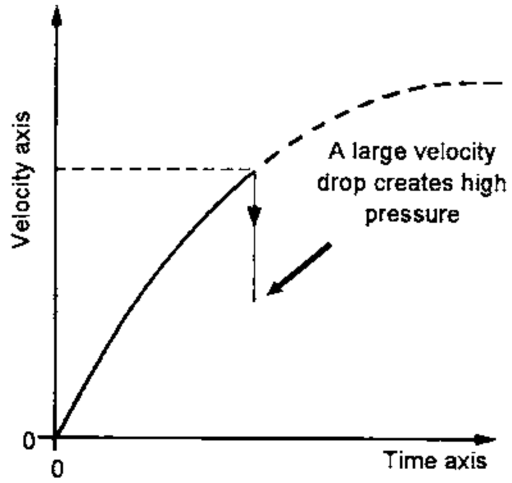
When the impulse valve on a ram pump closes, the water flowing down the drive pipe and through the pump has nowhere to go. The velocity of the water drops very quickly. As the velocity of the water drops, the energy of the moving water is turned into pressure energy. The more the velocity of the water drops the more the water pressure inside the pump rises.



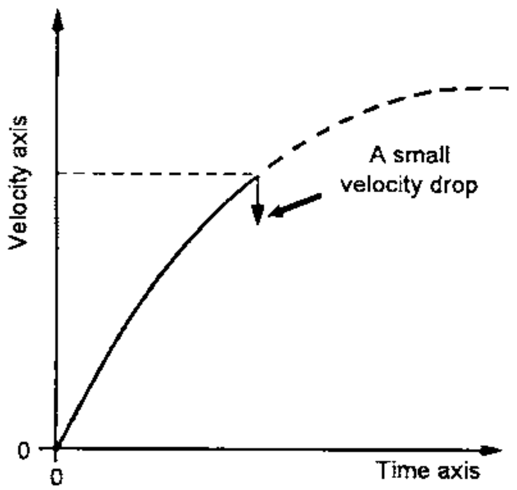
Delivery head

The velocity of the water continues to fall until the pressure in the ram pump is higher than the pressure in the air vessel. The pressure in the air vessel will depend on the height from the pump to the top of the delivery pipe. The height from the pump to the top of the delivery pipe is called the *delivery head.*

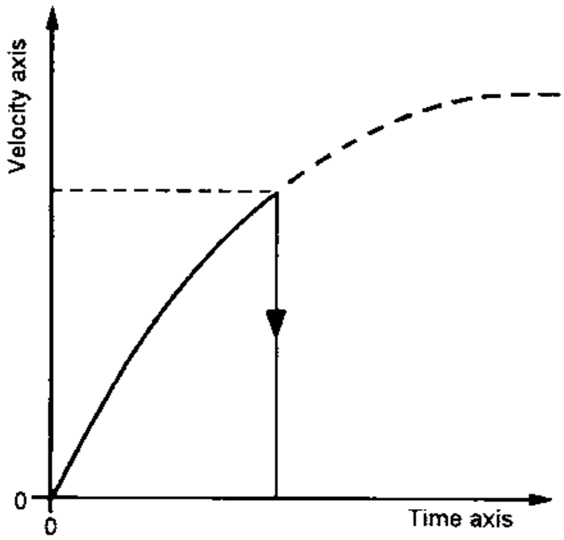
When the *delivery head* is high, there has to be a big drop in the velocity before the pressure in the pump is bigger than the pressure in the air vessel.



When the *delivery head* is small, the velocity only has to drop a small amount before the pressure in the pump is higher than the pressure in the air vessel.

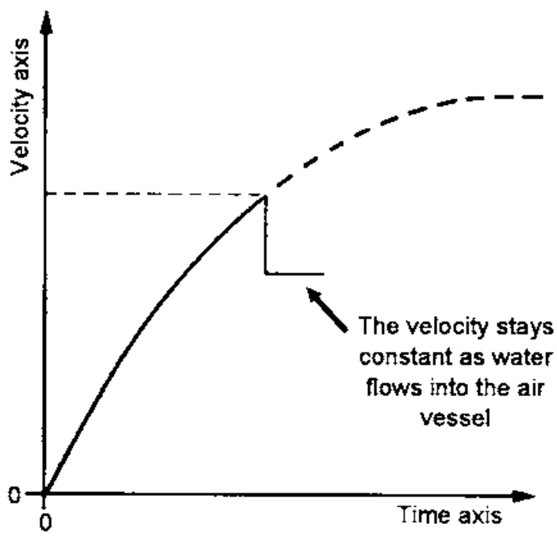


If the delivery head is very high, the closure velocity may be too small. When it is too small, there is not enough velocity to turn into pressure energy when the impulse valve closes. The pressure in the pump rises, but not enough to reach the delivery pressure. The water loses all of its velocity but the delivery valve does not open. The impulse valve may then open again and water flow through, allowing the velocity of the water to increase until the impulse valve closes again. When this happens a pump can work with the impulse valve opening and closing but without the delivery valve opening, so no water is pumped.



Delivery pressure

When the pressure in the pump is greater than the pressure in the air vessel, the delivery valve opens. Water flows through the delivery valve and into the air vessel. The water has lost velocity and gained pressure. It flows at the lower velocity into the air vessel. Water will continue to flow through the delivery valve at this velocity for a certain period of time. The amount of time will depend on the time it takes for a shock wave to travel up the water in the drive pipe to the drive tank and back down to the pump again.



This is more fully explained in the box on the next page. It is not necessary to understand all the details so you can miss out the box if it seems too complicated. All you need to know is that the water flows through the delivery valve at the reduced velocity for a certain period of time. The length of time will depend on the length of the drive pipe. If the drive pipe is short the period of time will be short. If it is long then the period of time that the water flows through the delivery valve will be longer. Remember that the pump is likely to be cycling every second or so, so the periods of time are all only fractions of a second.

More advanced explanation

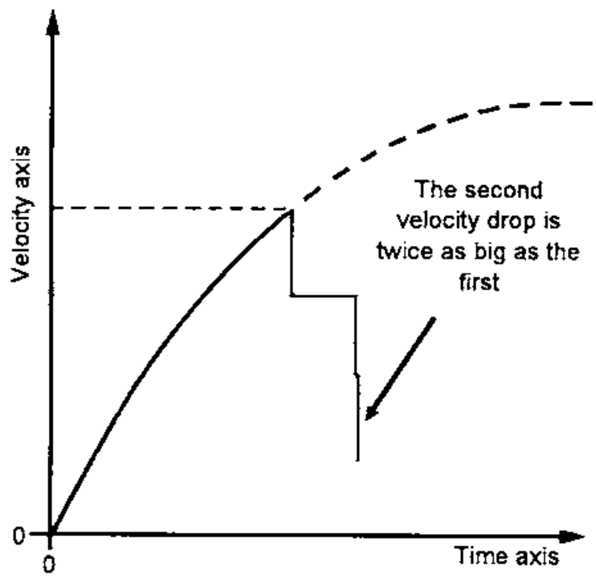
When the impulse valve closes, the water moving inside the pump suddenly slows down. When the first bit of water starts to slow down, its pressure rises. The water behind it also slows down and its pressure also starts to rise. Pressure rises along the drive pipe as all the water slows down and its pressure rises.

Try to imagine a line between the water in the pipe that has slowed down and the water that is still moving at full speed. This imaginary line travels very quickly from the impulse valve up the drive pipe to the drive tank. On one side of the line the water has slowed down and the pressure has risen. On the other side the water is still moving at closure velocity. This imaginary line is at the front of a "shock wave". The drop in velocity of the water across this shock wave is the same as the drop in velocity needed to open the delivery valve.

The shock wave travels at very high speed (usually over 1000 meters per second) up the drive pipe until it reaches the drive tank. The drive tank acts a bit like a mirror that turns things around and reflects them back. It is a low pressure shock wave which travels back down the drive pipe towards the pump.

The time taken for the shock wave to travel from the pump to the drive tank and back again will depend on the length of the drive pipe and the speed of the shock wave. The pressure in the pump, and the velocity of the water flowing through the delivery valve will remain constant until the shock wave has gone up the drive pipe, been reflected and come back down to the pump. The graph below shows that the velocity of water in the pump stays constant for a set period of time. The time will depend on how long it takes the shock wave to go up and down the drive pipe.

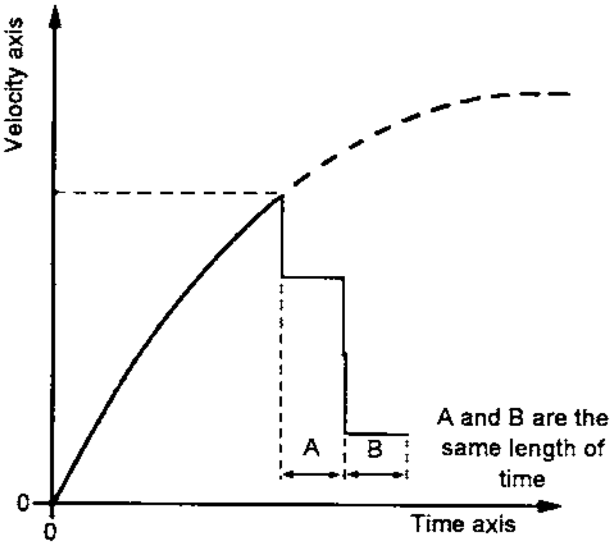
When the shock wave in the drive pipe has returned to the pump, the velocity of the water drops again. This keeps the pressure in the pump high enough to push water through the delivery valve into the air vessel. The second velocity drop is twice as much as it was the first time.



More advanced explanation

When the reflected shock wave reaches the pump the velocity there drops due to the lower velocity of the water *behind* the shock wave. It immediately drops again to recreate the pressure needed to keep the delivery valve open. Both of these drops in velocity are the same size as the first velocity drop when the impulse valve closed.

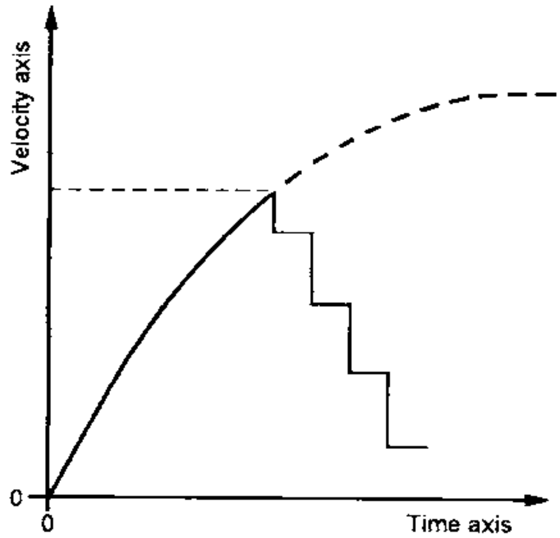
Water continues to flow through the delivery valve at the reduced velocity. It flows for the same amount of time as before, while the second shock wave travels up to the drive tank and back again.



More advanced explanation

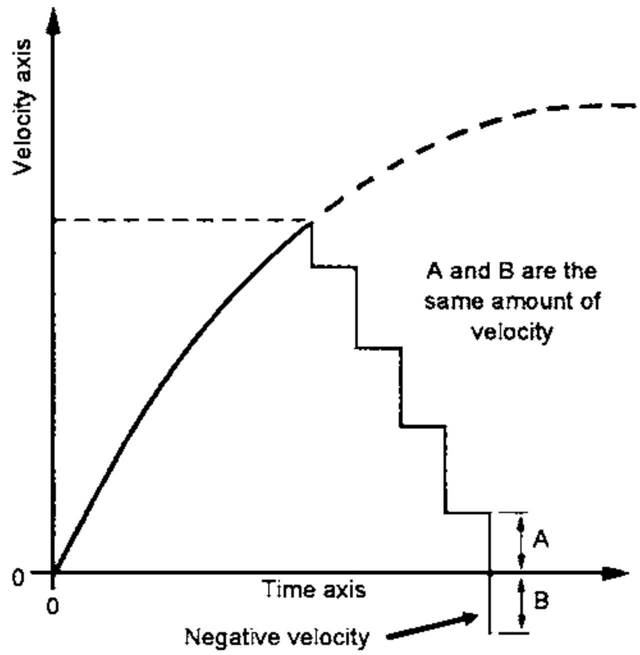
The second part of the second drop in velocity produces another increase in pressure that keeps the pressure in the pump greater than the pressure in the air vessel. Water flows through the delivery valve at the lower velocity. The increase in pressure starts another shock wave that travels up the drive pipe. The second shock wave is also reflected by the drive tank and another low pressure shock wave travels back down to the pump. Water continues to flow from the pump into the air vessel until the reflected shock wave reaches the pump.

Each time a shock wave comes back to the pump, the water's velocity drops again and a new shock wave travels up the drive pipe. Water continues to flow into the air vessel at the reduced velocity. The velocity in the pump continues to step down in this way until there is not enough velocity left to make another full step.



Recoil

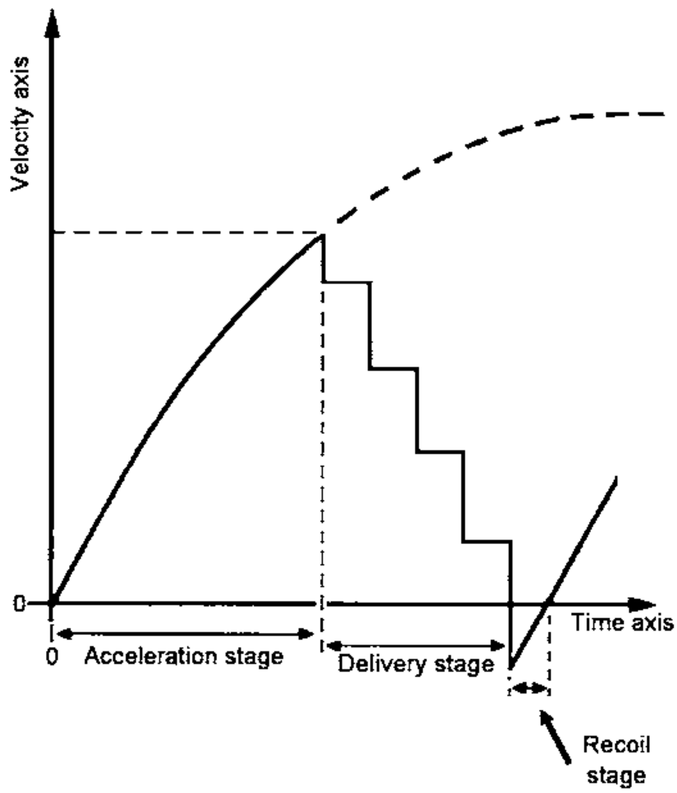
When there is not enough velocity left to make one complete step the pressure can no longer stay high enough to keep the delivery valve open, so it shuts. The water in the system is still moving with a low velocity down the drive pipe. The impulse and delivery valves are both closed so the water has nowhere to go. The water bounces back up the drive pipe towards the drive tank. The water is like a spring that is pushed against a wall so that it compresses and then is suddenly let go. It opens up and flies away from the wall.



As the water moves away from the pump it has a negative velocity, so the line on the graph goes under the base (under 0 on the Velocity axis). The negative velocity will be the same size as the positive velocity that was left when the delivery valve closed. When the water has a negative velocity it moves away from the pump towards the drive tank.

The negative velocity is shown where the line on the graph goes below 0 on the Velocity scale. The water flowing back up the drive pipe slows down and stops, then the line on the graph crosses back over the Time axis. After that, the impulse valve has reopened and the water begins to accelerate down the drive pipe. The line on the graph rises in the same way as before. The section of the graph below the line is called the "recoil" stage of the pump's cycle.

When the delivery valve closes and the water in the pump *recoils* back up the drive pipe, it is actually flowing away from the pump. This leaves a low pressure or vacuum in the pump. The low pressure lets the impulse valve drop open with its own weight.

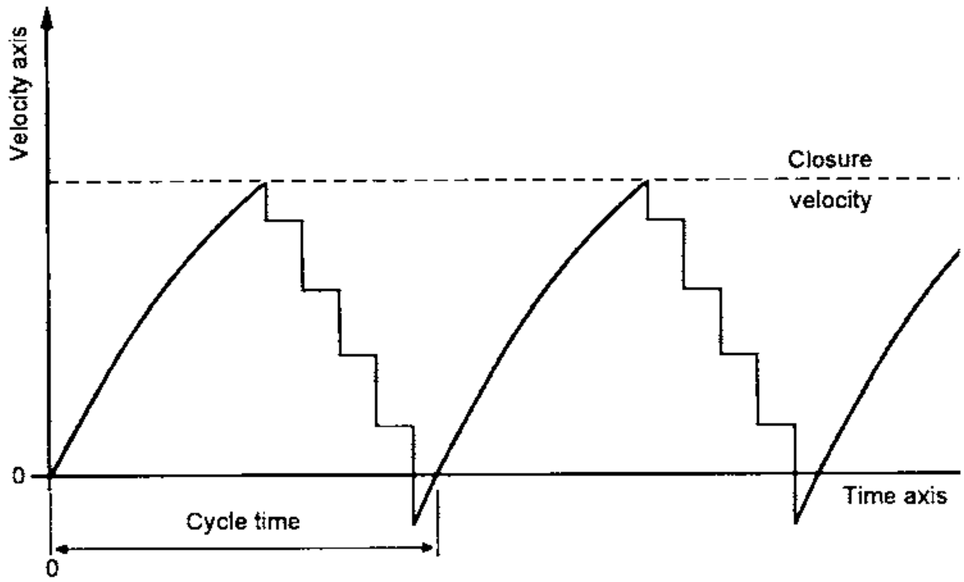


When the water has stopped recoiling, it starts to flow down the drive pipe and out through the open impulse valve again. It accelerates down the drive pipe and through the open valve as it did at the start of the pumping cycle.

The pump cycle

To make the operation of the pump easier to understand the pump cycle can be divided into three stages. They are called the *acceleration, delivery and recoil* stages. There is a very short pressurisation stage as well, but that can be ignored. The graph at the bottom of the previous page shows how these stages refer to different parts of the cycle.

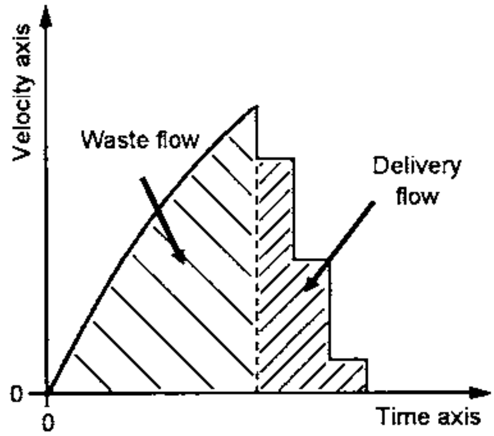
As the pump continues to operate, the same cycle is repeated. The water accelerates down the drive pipe and out through the impulse valve until the velocity of the water reaches closure velocity. Then the stages of delivery and recoil occur before the cycle starts again. The graph can be used to see the time taken for each cycle and work out how fast the pump is operating.



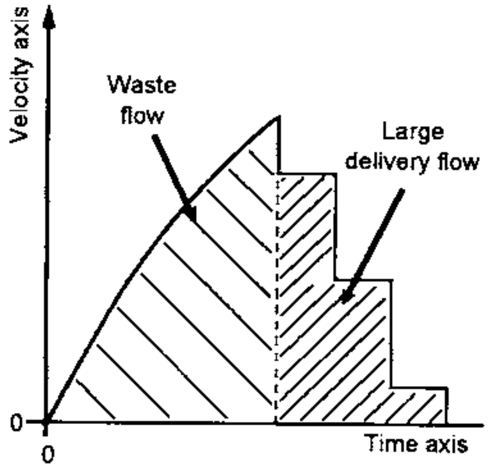
The impulse valve closes and the pump beats every time the line on the graph reaches closure velocity. For a pump working at 60 cycles a minute, the "cycle time" would be 1 second.

Delivery flow

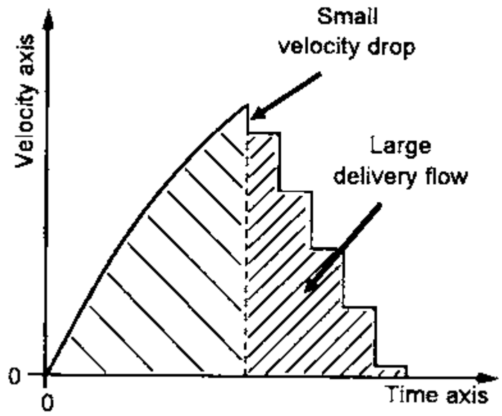
You saw on page 6 how the amount of waste flow could be represented by shading the area under the line on the graph. In the same way, the amount of water flowing from the pump into the air vessel can be represented by the area under the graph during the delivery stage. This is called the *delivery flow* per cycle.



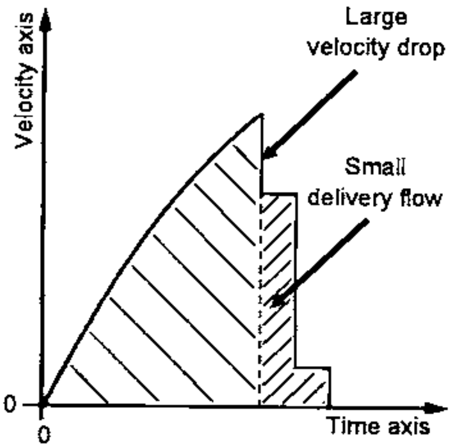
When the area under the falling part of the graph is large it means that there is a large *delivery flow*. Two things increase this area. One is increasing its height. The other is increasing its width either by having more steps or by having longer steps.



Systems with a low delivery head will only need a small drop in velocity to deliver water. The delivery stage will have many small drops in velocity before all the velocity is used up. The large area under the line on the graph shows that a large amount of water will be pumped with each cycle.



If the delivery head of the same system was increased, the acceleration of the water and the closure velocity would be the same but the drop in velocity required to reach the delivery head would be much higher. The graph shows that there would be a few large drops in velocity during the delivery stage of the cycle. The area under the graph during the delivery stage would be much smaller, showing that the flow through the delivery valve was small. Much less water would be pumped with each cycle.

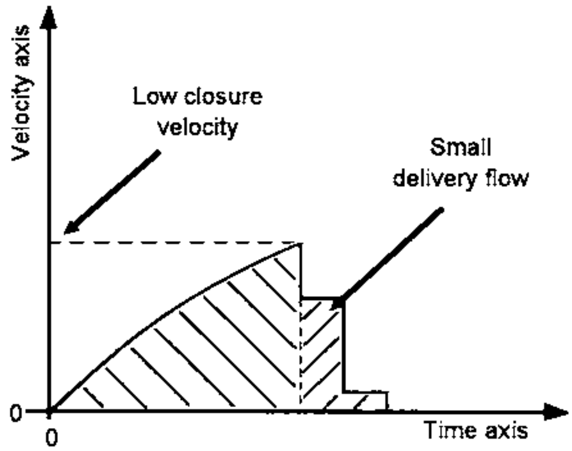


More advanced explanation

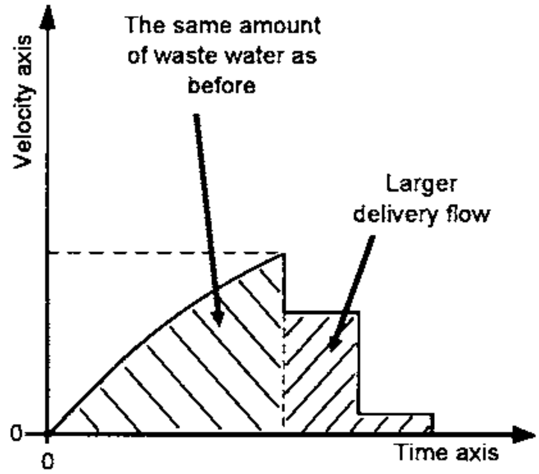
Because the drive pipe is the same length in both of the systems just described, the time taken for a shock wave to travel from the pump to the drive tank and back again would be the same. So the flat sections of the graphs, showing when the water is flowing at a steady velocity through the delivery valve, would each last the same length of time. The delivery stage of the pump cycle in a system with a low delivery head would be longer than it would be in the same system with a higher delivery head. The acceleration time would be the same in both cases and the time for the recoil stage would be very small. When the delivery head is increased, the length of the delivery stage is reduced and the pump operates faster.

Getting the best delivery flow

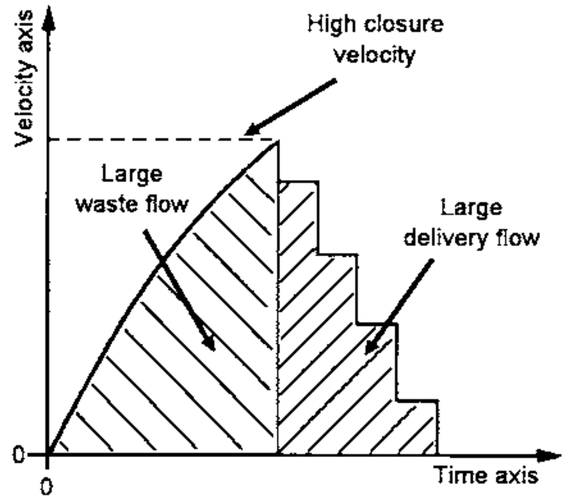
This is explained using an example. Suppose you are asked to design a ram pump site on a stream where the flow of water in the dry season is quite small. There is an obvious ram pump site but it only has a small drop, so the feed head would also be small. If a pump was installed there, the acceleration of the water would be quite low and so the graph would not rise very steeply. The impulse valve of the pump could be adjusted to use all the water available in the stream. With a low feed head, the acceleration and closure velocity would also be low. The pump would not deliver much water.



The owner of the site says that he needs more delivered water than the proposed site would give, so you have to look more carefully at the site design. Suppose that you find that you can double the feed head without too much trouble. Normally the drive pipe length will also double. By keeping the closure velocity the same, the feed flow will hardly change. The delivery head would be increased by an amount equal to the extra feed head, but this has little effect on the flows. Although the drops in velocity would be about the same as before, the steps are twice as wide and much more water would be delivered.



If the same site has plenty of water available in the rainy season, you could recommend changing the tuning of the impulse valve when the stream is full so that the closure velocity was increased. The valve would then take longer to close and much more water would flow through it. The delivery head would be the same as before so the drops in velocity would be the same size. Because the closure velocity would be higher there would be more drops in velocity during the delivery stage and more water would be pumped.



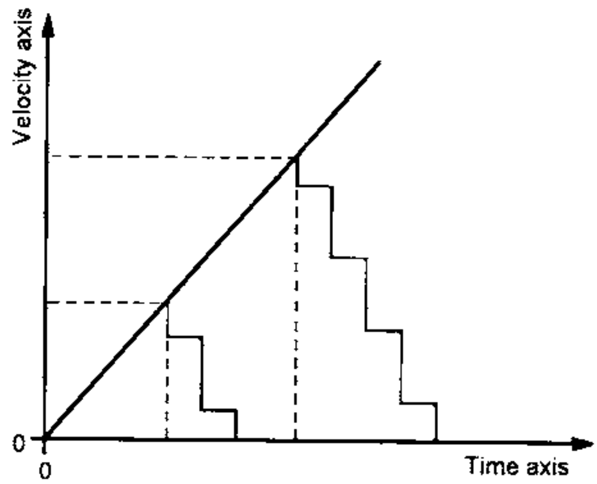
More advanced explanation

As the closure velocity is increased (by changing the settings of the impulse valve) three things happen:

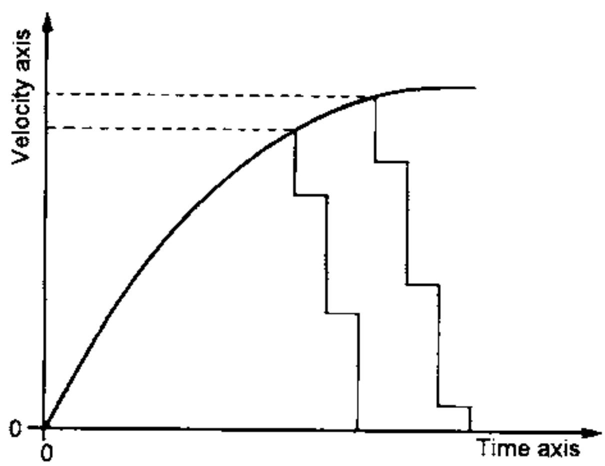
- the drive flow per cycle increases a lot
- the delivery flow per cycle increases a lot
- the pump works a little more slowly (fewer cycles per minute)

The overall effect is to increase both the delivery flow per minute and the drive flow per minute.

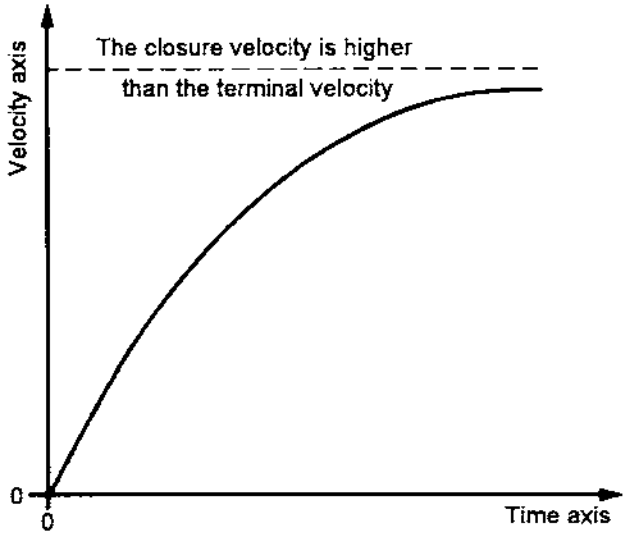
When the closure velocity is in the steep section of the line on the graph, increasing the closure velocity will increase the delivery flow per cycle and the total water delivered by the pump.



If the impulse valve was tuned so that the stroke was very long or the weight very heavy, the closure velocity would be in the flatter section of the line on the graph. The drive flow through the pump would increase a lot but the delivery flow would change very little. The time taken for each cycle would also increase so that there would be fewer cycles per minute. The pump would become less efficient and would actually pump less water per minute.



Increasing the weight and stroke of the impulse valve too much would stop the valve from closing at all. This is because the valve cannot close when the velocity needed to close it is more than the terminal velocity of the water through the pipe.



The following DTU Technical Releases give further information about ram pumps.

TR 11: The DTU S1 ram pump

TR 12: The DTU P90 ram pump

TR 14: The DTU S2 ram pump

TR 16: An introduction to hydraulic ram pumps (and the DTU range)

DTU

Ram Pump Programme

AN INTRODUCTION TO HYDRAULIC RAM PUMPS
(and the DTU range)

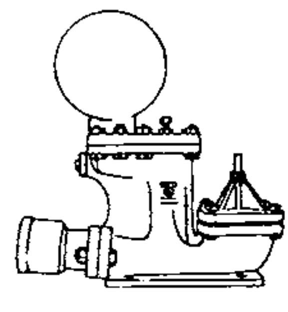
TECHNICAL
16
RELEASE

An introduction to hydraulic ram pumps

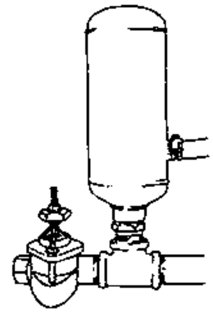
This Technical Release explains briefly what ram pumps are, how they work, and how you might select them. It finishes with a catalogue of the current DTU designs of pump for local manufacture in developing countries.

Contents

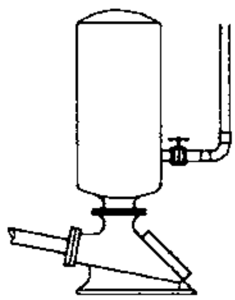
A much fuller description of how to design and install ram pumps systems can be found in the book *Ram pumps and the design of ram pump systems*, DTU, UK 1995, which is available from the DTU at the address on the back cover of this Technical Release. A list of Working Papers and other Technical Releases, several of which refer to ram pumps, can also be found on the back cover of this Technical Release.



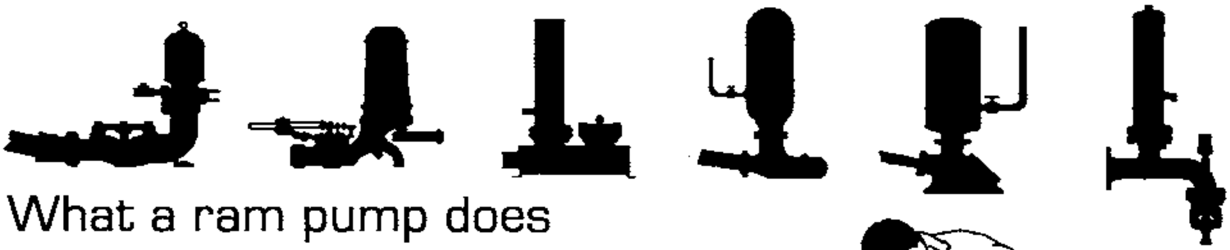
AN EASTON RAM PUMP
UK



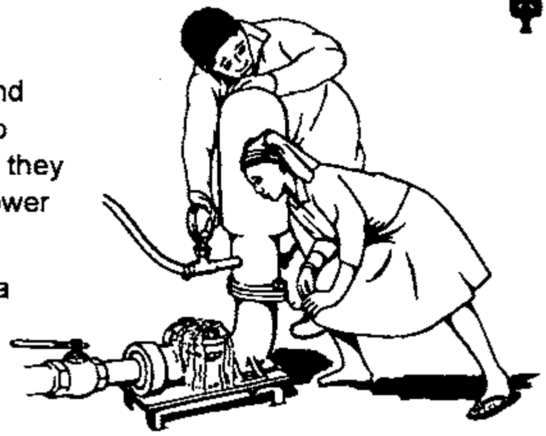
A GAVIOTAS RAM PUMP
Columbia



A "PREMIER" RAM PUMP
India



What a ram pump does

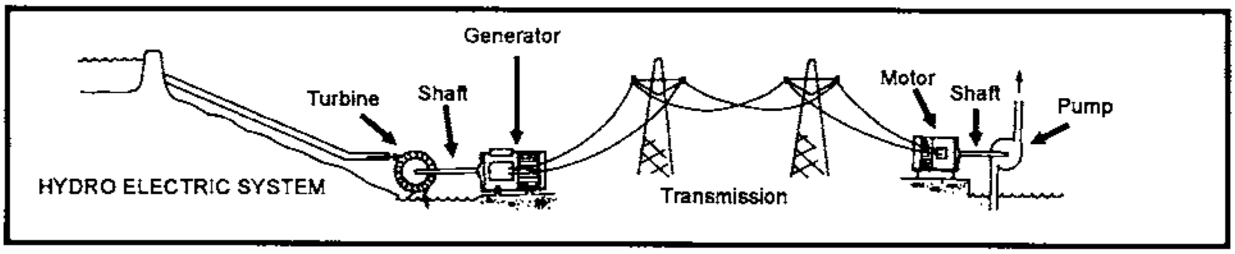


Some ram pumps are made from many parts and look very complicated. Others look too simple to work. Whether they look simple or complicated, they all work in the same way and they all get the power to run in the same way.

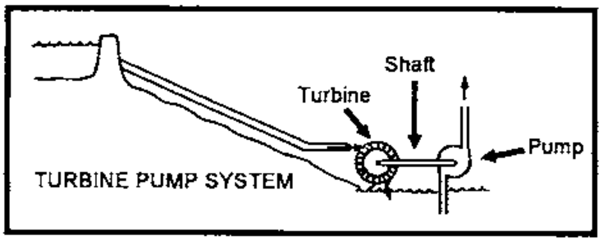
Many people are surprised when they first see a ram pump working. They think that it is driven by a hidden motor, or by magic. They watch it pumping and are amazed.

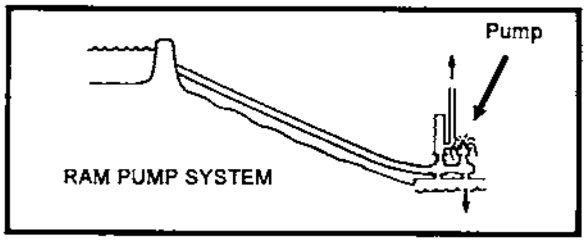
Every kind of pump needs a source of power. They can be powered by people or animals, the wind, falling water, electricity , or fuels such as diesel and petrol. The source of power is often separate from the actual water pump and the two can be joined together by, for example, a shaft. The ram pump uses the power of falling water and there is no separate motor or mechanism that turns the pump. This is why a well designed ram pump can look much simpler than some other pumps. It *is* a very simple machine, although the reasons for it working well are not simple at all.

The drawing below may help you to understand how much simpler a ram pump system is than other ways of using water power to pump water.



THREE DIFFERENT WAYS TO USE WATER POWER TO PUMP WATER

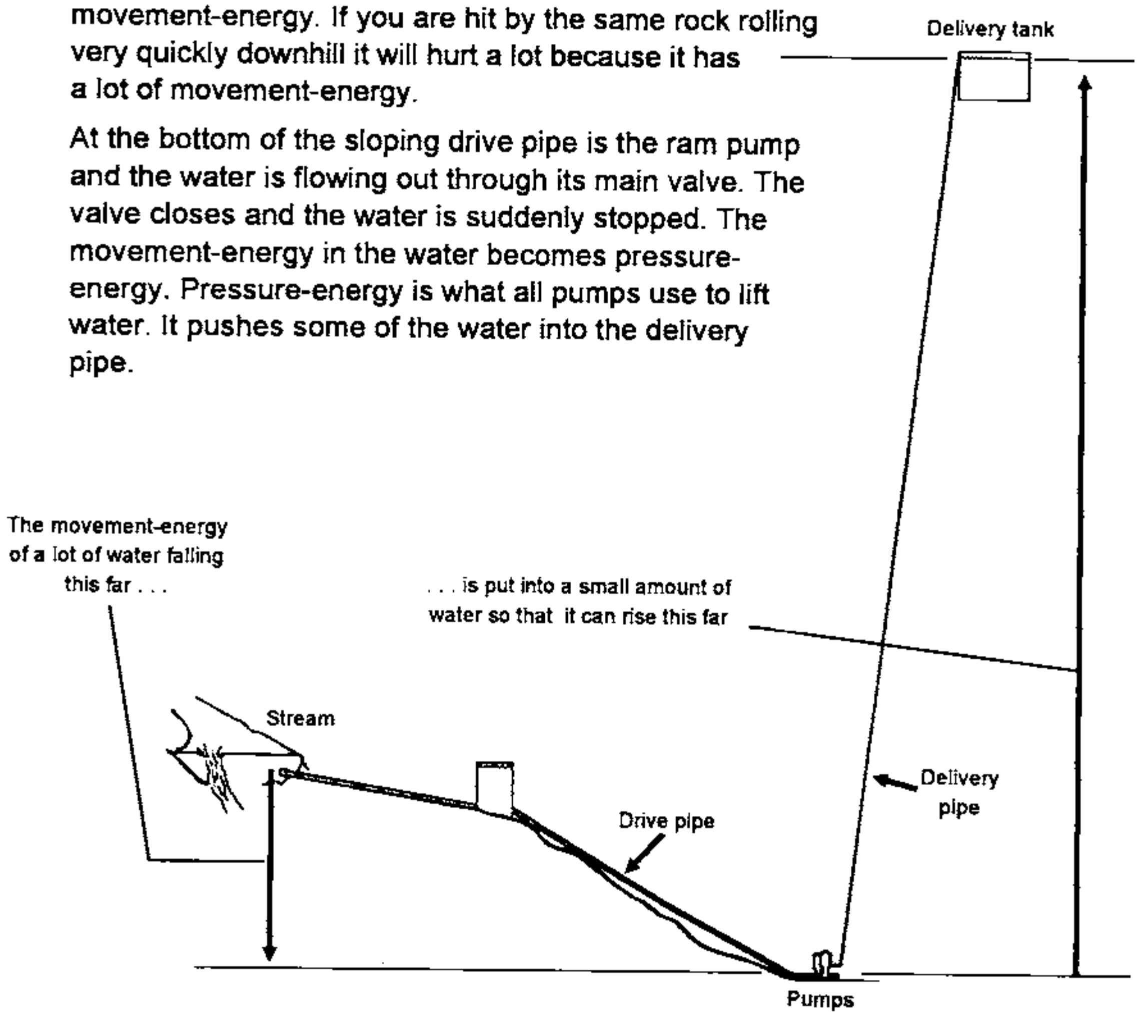




As far as we know, the ram pump was first discovered or invented 200 years ago. Since then it has been rediscovered many times.

Water is diverted from a spring or river and flows down a sloping drive pipe, gathering speed like a rock rolling downhill. As the water gathers speed it gathers energy. The faster it goes the more movement-energy it has. If you are hit by a small rock rolling slowly downhill it will stop without hurting you very much because it does not have much movement-energy. If you are hit by the same rock rolling very quickly downhill it will hurt a lot because it has a lot of movement-energy.

At the bottom of the sloping drive pipe is the ram pump and the water is flowing out through its main valve. The valve closes and the water is suddenly stopped. The movement-energy in the water becomes pressure-energy. Pressure-energy is what all pumps use to lift water. It pushes some of the water into the delivery pipe.



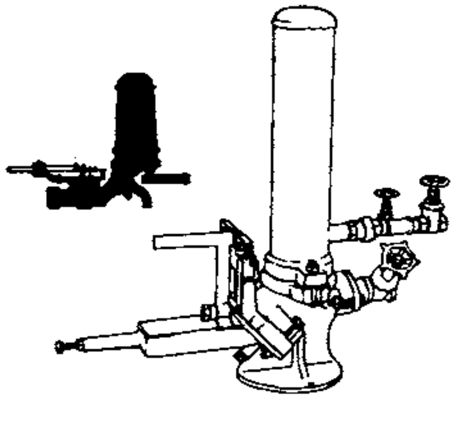
SKETCH OF A RAM PUMP SYSTEM

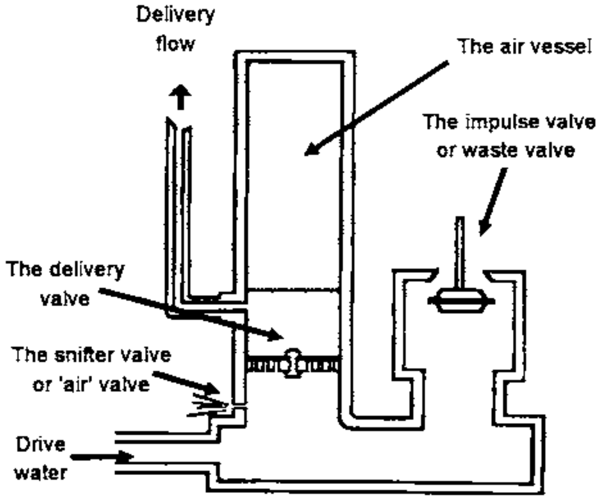
The movement-energy of all the water falling down the drive pipe is put into a small part of that water. The small part of water then has enough pressure energy in it to be 'delivered' to the place where it is needed.

All ram pumps need a lot of water falling down a pipe to provide the energy they use, and they only pump a small amount of the water. This is why they can only be used in places where there is more water than you need to pump. Usually, each pump will pump about 5 to 10% of its drive water. The rest goes back into the stream for other people to use.

The parts of a ram pump

Ram pumps come in many different shapes and sizes but they all have the same basic parts. They all have two main valves, which are an impulse valve and a delivery valve. They also have an air vessel of some kind.



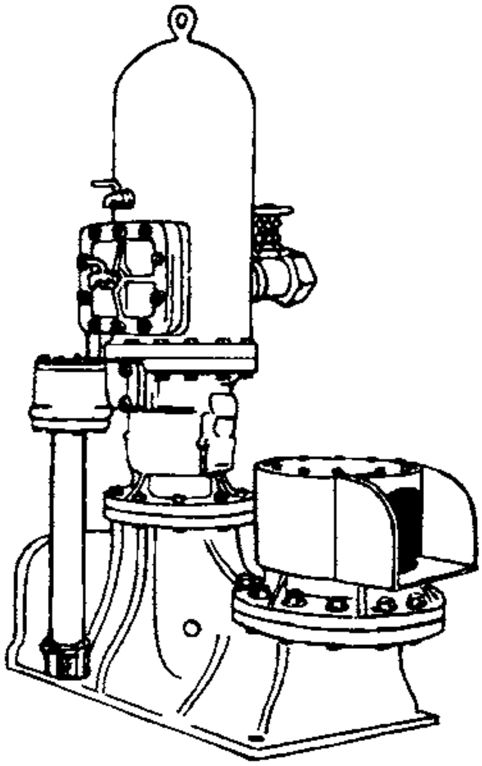


A DRAWING OF A RAM PUMP CUT IN HALF

Most pumps have an air vessel that is slowly filled with air by the snifter valve when the pump is started. The air acts as a shock absorber, absorbing the high pressure surges of the pump. A few modern pumps have an air vessel that is closed by a rubber diaphragm, like the Platypus pump shown on page 12. The pump operator pressurises the air vessel by pumping in air in the same way as you would pump up an inner-tube for a tyre. The diaphragm takes away the need for a snifter valve.

Some ram pumps have the air vessel and the impulse valve the other way around from the pump in the drawing so that the drive water reaches the impulse valve first. Some big ram pumps have more than one impulse valve working together to get the same effect.

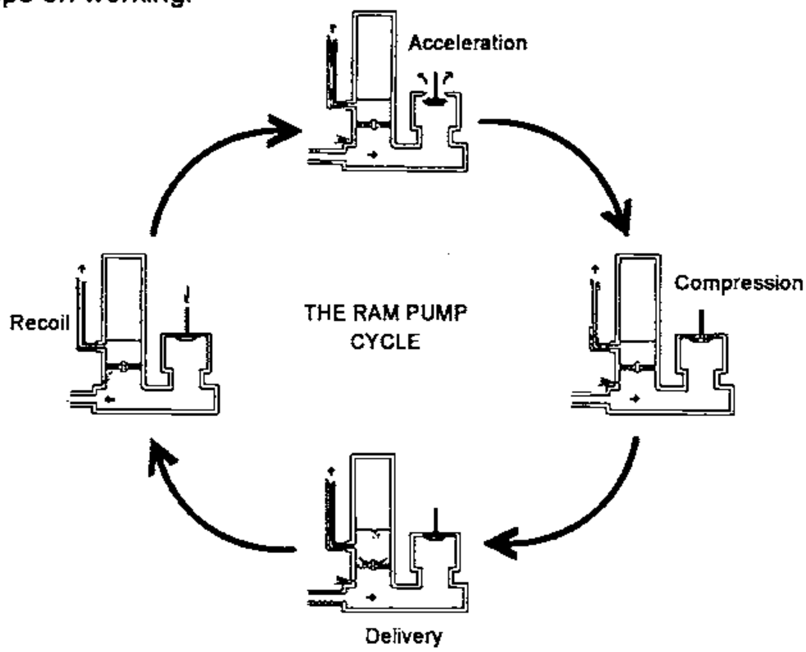
A few pump makers build ram pumps that keep the water that is pumped separate from the drive water. This allows you to pump clean water from a nearby spring while using the power of dirty water from a river or stream. These are sometimes called 'compound' rams. The pumps are more complicated and expensive than ordinary ram pumps, so they are not covered in this Technical Release.



A LARGE GREEN AND CARTER 'COMPOUND' RAM PUMP

The ram pump cycle

Ram pumps have a pumping cycle. The last part of each cycle is the first part of the next, so the pump keeps on working.



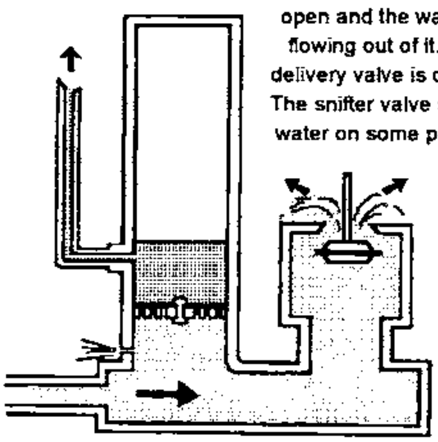
Each cycle happens very quickly, often about once a second. This means that there is no time to see what is happening. The time each stage takes is usually measured in 'milliseconds', which are thousandths of a second. A mosquito flaps its wings about 1000 times a second, so a millisecond is roughly the time it takes for a mosquito to flap its wings once.

It is easiest to explain how the ram pump cycle works by dividing it into four stages called ***Acceleration, Compression, Delivery*** and ***Recoil.***

In the explanation that follows the pump is cycling 60 times a minute, or once a second. The time that each stage takes is given in flaps of a mosquito's wings. Because the cycle takes one second, there are a total of 1000 flaps of the mosquito's wing.

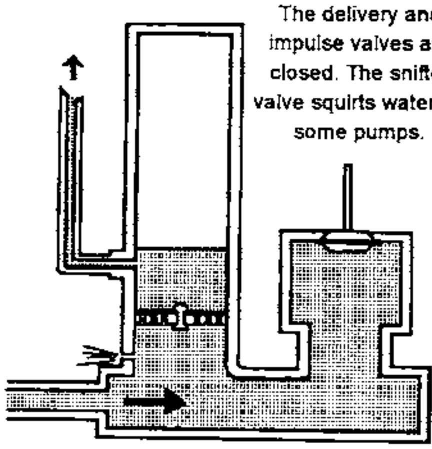
Acceleration (about 900 flaps of a mosquito's wing)
When the impulse valve on the pump is open, water flows down the drive pipe and comes out through the open valve. The water flowing past the open valve drags past it, trying to close it. The flow down the drive pipe and out through the impulse valve gets faster and faster. As it gets faster, it drags harder on the valve until it is strong enough to drag it closed.

The impulse valve is open and the water is flowing out of it. The delivery valve is closed. The snifter valve squirts water on some pumps.



ACCELERATION

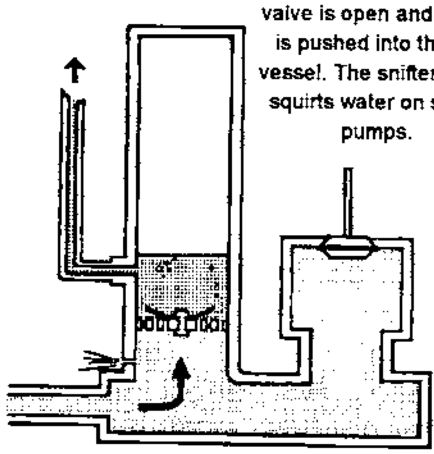
Compression (1 or 2 flaps of a mosquito's wing)
When the impulse valve shuts, the water flowing down the drive pipe cannot come out through it. At the moment the valve closes, the water is travelling very fast and suddenly it has nowhere to go. As it got faster, it gathered movement-energy like the rock rolling down a hill. The movement-energy changes to pressure-energy as it compresses the water in the body of pump. It is as if each small part of the water is bumping into the one ahead as they rush to come down the pipe and escape. As a result there is a sudden rise in pressure that is sometimes called "water-hammer". The pressure rises to a level much higher than the pressure in the pump's air vessel.



The delivery and impulse valves are closed. The snifter valve squirts water on some pumps.

COMPRESSION

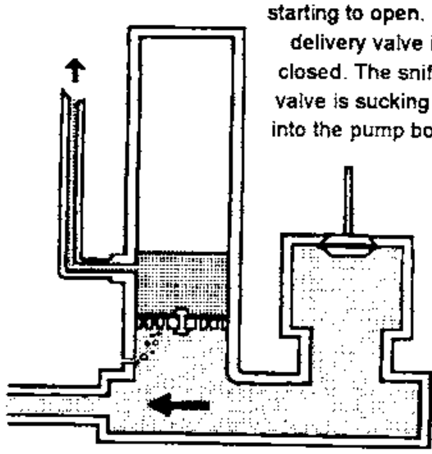
Delivery (about 50 flaps of a mosquito's wing)
Because the pressure rises higher than the pressure in the air vessel, the delivery valve is pushed open and water flows through it. The pressure in the pump body drops quickly to equal the pressure in the air vessel. The water coming down the drive pipe slows down and the pressure in the pump body drops. As soon as the pressure falls enough to be lower than the pressure in the pump's air vessel, the delivery valve closes. The delivery valve is a one-way valve, which stops water flowing back from the air vessel into the pump.



The impulse valve is closed. The delivery valve is open and water is pushed into the air vessel. The snifter valve squirts water on some pumps.

DELIVERY

Recoil (about 50 flaps of a mosquito's wing)
When the delivery valve closes, there is still some pressure in the pump body and drive pipe. The valves in the pump are closed, so the only direction in which the water can move is back the way it came. The water coming down the drive pipe has stopped, so the pressure-energy can be released by moving back up the drive pipe. The water in the pump body bounces back a little way up the drive pipe. This bouncing back makes the pressure in the pump body fall low enough for the impulse valve to reopen. On some pumps the impulse valve includes a spring to help it reopen, on some they reopen because of their own weight. The low pressure in the pump body means that a small amount of air is sucked through the snifter valve. The air waits under the delivery valve until the next cycle when it will get pushed into the pump's air vessel. This makes sure that the air vessel always stays full of air.



The impulse valve is starting to open. The delivery valve is closed. The snifter valve is sucking air into the pump body.

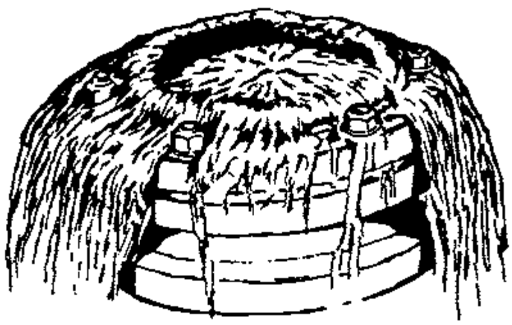
RECOIL

Very quickly, the pressure left in the pump body is released by bouncing back up the drive pipe. When the bouncing back is complete, water begins to flow down the drive pipe again. This is where the cycle started, and the water **Accelerates** down the drive pipe through the open impulse valve.

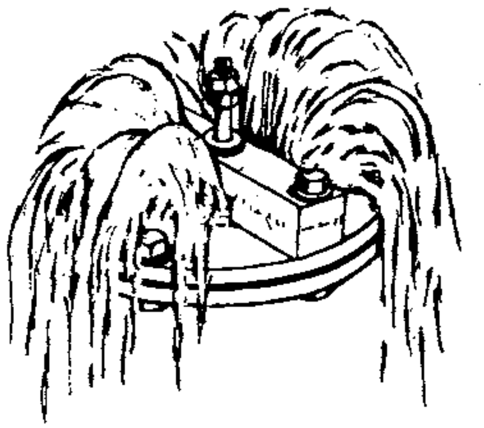
During each pumping cycle only a small amount of water is pumped. Most of the movement-energy harvested from a large amount of water is transferred into a small amount of water. The high pressure in the pump body pushes water through the delivery valve and into the air vessel. It provides the power to push the small amount of water much farther uphill than the big amount of water fell downhill.

Throughout each pumping cycle the pressure in the pump's air vessel is steadily forcing water up the delivery pipe. The air vessel smoothes the pulses of water coming through the delivery valve into a steady flow up the pipe to the delivery tank.

While a ram pump is working, water flows out of the impulse valve and splashes onto the floor of the pump house. This happens during the 'acceleration' phase of each cycle. It is the splashing water and the noise of the "water hammer" that people notice when they see a working ram pump. The water splashing out is often called 'waste' water. Although 'waste' water is not delivered by the ram pump, it is the movement-energy harvested from this water that pumps the water that is delivered. A better name for 'waste' water would be 'used' water. The noise varies from pump to pump. Pumps with impulse valves that have no moving metal parts are the quietest, but they can still disturb people who live nearby. This is because of the water hammer "drumming" in the drive pipe.



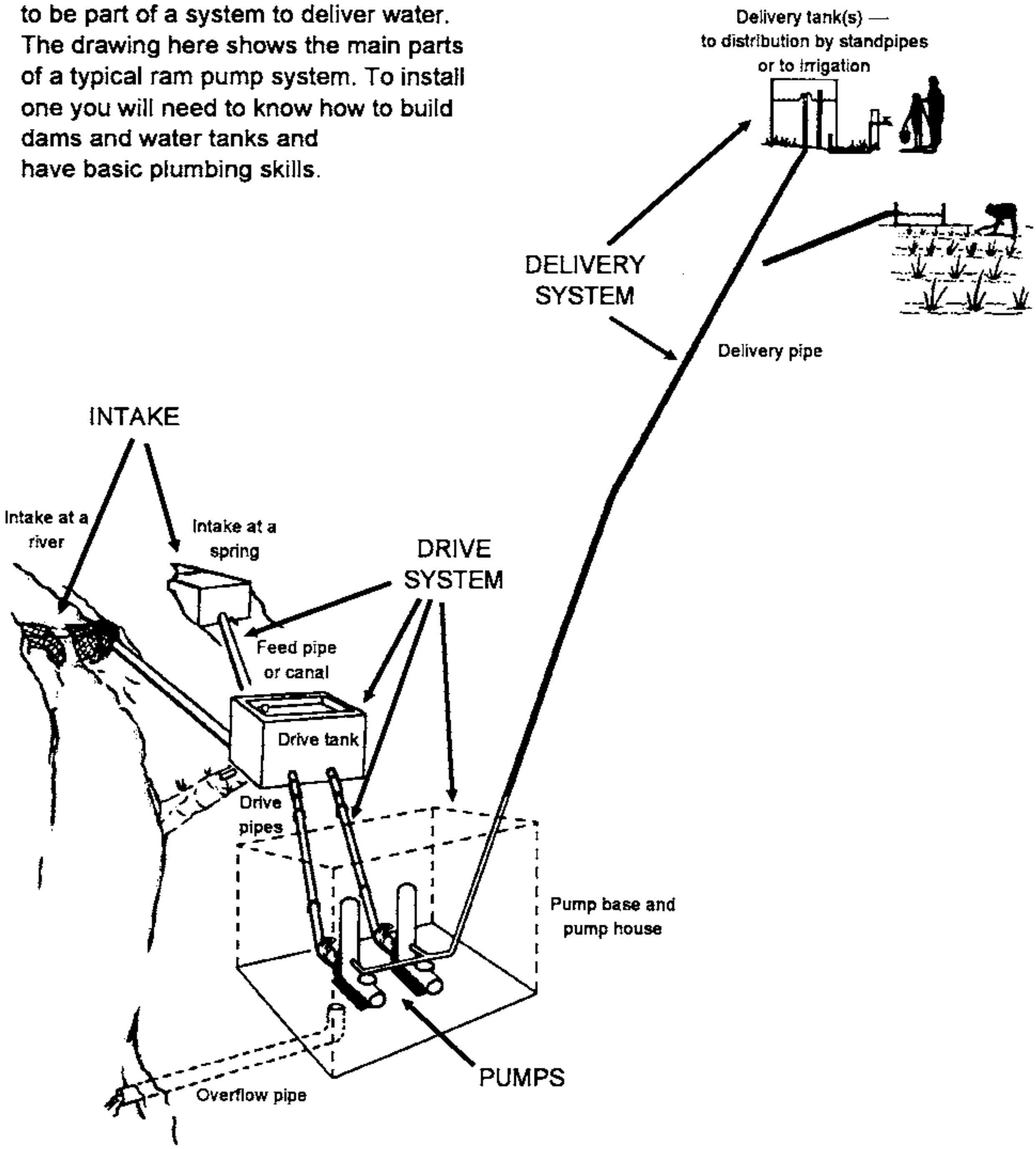
THE IMPULSE VALVE OF A BLAKES RAM PUMP
This has no moving metal parts. It pulses gently and is fairly quiet



THE IMPULSE VALVE OF A DTU M8 RAM PUMP
This splashes and is noisy.

The parts of a ram pump system

A ram pump does not work alone. It has to be part of a system to deliver water. The drawing here shows the main parts of a typical ram pump system. To install one you will need to know how to build dams and water tanks and have basic plumbing skills.



THE MAIN PARTS OF A RAM PUMP SYSTEM

As you can see, ram pump systems can have more parts than some other water pumping systems. This means that they may be more expensive to install. The main advantages of using ram pumps are that they can often be maintained by the users and cost little to maintain and nothing to run.

When to use ram pumps

Ram pumps can only be used where water flows downhill and where there is much more water flowing than you want to pump. They are usually used to lift water from springs or streams in hilly areas. Sometimes they are used for irrigation, but more often they are used for community water supplies.

There is a limit to how high a ram pump can lift. This varies from pump design to pump design. Some pump makers make unrealistic claims for their pumps. Generally, ram pumps with up to 4" drive pipes can deliver a useful amount of water to heights of up to 100 meters. Some pumps cannot pump this high, so check a pump's instructions before choosing which pump to use.

In most cases it is important that the pumps work all year round. They cannot do this if the springs or streams from which they get their water run dry or get very low in the dry season.

A ram pump system is very cheap to run but it does need care and maintenance. The people using the system must be committed to its maintenance and understand how to do it.

The water supplied by any water system to a community must be distributed in a way that meets the people's needs and is seen to be fair. The people must be involved in planning the system, especially the way the water is distributed. They must also be prepared and able to pay for occasional replacement parts.

So, ram pump systems can be useful when there are the following things.

- A flow of water that is dropping quite quickly. It does not have to be flowing steeply in the stream or river. The system designer can usually make the water do that.
- A source of water that has a much bigger flow than you want to pump.
- A source of water that does not get very low or dry up at some times of year. This applies especially to irrigation systems because the time they are needed is usually the time when the water is lowest.
- A place for the pumps that is not more than 100 meters below the place where the water must be delivered.
- A willingness for system care and maintenance to be provided by the community that uses the water.
- A community involved in planning and paying for the system.

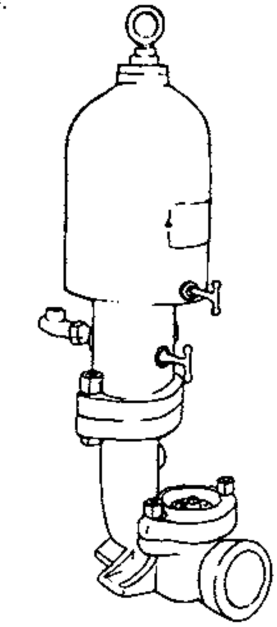
If a site meets these conditions, it is probably worth carrying out a site design survey. A design survey will give a good idea of the amount of water that can be delivered and how much a system will cost.

When you are installing a community supply, remember that many sources of water are not safe to drink. Springs are usually safe but water from streams and rivers is usually not safe to drink. The water must be filtered or boiled before you use it. Slow sand-filters that clean the water can add quite a lot to the cost of a system.

Choosing which ram pump to use

Ram pumps have been made for general sale for well over a hundred years. Many of the early designs were much stronger than they needed to be. Some were also more complicated than they needed to be. A few of the pumps made early in this century are still being used around the world. Many pumps made fifty years ago are still working even though the makers may have stopped selling them a long time ago.

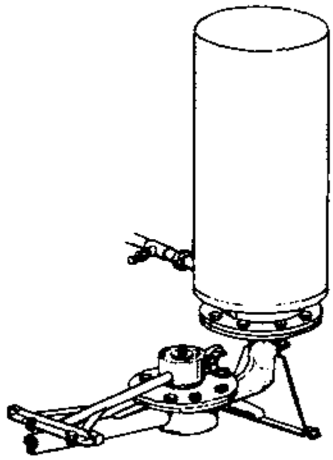
Some of the traditional ram pump makers are still in business at the time this is written. The most famous is perhaps John Blake in the UK. The basic design of their pumps has been so successful that it has been copied by a number of makers from China to Africa.



A SMALL 'JOHN BLAKE' RAM PUMP, UK

Buying ram pumps from traditional makers can be very expensive if they have to be imported. Although some run for years without needing spare parts, others need them frequently. It can be expensive and difficult to get the spare parts and advice you will need. The only way to avoid the spare parts problem is to buy pumps that are made locally or arrange to have your own pumps made. You are probably reading the Technical Release to help solve the problem of lack of advice.

Recently, a number of small engineering firms have started to make ram pumps to meet local demand. Be warned that some of these businesses do not understand ram pumps very well and their pumps can be poorly designed and made. Try to see an example of the pump installed and working before buying one.



A 'DCS' RAM PUMP
Nepal

If you want to make your own ram pumps, several designs are freely available from European organisations. The DTU believes that its own range of designs are the easiest to make in small workshops and give the best performance.

Ram pumps vary a great deal in size, but a single ram pump does not normally pump a lot of water. If a lot of water is needed, two or three pumps are usually installed and they are all left working constantly. For example, if a single pump delivered 5 liters of water a minute to a tank, it would deliver 7,200 liters of water each day. If three pumps were side by side in the system, they could deliver together 21,600 liters of water each day.

Some pump makers produce a range of pump sizes so that you can choose from it a single pump that will deliver enough water for your needs. If you know that the pumps are very reliable and easy to tune, it can be sensible to just use a single pump. If you are buying pumps that are locally made and do not have a well known reputation, it is better to buy two smaller pumps than one big one. Then, when a pump needs to be maintained or repaired, the other pump will keep working and some water will be delivered.

Buying a pump

Which pumps you buy will depend on which pumps you can get.
You will need to find out these things from the maker's instructions:

- The size of the pump's drive pipe. This is the pipe's internal diameter (ID).

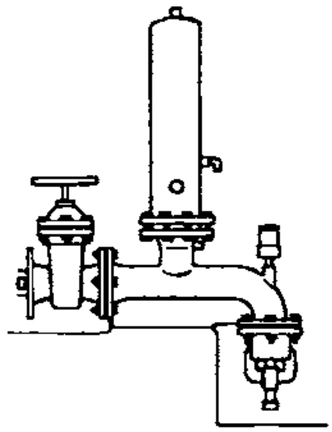
  ☛ *If you choose a pump with a drive pipe bigger than 4" you should get the manufacturers to help design the system.*

- The pump's maximum and minimum drive flow. This is the most drive water it can use, and the least drive water it can use.
- The pump's maximum "feed" or "drive" head. This is the highest feed head that it can use.
- The pump's maximum delivery head. This is the highest that it can pump to.

The instructions also tell you the pump efficiency. It is always useful to know a pump's efficiency but unfortunately pump maker's claims about efficiency are not often reliable.

Imported pumps

If you can afford to buy imported and well known pumps, that may be the best choice. They often cost five to ten times as much as a locally made copy, and between ten and twenty times as much as a pump you make yourself. There are so many ram pumps available for import that it is not possible to assess them here, but most well known names give good and reliable service. Remember that all pumps will need spare parts eventually, and some imported spares are very expensive. If you are installing a system for someone else, remember to make sure that *they* will be able to get the spares when they need them. There are many ram pumps around the world that no longer work because the people who installed them have left the area. After some time the pumps needed parts and the users had no idea how to get them.

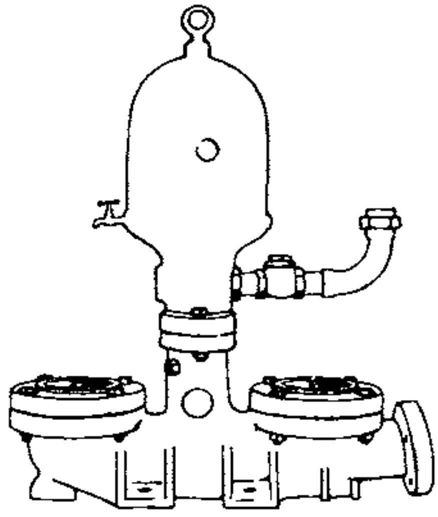


A PFISTER AND LLANGHAUS RAM PUMP, Switzerland

Locally made pumps

There are advantages if you buy from a maker that can be contacted easily when spare parts are needed. Try to make sure that the maker has been in business for some time so that they are still likely to be in business when the spares are needed. Locally made copies of pumps may need a spare part much more frequently, but if the spares are cheap and easy to get, that is not usually a problem. Check any pump that is locally designed very carefully.

The drawing on the right is of a pump made by Jandu plumbers in Tanzania. It is based on a heavy cast-iron John Blake design. Notice that it has two impulse valves so that it can use more drive water and harvest more energy to pump the delivered water.



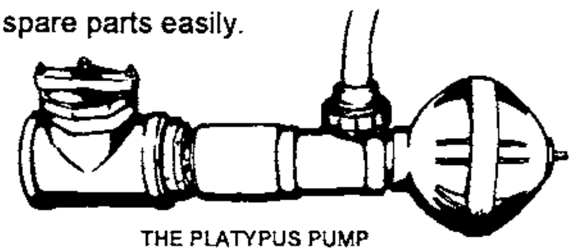
A JANDU PLUMBERS
TWIN-IMPULSE RAM PUMP
Tanzania

What to look for in a pump

With any make of pump, the first thing to check is that the makers give the pump's operating ranges. Do not buy pumps that do not include some instructions that tell you the operating ranges. Even if the pump is good, you will not know how to design the system properly and it may let you down.

- Look at the whole pump. If it looks very complicated with lots of small parts, it is probably more likely to break down.
- Check that the pump is well made. Inspect any welds carefully to see if they are fairly regular and not lumpy.
- Look at the hole or holes around the impulse valve where the water comes out. The combined area of those holes should be at least as big as the hole inside the drive pipe. If it is not, the drive pipe is bigger than it need be and the pump is badly designed. Some pumps with a body that is screwed together from pipe fittings have this problem. Surprisingly, some of the best known pumps also have it. If it is the only thing wrong with a pump it can be ignored.
- Look for any parts that look as if they may bend or break easily and avoid these designs if you can. Pay special attention to the impulse valve arrangement, which has to open and close millions of times a year.
- Check that the impulse valve moves up and down freely and appears to seal. If it has a spring or lever arrangement, make sure it is protected against rust.
- Check that the air vessel is made from thick steel. Usually 3mm is thick enough. If possible, check that the air vessel has been painted or galvanised inside and out.
- Avoid pumps that have already started to rust. They may be old stock, or carelessly made. If the nuts and bolts are rusty it may make the pump hard to maintain. When a pump is being used, the water will wear paint away from some parts very quickly. Those parts will not rust much because the moving water will keep them clean.
- Check that the pump includes a way to attach it firmly to a base or cradle. Pumps made from cast iron often have feet with holes for bolts. Pumps made using pipe fittings may use "U" bolts.
- Look at any rubber parts carefully. If the rubber is starting to perish the pump may be old stock and the rubbers will need to be replaced soon.
- Check that the pump has a snifter valve. It may be a small hole or a valve and is usually in the body of the pump just below the delivery valve. Very rarely the air vessel is made with a diaphragm inside like the Platypus pump shown below. These can be pumped up like a car tyre and do not need a snifter valve. All other pumps must have one. If they do not they can be dangerous.
- Make sure that you can buy spare parts easily.

The Platypus pump has an inflatable air vessel and is designed to run completely under water.



THE PLATYPUS PUMP
Australia

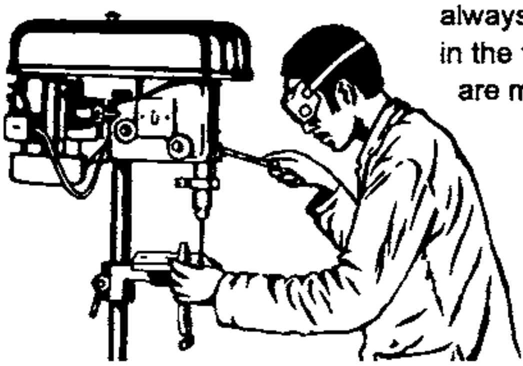
Using an old ram pump

If you have found an old pump that you want to use, contact the makers before deciding to do so. If the makers do not make the pumps any more, you may not be able to get spare parts. Also, you may not be able to find out the operating ranges of the pump.

Engineers may be confident that they can make any part that needs to be replaced. Be warned that the rubber parts on European ram pumps are nearly always made from a very special compound and the pump will not work reliably with anything less.

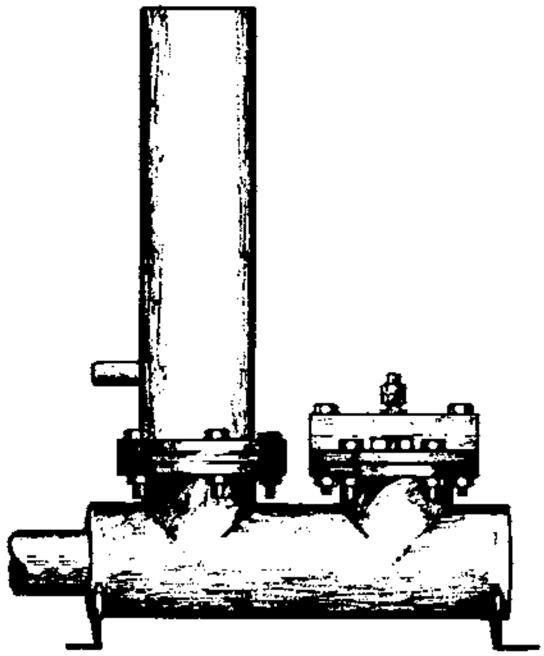
Making a ram pump in a small workshop

There are two big advantages in making the pumps or having them made in a local workshop. The first is that spare parts are easily made and so always available. The second is that the pumps cost so little in the first place. In the case of the DTU designs, the pumps are made from materials that are cheap and easy to get, so the spare parts are also cheap.



There are also disadvantages in making the pumps or having them made in a local workshop. The workshop must have a skilled metalworker to make the pumps or they will almost certainly be unreliable. Also, using cheap and available materials means that some small parts wear out and need to be replaced quite often.

The DTU S2 pump shown in this drawing is built from steel pipe and plate that is widely available around the world. This pump has a high efficiency. It can be made in a small workshop that has welding facilities, a gas cutting torch and a pillar drill.



THE DTU S2 RAM PUMP

The DTU range of ram pumps

Since 1988 the Development Technology Unit has been developing designs for ram pumps. These have been intended to enable small industries in developing countries to manufacture reliable pumps using commonly available materials. The pumps are much cheaper than imported ones but their lives are shorter. Indeed, it is assumed that they will need and get some maintenance over their working lives. Parts most likely to wear out have been made easy to copy and replace.

Three pumps are described briefly here. All are latest versions of pumps used in Africa and Asia for some years. Two are of steel (for respectively 1" and 2" galvanised iron drive pipes) and one is of plastic the P90 (for use with 90mm plastic pressure pipe) The P90 could be made entirely with hand tools. The S1 and S2 require simple power tools (welder, pillar drill) and the skills to go with them. Whilst a lathe or gas cutter are also required to make the steel pumps, the operations involving these machines are so few and simple that they could be sub-contracted to an urban machine shop at little expense.

TABLE OF PUMP CHARACTERISTICS

	S1	S2	P90
Material (mainly piping)	Steel	Steel	PVC or ABS
Drive flow range (in liters per minute)	20 — 60	40 — 120	100 — 360
Drive pipe size (high drive flows)	1"	2"	90mm
Drive pipe size (low drive flows)	¾"	1½	90mm
Maximum delivery head in meters	80	100	20
Maximum drive head in meters	15	15	3
Typical delivery flow in liters per minute	½ — 10	1 — 20	3 — 40
Typical life (assuming minor repairs)	5 years	5 years	2 years

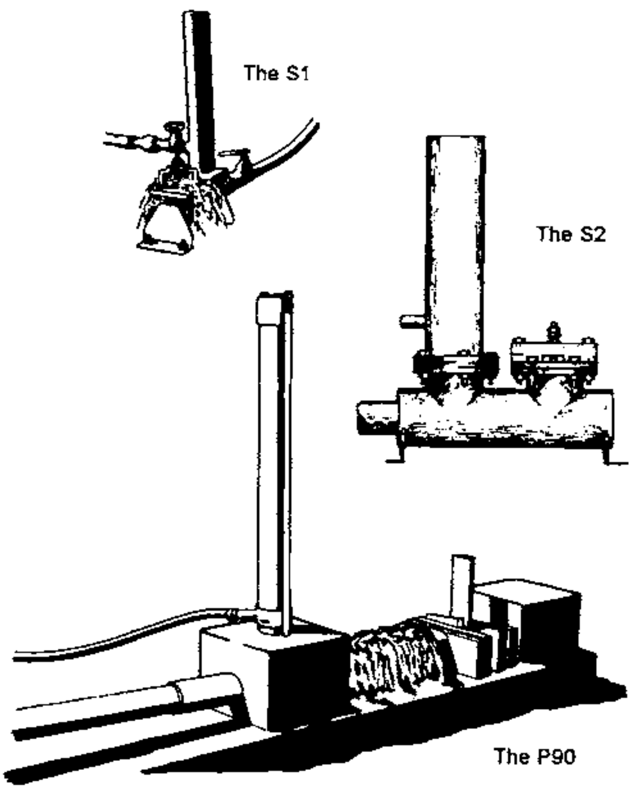
Note that if two pumps are used in parallel, the drive flow and delivery flow of the system will be doubled

S1 pump. This small pump is normally used to supply drinking water to a house or small group of houses. The drive flow is usually taken from a spring.

S2 pump (formerly M8). This medium size pump can be used for water supply to a village or institution, or to irrigate gardens or to water cattle. Its drive flow is usually taken from a large spring or a small stream.

P90 pump. This large pump is mainly used for irrigation, drawing its drive water from a (diverted) stream or drop-structure in a canal.

For full details of each pump see other DTU Technical Releases:
TR 11: The DTU S1 ram pump
TR 12: The DTU P90 ram pump
TR 14: The DTU S2 ram pump.



KENDAT

DTU

Animal Cart Programme

TECHNICAL
20
RELEASE

STEEL FOUR FRAME CART FOR ONE DONKEY

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

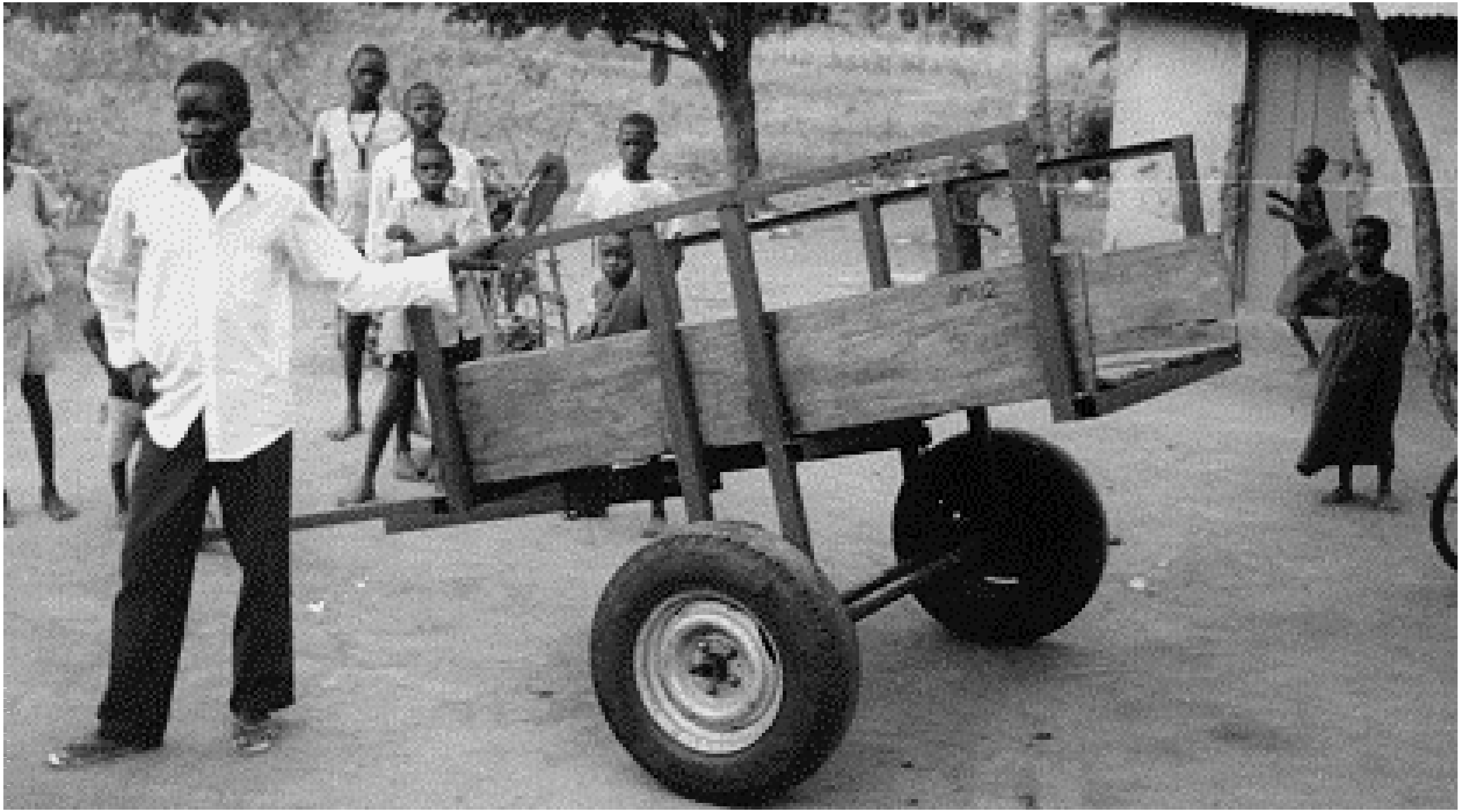


Figure 1: four frame cart with extended frames and twin live axles.

Steel four frame cart for one donkey

This is a light cart for one donkey or for use as a handcart. It is made from square steel tube welded together and timber planks fixed with clenched studs. The planks are part of the structure of this cart and so must be fitted.

Suitable axles

We recommend that you use the PVC plain bearing fixed axle described in Technical Release 28 with this cart, or one of the twin offset axle systems described in Technical Release 36, 37 or 41. An alternative is the needle roller bearing axle described in Technical Release 21, but this is more difficult to make.

You should find that you can make the body for about £$_{UK}$50, depending on the cost of the materials and labour. Once you get organised, two men can probably make one body in a day. We've designed these carts to be easy to make.

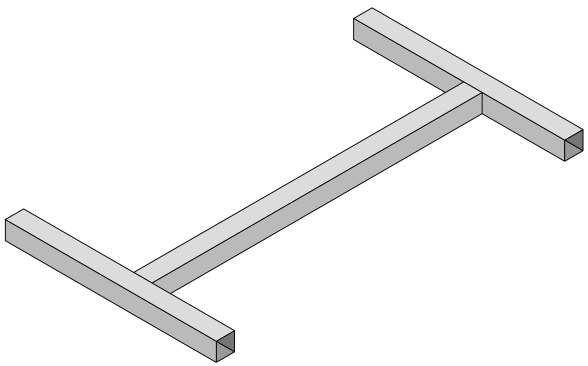


Figure 2: an H shaped centre frame.

To make this cart you must have a simple welder, a woodsaw, a hacksaw, and a hammer.

Construction step by step

Table 1 shows a cutting list for a complete cart body. Recent (1998) prices of materials in Kenya are shown converted into £$_{UK}$.

1) Start by getting all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete.
2) Cut the 50 × 50 box section steel into the right lengths, as in the cutting list, then cut the bottom and side planks. Lastly cut the 6 mm or 8 mm diameter re-bar for the fixings ie studs.

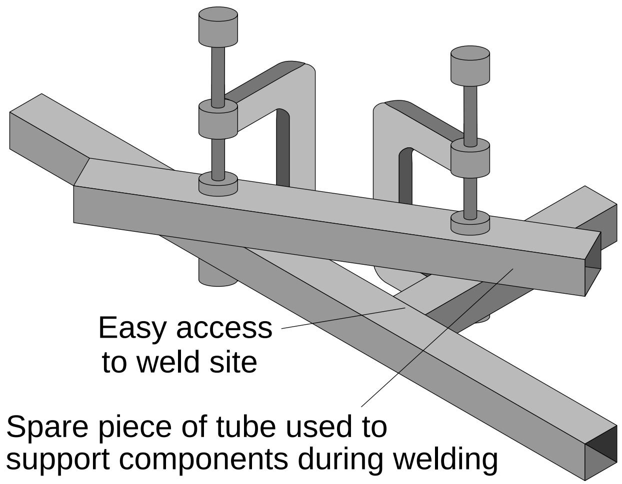


Figure 3: supporting components during welding.

3) Next make up the two H shaped central frames which will support the axle. Figure 2 shows one of these. Figure 3 shows how two pieces of square tube may be held in the right place when welding. Once you have made one frame, make the other the same by welding it on top of the first.

4) Then make up the U shaped end frames as shown in Figure 4. Again, make them as accurately as you can by building them on top of the H frames.

5) Now stand the H frames on the two axle support beams, tack or spot weld them and check that all the joints are square. Figure 5 shows what the finished centre frame assembly should look like.

Table 1: materials and costs.

component	material	# lengths & length reqd [#*mm]	total material in cart [mm]	materials cost in Kenya [£UK]
animal shafts	50×50 RHS	2×2200	4400	8.80
body frame bottoms	50×50 RHS	4×1000	3000	6.00
body frame sides	50×50 RHS	4×325	1300	2.60
axle struts	50×50 RHS	4×625	2500	5.00
axle beams	50×50 RHS	2×400	800	1.60
shaft strengtheners	8mm to 12mm round bar1	8×600	4800	1.52
draw pole loop	12mm round bar	2×500	1000	0.32
axle strut braces	8mm to 12mm round bar	2×600	1200	0.38
axle fixing studs	M12 threaded rod or bolts	2×100	200	2.00
axle fixing loops	6mm dia re-bar or similar	2×200	400	0.04
plank fixing staples	6mm dia re-bar or similar	30×250	7500	1.25
tray bottom planks	1"×6" or similar timber	6×1800	10800	3.54
tray side planks	1"×6" or similar timber	4×1800	7200	2.36
tray ends	1"×6" or similar timber	4×900	3600	1.18
TOTAL				36.59

6) Now you can fit the side and the bottom planks to the end frames and then the middle frame. Figure 6 shows how to position these studs and how they can be tightened with a hammer and a weight or another hammer.

7) Fix the axle to the axle support beams as described in the Technical Release on the axle you have chosen.

8) Nearly there! Now you need to fix the draw poles or 'shafts'. It is best to fix them to the body so they can be taken off and replaced if they get damaged. Figure 7 shows how they can be fixed using short lengths of round bar.

9) Figure 8 shows how you can make the ends of the load tray easily removable.

10) Paint or creosote the cart. You've finished it!

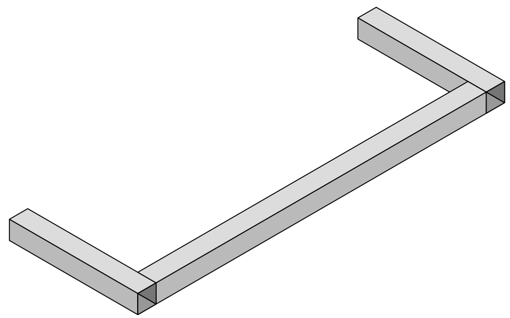


Figure 4: a U shaped end frame.

Modifications

You can try longer or shorter carts and you can make them wider or narrower. When you do this, check the length and width of the planks of wood that you will use to avoid waste. Figure 1 shows a cart made in Uganda with axle support struts extended upwards to carry light foliage.

Cart Drawings

You will find two drawings on the next pages, the first one gives a general view of the cart, and the second gives a view of the main components. As we have said you can vary the size of the cart quite a bit.

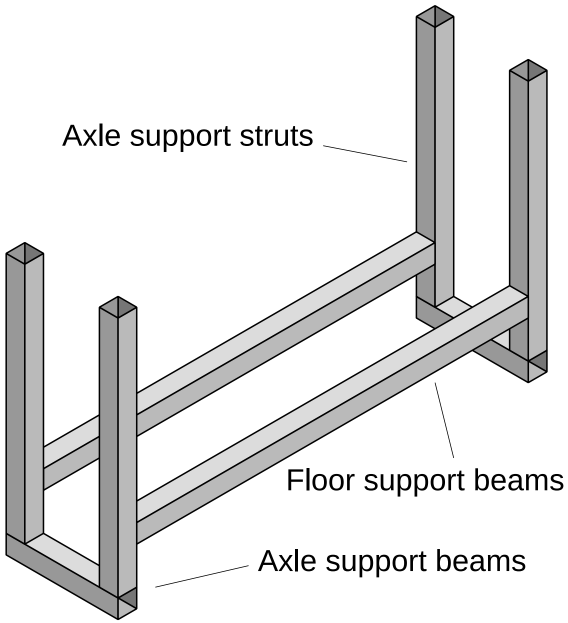


Figure 5: finished centre frame assembly.

Other DTU cart developments

The DTU has been working on a range of cart body types for use with both donkeys and oxen. It has designs for both wooden and steel framed types. The wooden types are cheaper in material terms, but the steel framed ones are easier to make because the joints are more straightforward - but you can make either type of cart in only a day or two.

The DTU has also been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. We have developed easily made wooden bearings, bearings from PVC pipe, axles using old ball races and axles where you make your own roller bearings.

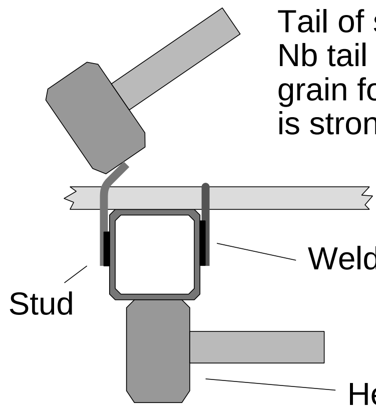


Figure 6: fixing planks to frame with clenched studs.

Technical releases for all these are available.

Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.

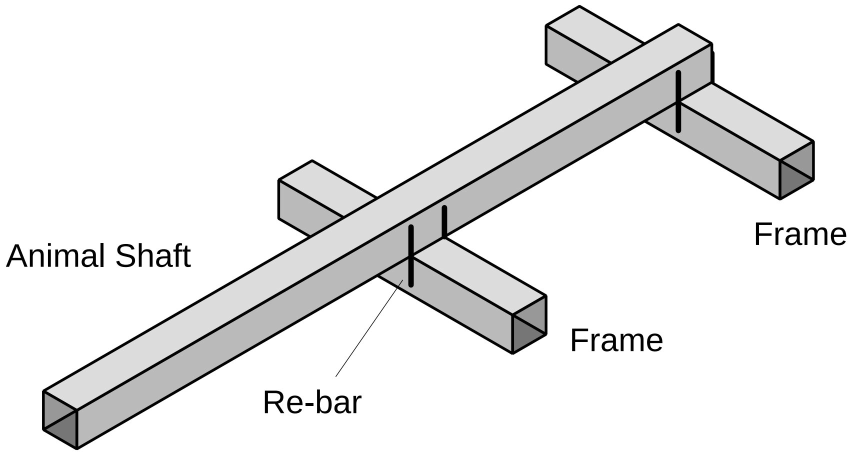


Figure 7: fixing to frame with studs.

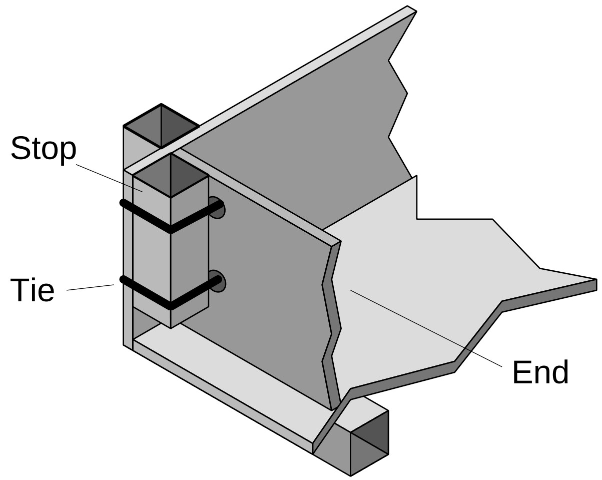
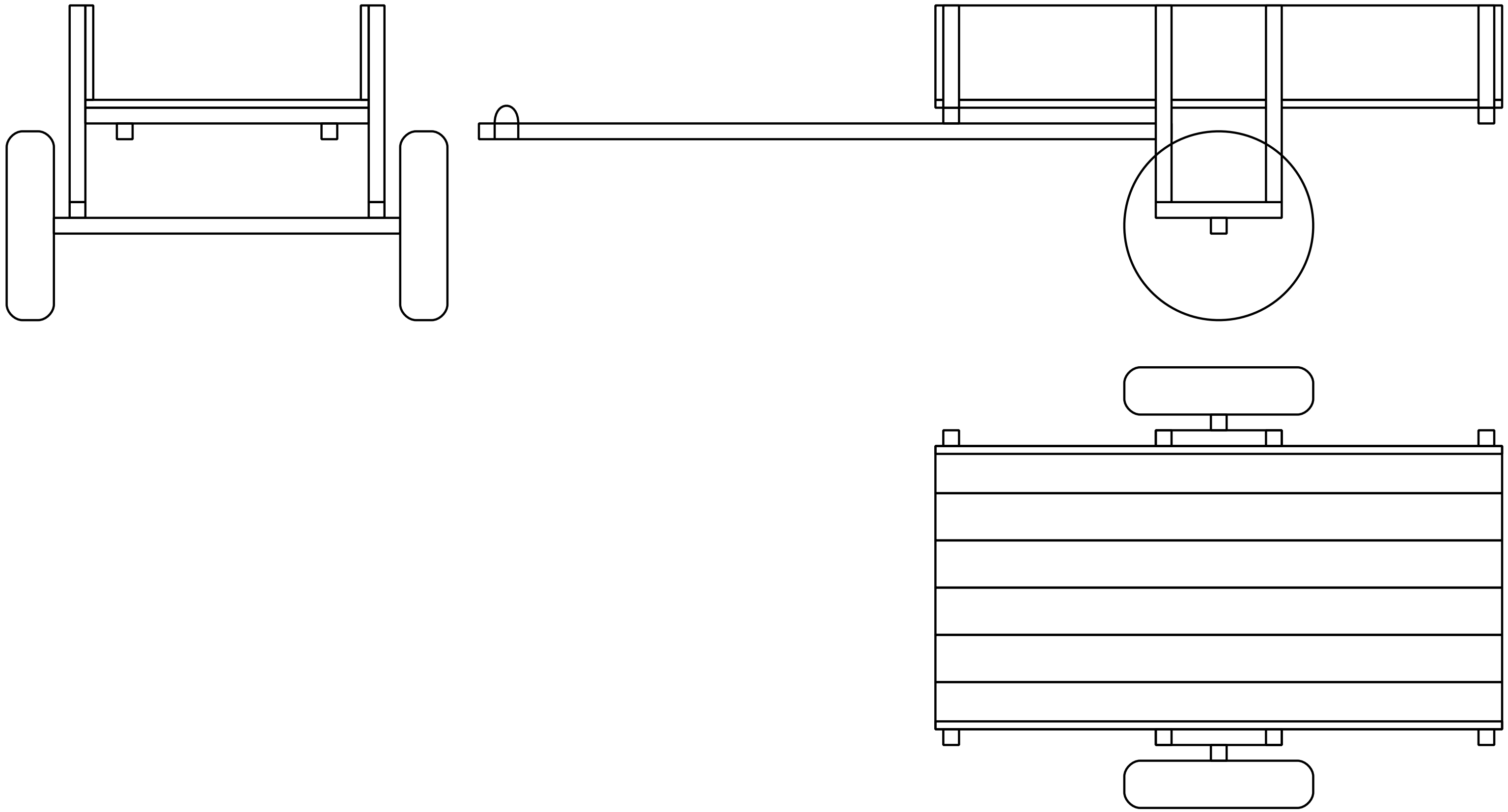
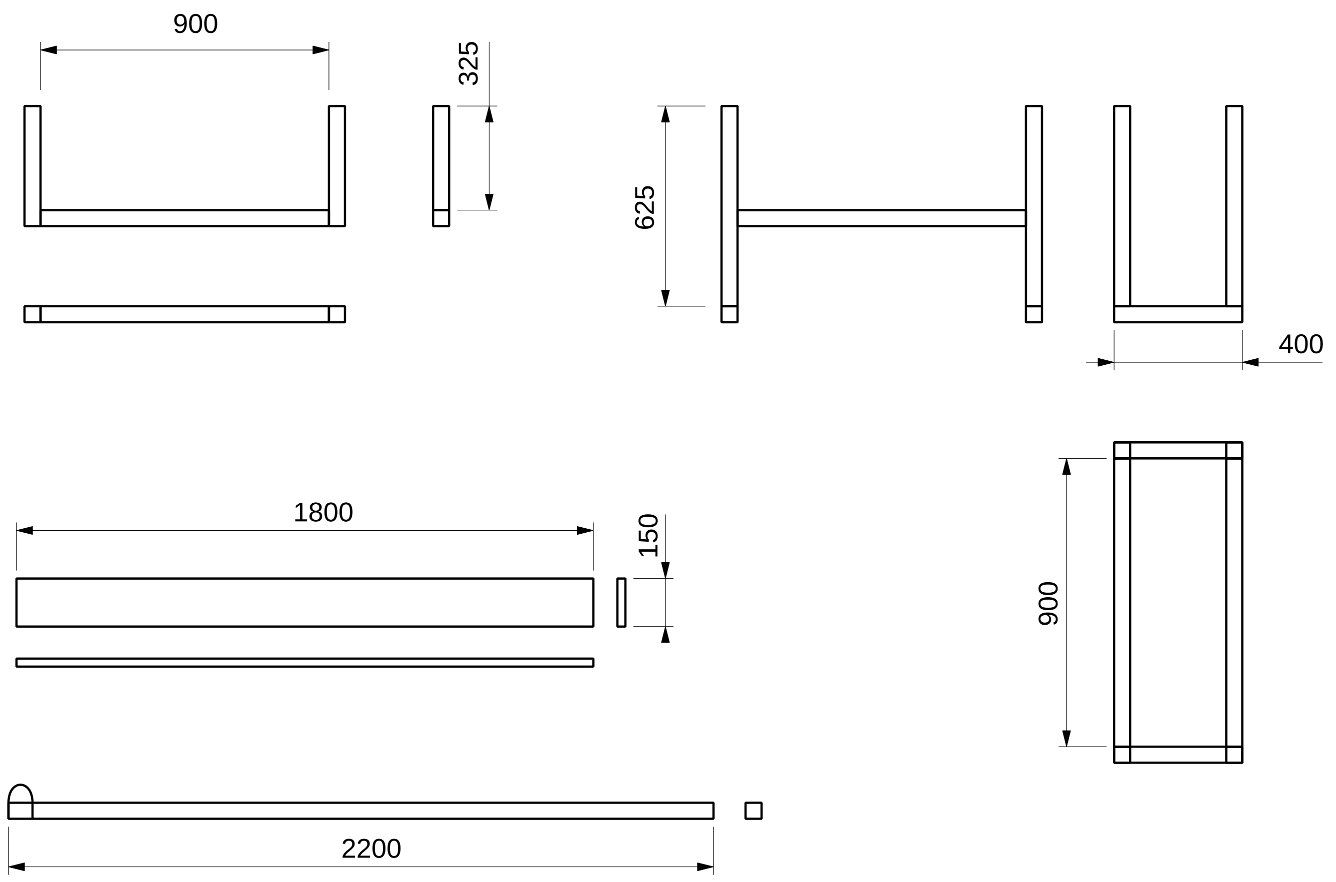


Figure 8: method of fixing tray ends with rubber or rope



					Scale	100 mm	Title 4 FRAME STEEL & WOOD CART FOR SINGLE DONKEY	Drawn by	CEO
					Date	2/7/99		Dwg No.	1/2



					Scale	100 mm	Title COMPONENTS FOR 4 FRAME STEEL & WOOD DONKEY CART		Drawn by	CEO
					Date	2/7/99			Dwg No.	2/2

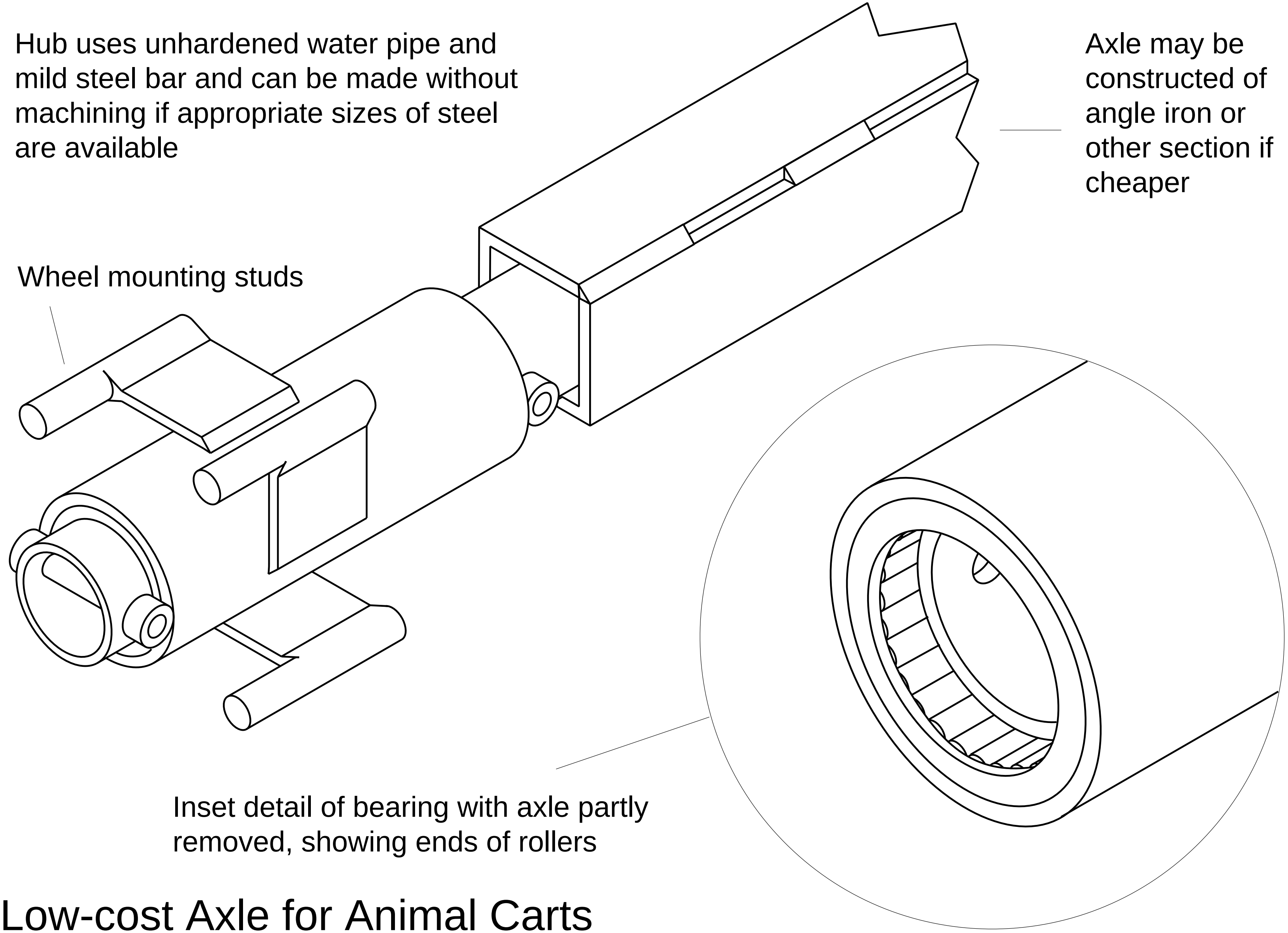
DTU

Animal Cart Programme

TECHNICAL
21
RELEASE

PIPE AND ROLLER DONKEY CART AXLES

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)1203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@uk.ac.warwick.eng



Low-cost Axle for Animal Carts

TR21: 1st August 1994

Pipe and Roller Donkey-Cart Axles

Introduction

Not enough farmers in Africa have animal carts. Those who have carts can take their produce to places where they can get the best prices. They can also get into town and buy fertilizer and better seeds and move stuff around their farm easier. The trouble is that carts are too expensive for many farmers. The question is what can be done about it?

Carts are made in many different places. Some carts are made in factories in industrial countries and some are made in factories in Africa, but most are made by local blacksmiths or carpenters using scrap car and Land-Rover axles. These people cannot get enough axles to meet the demand so the price is high. Another problem is that the axles are often so worn that they do not last long. Lots of farmers take the diff (differential) unit out of the axle too, which makes it break sooner and lets the dirt in.

What you need is an axle which blacksmiths and fabricators can make with fairly simple tools - without having to get parts machined. There are usually blacksmiths and fabricators in the small market towns used by the farmers. Experts think that having the cart maker close to the farmer is a good thing because they can talk to each other easily and sort out any problems. And of course if the cart is made locally, it can be repaired locally, so there will not be problems with spare parts.

Idea behind design

The idea behind the design of axle described in this technical release is to allow construction without the use of machine tools (lathes and milling machines), and using materials which should be readily available. The materials can be used 'as bought' - no hardening of any of the components is needed. The only tools which you must have are a hacksaw, a file and a drill able to drill a 13mm hole in steel. Having a vice is also very handy.

Of course if you do have power tools - especially a power hacksaw or cutoff wheel - things can be made much faster. This axle is suitable for a wide range of production methods.

The long thin needle rollers in this design have been used so that the hubs will usually fit scrap car wheels. Most wheels have a hole in the middle for the axle. This hole is usually about 60mm diameter, or a bit bigger. Sometimes they are smaller and the wheel will not fit. You can sometimes saw or file the hole bigger. Putting the fixing studs on struts like you see in the drawings means that you might be able to bend the struts a bit to fit a different wheel. Or if that does not work you can even cut nearly through the welds and then weld them in the right place. You could even cut the struts right off and weld on a different number if your wheels have a different number of holes.

Performance

We have tested axles like these for more than 10 000km in the laboratory and some axles have been ok for 30 000km. Usually we set the load at 200kgf per wheel, but we have used them at 400kgf for thousands of km. Sometimes you need to clean them out and regrease them. Some of the axles we have tested have not been very round at all - one was more than 1mm out, but they have still worked. We have tried rollers made out of 6" nails with the heads cut off. They still worked even though they were slightly bent. (The symbol " means inches so 6" means about 150mm since there are 25mm in one inch.)

component	material	number & lenghth required [no.×mm]	total material in axle [mm]	materials cost in Nigeria [$us]
central axle	2" angle iron[1]	2×1200	2400	3.94
hub stub axles	1¼" BSP malleable iron pipe	2×350	700	4.71
hub tube	2" BSP malleable pipe	2×164	328	1.47
rollers	5 mm or 3/16" dia BMS[2]	60×152	9120	2.49
roller retaining rings	5mm or 3/16" dia BMS	4×167	668	0.20
axial thrust rollers	16 mm dia BMS[3]	4×8	32	not used
hub restraint bolts	M8 or M10 bolts ×70mm	70×4	280	1.28
wheel studs	12mm studding	8×70	560	2.55
wheel stud struts	6×40 black steel strip	4×37	148	0.68
stud washers	3×40 BMS strip 4	4×40	160	not used
TOTAL COST =				17.32

1 Axle could be one piece of pipe with the stub axles - see text.

2 BMS = bright mild steel round bar.

3 Thrust rollers can be made from a stack of washers. They are not essential but give better performance.

4 Backing washers, placed on the stud before the wheel, make the wheel more secure if it has large stud holes.

Cutting list and costs

The table shows a cutting list for a complete axle - two wheel hubs and stub axles joined by an angle iron section in the middle. We did this because angle iron was much cheaper than pipe. But if it is not where you are then make the whole axle out of one piece of pipe. Recent prices of materials in Nigeria are shown in $\$_{US}$. The 2" BSP (British Standard Pipe) is about 60.8mm outside diameter, or a bit less, with a wall thickness of about 3.6mm. The 1¼" BSP is about 42.9mm outside diameter, with a wall thickness of about 3.2mm.

Construction step by step

These instructions deal with making an axle to the design shown in the drawings. If you find that you cannot get the right sizes you might still be able to make an axle with other sizes. See the **Modifications** section on page 6.

1. The first and probably most difficult job, is to get some suitable pipes and roller material. Obviously the axle has to be strong enough to carry the cart, so it should **not** be made from pipe smaller than about 40mm outside diameter. You must make sure that the pipe has a wall thickness of more than 2.5 mm.

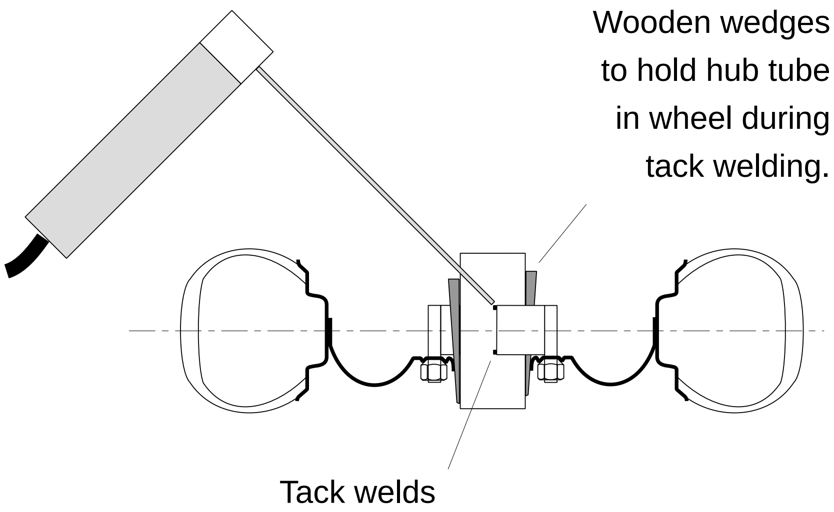
   The hub pipe must also have a wall thickness of 2.5mm or more. And it must have a bore (or inside diameter), which goes over the axle with enough room for rollers all the way around between it and the axle. Rollers must be 4.5mm, 3/16" or 5mm diameter (if they will go in). There can be quite a lot of play (slackness, looseness or clearance) between the rollers and the axle (say 1mm) - it does not have to be tight like an ordinary ball race.

2. When you have worked out how to get the right pipe and roller sizes, then you can cut the two hub pipes 164mm long and the two stub axles 300mm long. You also have to cut enough rollers to fill up between the stub axle and the hub pipes. You will probably need about 25 or 27 for each hub and the rollers must be about 152mm long. Do not try to squeeze the hub full of rollers - the best way to find out how many you need is to put as many in as you can and then take one or two out. You should clean up the ends of the rollers with a file after you have sawn them.

3. The next step is to weld the stud bolts or bits of threaded rod onto the struts. When you've done this you can put the studs into the wheels, put the nuts on, and get everything even and straight with the hub pipe in as well. You might find that holding the hub tube in with some wedges as shown in Figure 4 is a good way to do it. You also want to get the middle of hub pipe level with the middle of the tyre, as is shown in Figure 5. Most car wheels need the studs to be about 40mm offset and this is what is on the drawings. When you are happy, tack weld the struts to the hub pipe, remove the wheel and wedges so you've got room and weld the stud supports to the hub tube. Repeat this for the

other hub. If you are going to make several axles you can make up a simple jig, rather than the wedges, to hold everything for welding. We have used a piece of plywood with a central hole to fit snugly over the hub tube and four holes for the studs. In other words its a bit like a dummy wheel. Its best to check that the hubs, rollers and stub-axles still go together when you've finished welding - sometimes weld contraction can pull it all out of shape and make it all too tight. You might need to file off some high spots inside the hub. If you can get it together without a hammer you'll be ok because it will wear to the right shape.

Figure 5: CROSS SECTION OF TYRE WHEEL AND HUB TUBE DURING TACK WELDING OF STUD SUPPORT STRUTS



4. Now take the wheels off the hubs and make up four rings (called roller restraint rings on the drawing) from the same material as you used for the rollers. The rings have to be welded just inside the ends of hub pipes to stop the rollers falling out. When they're welded in you need to use a half round file to open the hole in the middle where the axle pipe will go so that it's got plenty of room - you do not want it rubbing on the axle.

5. Next drill the four holes for the cross bolts. Put the end ones about 15mm from the end of the axles. At the other end you need to make them just far enough away from the end of the hub so that you can turn the nuts to get them on. Again about 15mm seems to do the trick. It does not matter of they are a bit loose.

6. Nearly there! Now you need to cut two bits of angle iron and weld them together to make the center axle to join the stub axles. You must put the stub-axles in position when you weld - the contraction of the angle iron when the weld cools down clamps everything (if you are lucky). Otherwise

just put a few tack welds on to hold it in place.

7. Now put it together! Put some grease into the hub and put the right number of rollers in so that they are in place against the inside of the hub tube. You can hold them in with a bit of rag or a plastic bag if the grease does not stick them. Then just slide them onto the stub axles and fit the cross bolts. (If a plastic bag or rag was used to hold the rollers it should be pushed out by the axle.)

8. You've done it!

Modifications

If you cannot get suitable sizes of pipe and round bar, then pipes can be made slightly bigger (up to 1mm bigger) by forcing a short piece of round bar of the right diameter through them with a press. Another way to do it is to saw the pipe along its length and open to the right size and then weld it. You can also make it a bit smaller like this by cutting a wider slot and squashing the pipe down. Do not worry about the rollers rolling over the groove - as long as you clean the weld back flush with the tube it will be ok.

If you find that you cannot get anything like the materials talked about in the cutting list then maybe you can adapt the design a bit. If the hole in the middle of the wheels is big then you stand a better chance of finding a combination of pipes and rollers that will fit. You can often cut a bit out of the middle of the wheel to make the hole bigger. The hole in Land Rover wheels is big and you can get 4" pipe into them. You may find that you can use small pipe say 1/2" pipe to make the rollers for example. Of course the shaft does not have to be a pipe - it could be solid and then it could be a bit smaller, say 30mm diameter if the steel is high quality.

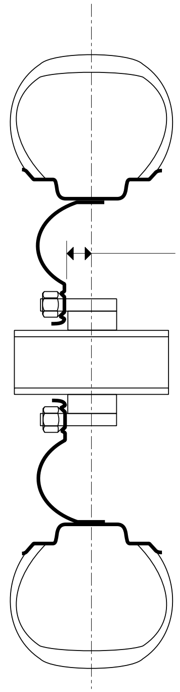


Figure 5: CROSS SECTION OF TYRE WHEEL AND HUB TUBE SHOWING CENTERING OF HUB TUBE IN WHEEL

This distance must be set to get the hub tube in the middle of the tyre.

Hub tube.

Getting the hub in the middle of the wheel and tyre means that the bearings in the hub are evenly loaded.

Another idea that we have tried is to use a hardwood as the hub and even the wheel. If you think about it, the wear on something which is rolling must be less than when something is sliding over it, so a wooden bearing should be better than a sliding one. Some bearings we have tried have had a steel ring fitted inside so that the rollers roll on this steel. We have also tried making these rings from round bar like wire so that it's like the rollers roll on the inside of a spring. This seemed to work quite well.

The main thing to remember with these bearings is that the rollers must be long compared to the diameter of the axle. In the axle and hub shown in this technical release the rollers are about four times as long (150mm) as the diameter of the axle (43mm). Also bigger rollers work better. In our design for Land-Rover wheels on bullock carts we use 3/8" or 10mm diameter rollers and we have successfully used rollers over 20mm diameter in some experiments.

Other ways of making the thrust bearings

We have tried some other ways of making the thrust rollers at the ends. You can have just one roller on each cross bolt and this one can have a countersunk hole in its end so that the

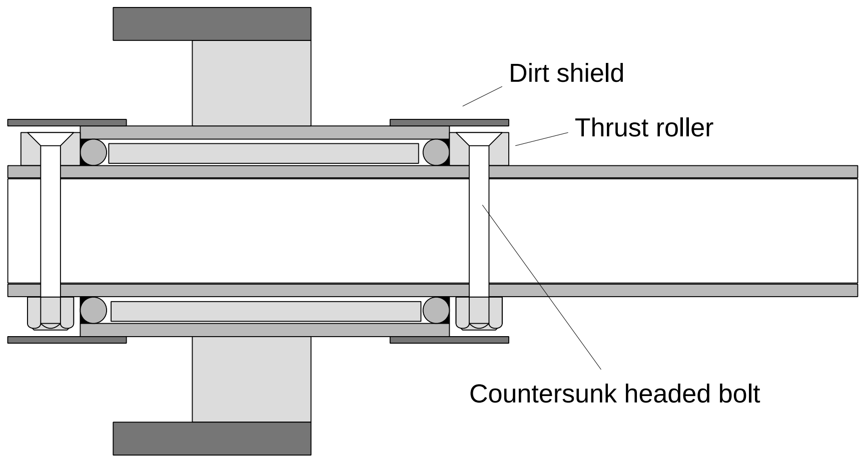


Figure 6: axial restraint rollers on countersunk head cross bolts.

cross bolt head (if its countersunk) can fit in and not stick out. This makes it easier to put dust shields on. You can see this arrangement in Figure 6.

Another way that we have used rollers is to put them inside the dirt shields without any bolts. You can see this in Figure 7. The rollers have no holes - they are just plain pieces of rod. You need to use big rod so that its diameter is a bit bigger than its length. If you do not do this the rollers can turn over and jam.

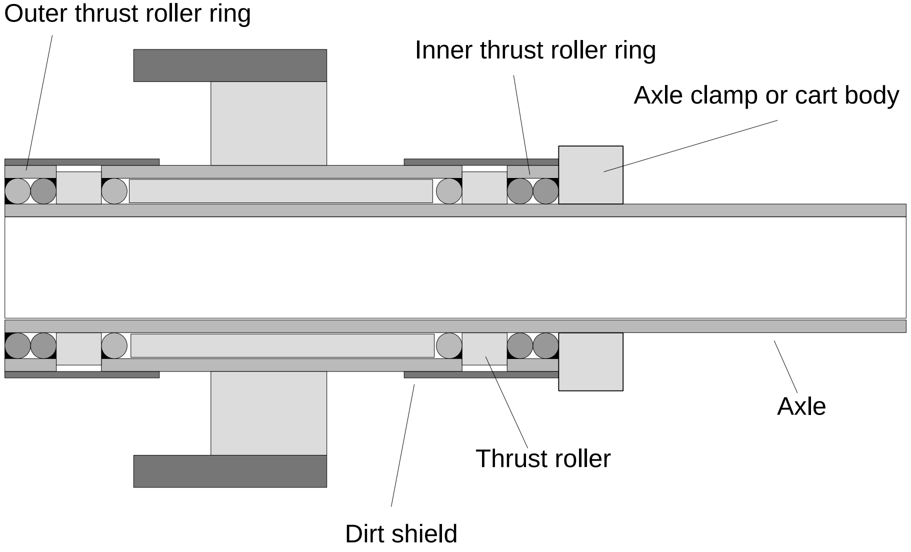


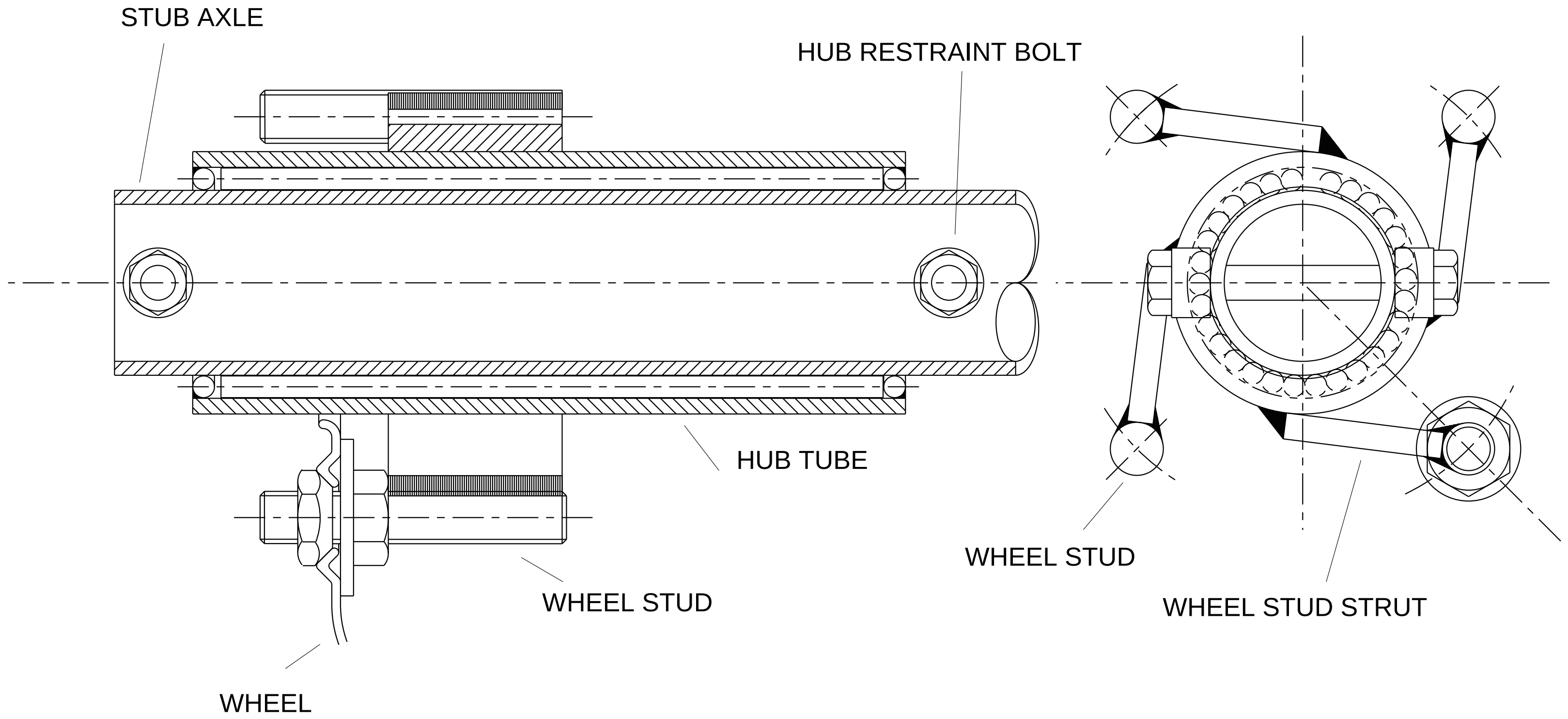
Figure 7: axial restraint rollers held inside dirt/ oil shields.

You must clean up the ends of the rollers with a file and round off the corners a bit so that they can slide inside the dirt shield and not catch on anything. The rollers roll against thrust roller rings made of the same pipe as the hub tube is made from and the same rod as the rollers. The outer thrust roller ring should be welded to the axle. The inner one is held on by the cart body or the clamps which hold the axle to the body. You can put one or more thrust rollers in.

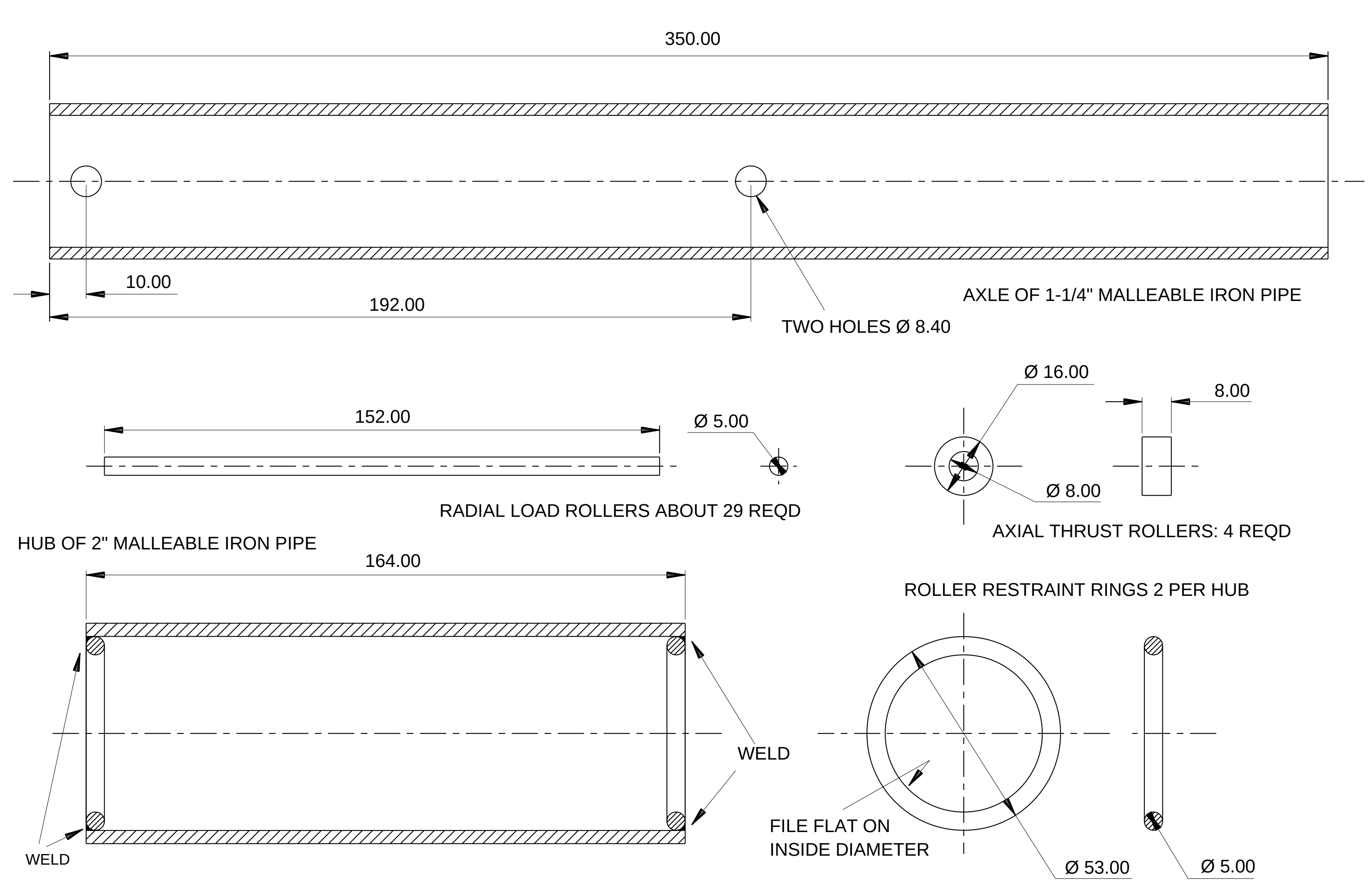
One more advantage of this method of axle construction is that you do not need to drill any holes.

Other DTU cart developments

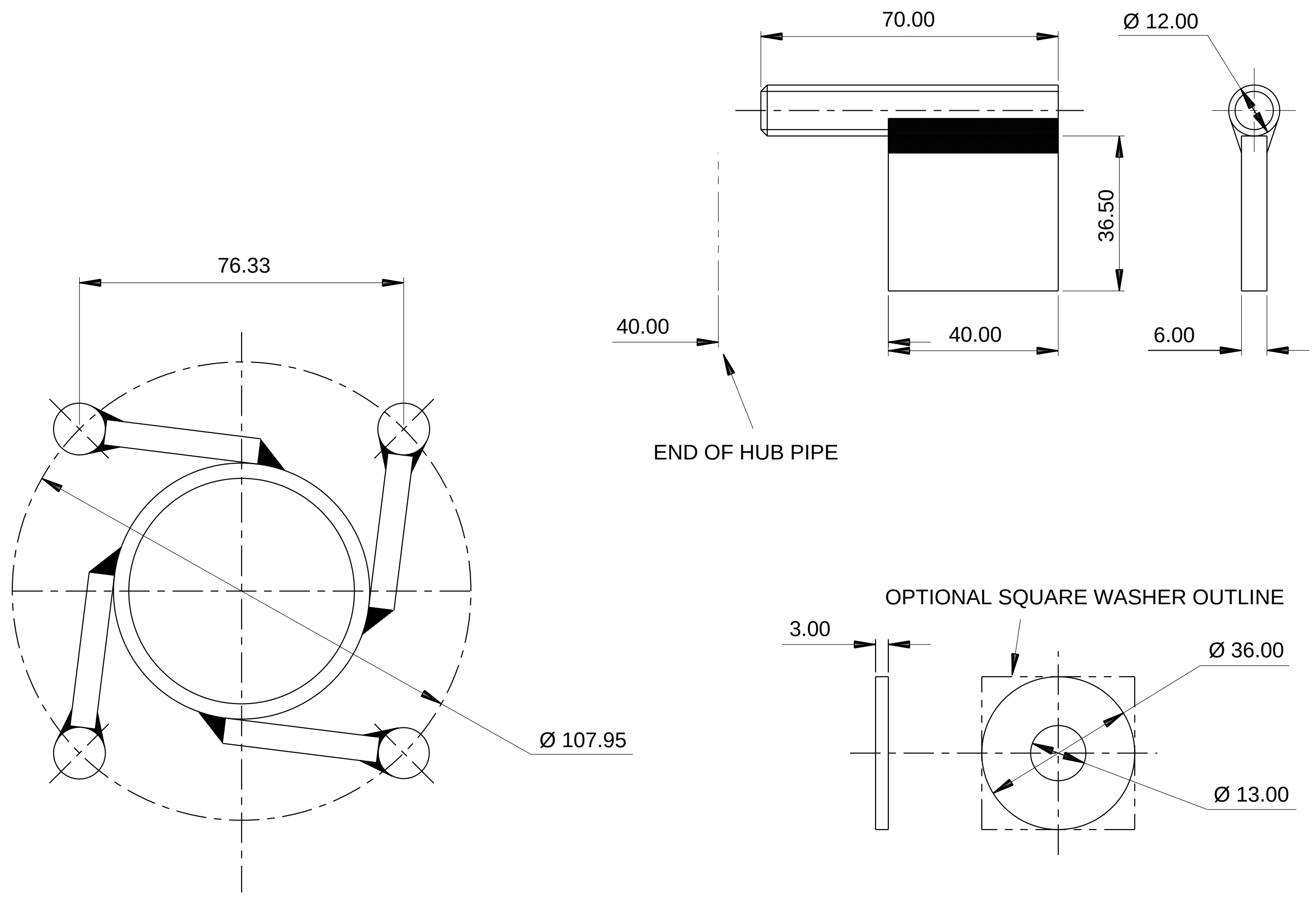
Other methods of hub design using aluminium castings, for example, which might need no machining, are under development at Warwick and wheel designs in steel sheet, cast aluminium and timber are also in manufacture or under development. A range of designs for donkey and ox carts made of steel and wood, is also available, some of which are in production in Nigeria.



GENERAL ASSEMBY DRAWING	Scale	10mm	Title PIPE AND ROLLER DONKEY CART AXLE	Drawn by	CEO
	Date	11-2-93		Dwg No.	1/3



	HUB AND AXLE COMPONENTS	Scale	10mm	Title PIPE AND ROLLER DONKEY CART AXLE	Drawn by	CEO
		Date	11-2-93		Dwg No.	2/3



	WHEEL MOUNTING COMPONENTS	Scale	10mm	Title PIPE AND ROLLER DONKEY CART AXLE	Drawn by	CEO
		Date	11-2-93		Dwg No.	3/3

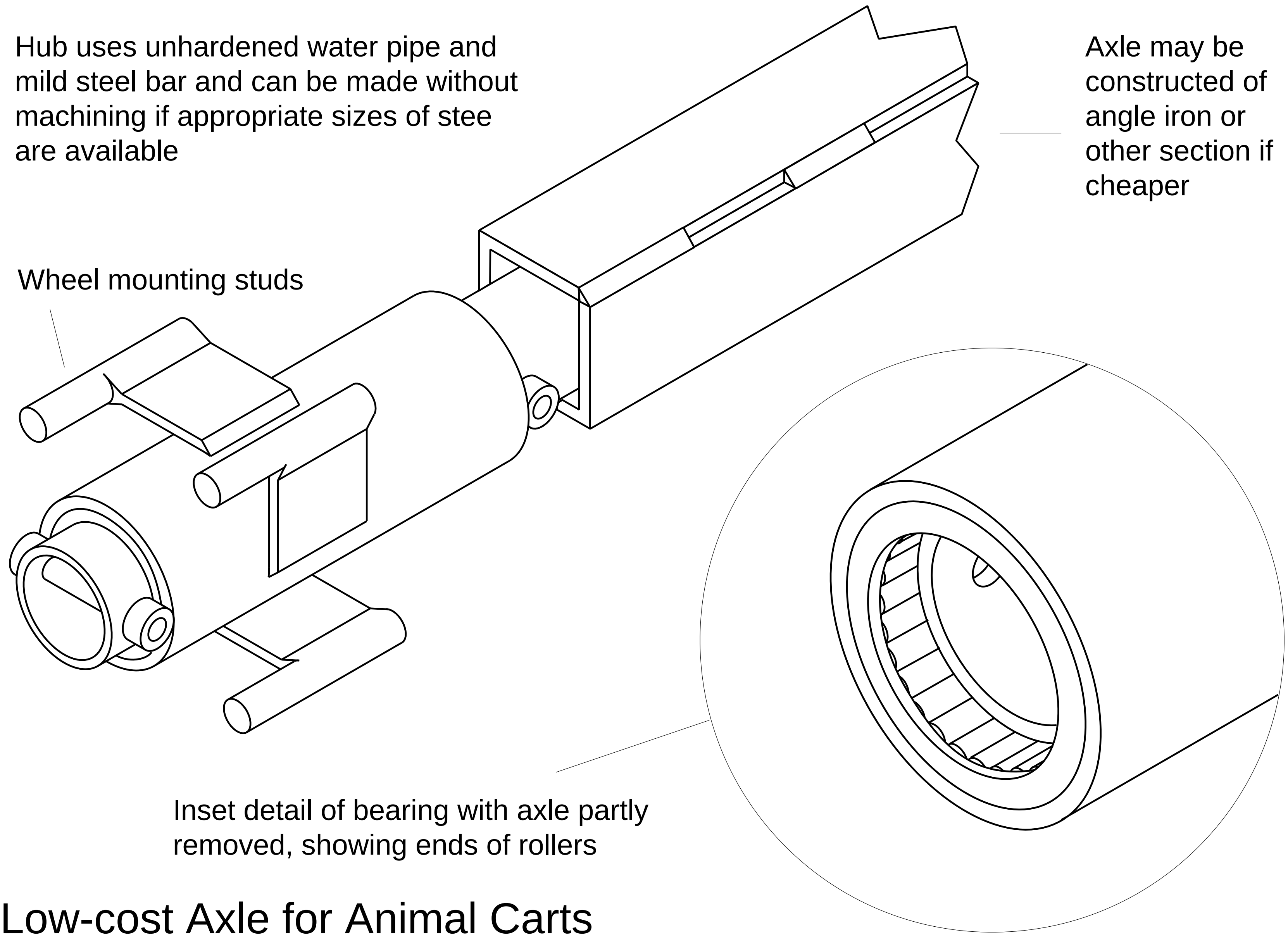
DTU

Animal Cart Programme

TECHNICAL
22
RELEASE

PIPE AND ROLLER AXLE FOR OX CARTS

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@uk.ac.warwick.eng



Low-cost Axle for Animal Carts

Pipe and Roller Ox-Cart Wheel Hubs

Introduction

Not enough farmers in Africa have animal carts. Those who have carts can take their produce to places where they can get the best prices. They can also get into town and buy fertilizer and better seeds and move stuff around their farm easier. The trouble is that carts are too expensive for many farmers. The question is what can be done about it?

Carts are made in many different places. Some carts are made in factories in industrial countries and some are made in factories in Africa, but most are made by local blacksmiths or carpenters using scrap car and Land-Rover axles. These people cannot get enough axles to meet the demand, so the price is high. Another problem is that the axles are often so worn that they do not last long. Lots of farmers take the differential unit out of the axle too, which makes it break sooner and lets the dirt in.

What you need is an axle which blacksmiths and fabricators can make with fairly simple tools - without having to get parts machined. There are usually blacksmiths and fabricators in the small market towns used by the farmers. Experts think that having the cart maker close to the farmer is a good thing because they can talk to each other easily and sort out any problems. And of course if the cart is made locally, it can be repaired locally, so there won't be problems with spare parts.

Idea behind design

The idea behind the design of axle described in this technical release is to allow axle construction without the use of machine tools (lathes and milling machines), and using materials which should be readily available. The materials can be used 'as bought' - no hardening of any of the components is needed, and you do not need to be super accurate - good hand working is good enough. The only tools which you must have are a hacksaw, a file and a drill able to drill a 13mm hole in steel. Having a vice is also very handy!

Of course if you do have power tools - especially a power hacksaw or cutoff wheel - things can be made much faster. This axle is suitable for a wide range of production methods - you could even tool up for manufacture with some specially made tools!

The long thin needle rollers in this design have been used so that the hubs will usually fit scrap Land-Rover and Japanese four-wheel-drive vehicle wheels. These wheels have a hole in the middle for the axle which is 115mm diameter (ie 4½") on a

Land-Rover wheel, but is smaller on the Japanese vehicles. If the wheel won't fit you can sometimes saw or file the hole bigger. Putting the fixing studs on struts like you see in the drawings means that you might be able to bend the struts a bit to fit a different wheel. Or if that doesn't work you can even cut nearly through the welds and then weld them in the right place. You could even cut the struts right off and weld on a different number, if your wheels have a different number of holes.

Performance

We have tested smaller versions of axles like these for more than 10 000km in the laboratory and some axles have been ok for 30 000km. With the smaller axles we usually set the load at 200kg per wheel, but we have used them at 400kg for thousands of km. With these big axles we expect double the load capacity - say up to 800kg on each wheel. You will probably find that the axles need to be cleaned and regreased every six months or year depending on how much they are used. The materials that you use do not have to be perfect - some of the axles we have tested have not been very round at all - one was more than 1mm out, but they have still worked. We have tried rollers made out of 6" nails with the heads cut off on the small version of the hub. They still worked even though they were slightly bent. Another material which might work for small rollers is welding electrode with the flux knocked off. Of course none of these axles are as good as axles with proper bearings in them, but they are a lot cheaper and easier to make and they should still last many years.

Cutting list and costs

Table 1 shows a cutting list for a complete axle - two wheel hubs and stub axles joined by an angle iron section in the middle. We did this because angle iron was much cheaper than pipe. But if it isn't where you are, then make the whole axle out of one piece of pipe. Recent prices of materials in Nigeria are shown converted to $\$_{US}$. The 3"BSP (British Standard Pipe) is about 89mm outside diameter with a wall thickness of up to 5mm. The 2"BSP is about 61mm outside diameter with a wall thickness of about 4.5mm.

Construction step by step

These instructions deal with making an axle to the design shown in the drawings. If you find that you can't get the right sizes you might still be able to make an axle with other sizes. See the Modifications section on page 6.

1. The first and probably most difficult job, is to get some

Table 1 Cutting list and costs for pipe and roller ox cart axle				
component	material	number & length required [no.×mm]	total material in axle [mm]	materials cost in Nigeria [$us]
central axle	75mm (3") angle iron[1]	2×1200	2400	2.98
hub stub axles	2" BSP malleable iron pipe	2×500	1000	5.67
hub outer race	3" BSP malleable pipe	2×250	500	1.24
rollers	10 mm or 3/8" dia BMS[2]	40×228	9576	26.49
roller retaining rings	10 mm or 3/8" dia BMS	4×219	880	2.43
axial thrust rollers	25 mm dia BMS[3]	4×12	48	0.41
hub restraint bolts	M12 bolts ×100mm	4×100	400	1.28
wheel studs	12mm studding	8×70	560	2.55
wheel stud struts	6×40 black steel strip	5×62	310	0.47
stud washers	3×40 BMS strip 4	5×40	200	0.14
TOTAL COST =				43.66
1 Axle could be one piece of pipe with the stub axles - see text.				
2 BMS = bright mild steel bar.				
3 Thrust rollers can be made from a stack of washers. They are not essential but give better performance in dusty environments.				
4 Backing washers, placed on the stud before the wheel, make the wheel more secure if it has large stud holes.				

suitable pipes and roller material. Obviously the axle has to be strong enough to carry the cart, so you must make sure that the pipe has a wall thickness of more than say 3.5 mm.

The hub pipe must also have a wall thickness of 3.5mm or more. It must also have a bore (or inside diameter), which goes over the axle with enough room for rollers all the way around between it and the axle. Rollers must be 9mm, 3/8" or 10mm diameter (if they will go in). There can be quite a lot of play (looseness, space or clearance) between the rollers and the axle (say 1mm) - it does not have to be tight like an ordinary ball race.

2. When you have worked out how to get the right pipe and roller sizes, then you can cut the two hub pipes 250mm long and the two stub axles 450mm or so long. You also have to cut enough rollers to fill up between the stub axle and the hub pipes. You will probably need about 20 or 21 for each hub and the rollers must be about 228mm long. Do not try to squeeze the hub full of rollers - the best way to find out how many you need is to put as many in as you can and then take one or two out. You should clean up the ends of the rollers with a file after you have sawn them.

3. The next step is to weld the stud bolts or bits of threaded rod onto the struts. When you've done this you can put the studs into the wheels, put the nuts on, and get everything even and straight with the hub pipe in as well. You might find that holding the hub tube in the wheel with some wedges, as shown in Figure 2 makes things easier. Also don't forget that you want to get the middle of hub pipe level with the middle of the tyre, as is shown in Figure 3. Most wheels need the studs to be about 40mm offset and this is what is on the drawings. When you are happy, tack weld

the struts to the hub tube, take the wheel off so you get at the tube easily and weld the stud support struts to the hub tube. Repeat this for the other hub. If you are going to make several axles you can make up a simple jig to hold everything for welding. We have used a piece of 18mm plywood with a central hole that fits the hub tube tightly and has a hole for each of the studs. Really this is like a dummy wheel. After welding its best to check that the hubs, rollers and stub-axles still go together - sometimes weld contraction can pull it all out of shape and make it all

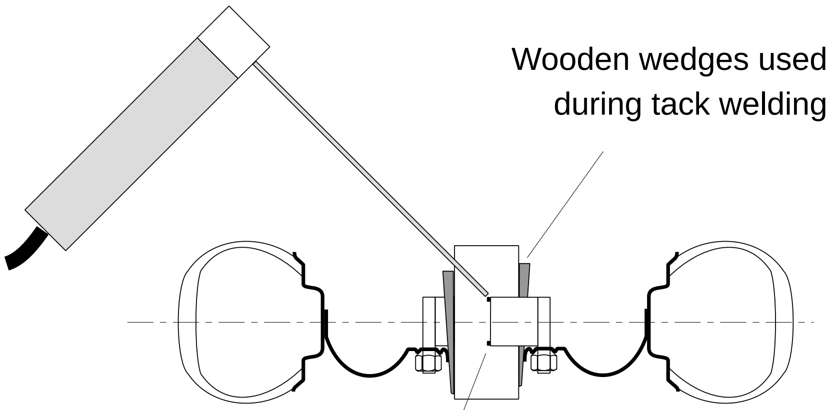


Figure 2: CROSS SECTION OF TYRE WHEEL AND HUB TUBE DURING TACK WELDING OF STUD SUPPORTS

too tight. You might need to file off some high spots inside the hub or you may even have to use smaller rollers. If you

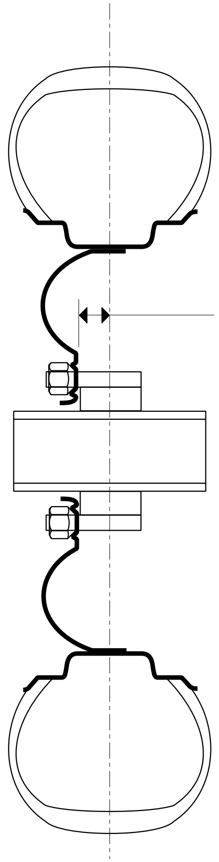


Figure 3: CROSS SECTION OF TYRE WHEEL AND HUB TUBE SHOWING CENTERING OF HUB TUBE

can get it together without a hammer you'll be ok because it will wear to the right shape.

4. Now take the wheels off the hubs and make up four rings (called roller restraint rings on the drawing) from the same material as you used for the rollers. Bend them in a vice if you have it, and cut the rod so that there is say a 4mm gap between the ends when they are the right diameter. Then you can push them into the end of the hub tube and weld the ends together. It's best to remove them then from the tube and clean up the weld with a file. The rings have to be welded just inside the ends of hub pipes to stop the rollers falling out so do this next. When they're welded in you need to use a half round file to open the hole in the middle where the axle pipe will go so that it's got plenty of room - you don't want it rubbing on the axle.

5. Next drill the two holes in each stub axle for the cross bolts. Put the end ones about 20mm from the end of the stub axles. You need to make the holes for the other two bolts just far enough away from the end of the hub so that you can just get the thrust rollers on. Probably the best way to do it is to assemble the hubs on the axles after you've drilled the first holes, offer up the other roller and mark the position. It doesn't matter of the hubs are a bit loose on the axles.

6. Nearly there! Now you need to cut two bits of angle iron and weld them together to make the center axle to join the stub axles. You must put the stub-axles in position when you weld - the contraction of the angle iron when the weld cools down clamps everything (if you are lucky). Otherwise just put a few tack welds on to hold it in place.

7. Now put it all together! Put some grease into the hub and put the right number of rollers in so that they are in place against the inside of the hub tube. You can hold them in place with a bit of rag or a plastic bag if the grease does not stick them. Alternatively you can put the hub a little way onto the shaft and then just ease the rollers into place one by one. Then just slide the whole thing onto the stub axles and fit the cross bolts. (If a plastic bag or rag was used to hold the rollers it should be pushed out by the axle.)

8. You've done it!

Modifications

If you cannot get suitable sizes of pipe and round bar, then pipes can be made slightly bigger (up to 1mm bigger) by forcing a short piece of round bar of the right diameter through them with a press. Another way to do it is to saw the pipe along its length and open to the right size and then weld it. You can also make it a bit smaller like this by cutting a wider slot and squashing the pipe down. Don't worry too much about the rollers rolling over a groove, but you will need to clean any flash or weld bead off from the bore.

If you find that you cannot get anything like the materials talked about in the cutting list then maybe you can adapt the design a bit. If the hole in the middle of the wheels is big then you stand a better chance of finding a combination of pipes and rollers that will fit. You can often cut a bit out of the middle of the wheel to make the hole bigger. The hole in Land Rover wheels is big and you can just get 4" pipe into them. You may find that you can even use small pipe say 1/2" BSP pipe to make the rollers for example, but you must make sure that it has a thick wall. Of course the shaft does not have to be a pipe - it could be solid and then it could be a bit smaller, say 38mm diameter or bigger if the steel is high quality.

Another idea that we have tried is to use a hardwood as the hub and even the wheel. If you think about it, the wear on something which is rolling must be less than when something is sliding over it, so a wooden bearing should be better than a sliding one. Some bearings we have tried have had a steel ring fitted inside so that the rollers roll on this steel. We have also tried making these rings from round bar like wire so that it's like the rollers roll on the inside of a spring. This seemed to work quite well.

The main thing to remember with these bearings is that the rollers must be long compared to the diameter of the axle. In the axle and hub shown in this technical release the rollers are about four times as long (228mm) as the diameter of the axle (about 60mm). Another rule is that bigger diameter rollers work better.

Other thrust bearing arrangements

We have tried some other ways of making the thrust rollers at the ends. You can have just one roller on each cross bolt and this one can have a countersunk hole in its end so that the

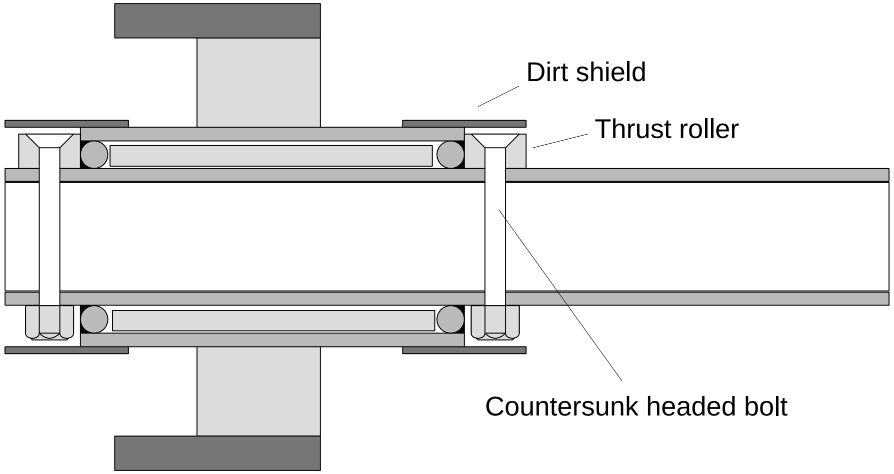


Figure 6: axial restraint rollers on countersunk head cross bolts.

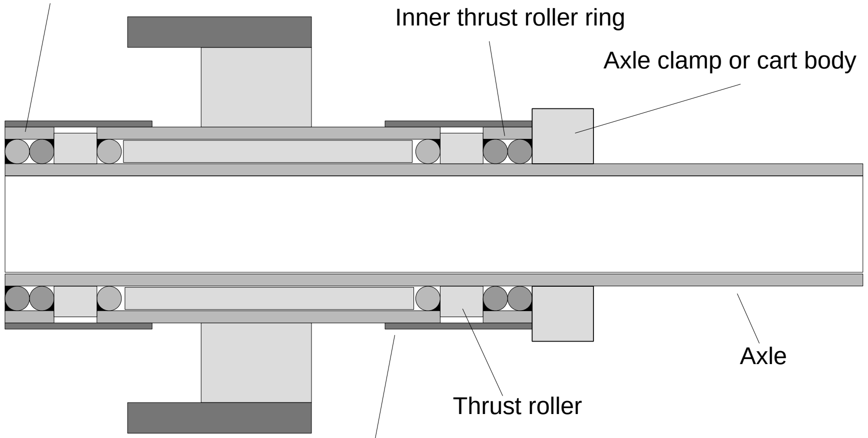


Figure 7: axial restraint rollers held inside dirt/ oil shields.

cross bolt head (if it is countersunk) can fit in and not stick out. This makes it easier to put dust shields on. You can see this arrangement in Figure 6.

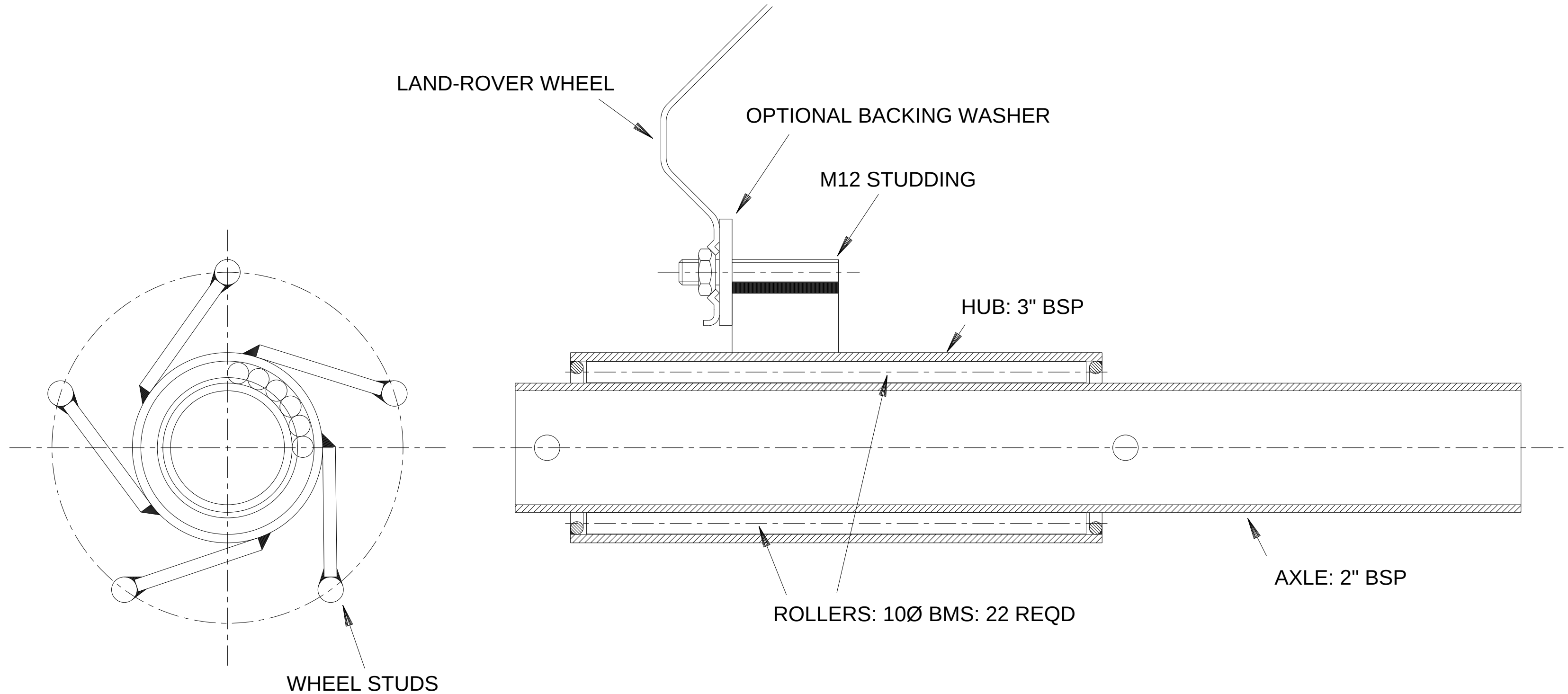
Another way that we have used rollers is to put them inside the dirt shields without any bolts. You can see this in Figure 7. The rollers have no holes - they are just plain pieces of rod. You need to use big rod so that its diameter is a bit bigger than its length. If you do not do this the rollers can turn over and jam. You should clean up the ends of the rollers with a file and round off the corners a bit so that they can slide inside the dirt shield and not catch on anything. The rollers roll against thrust roller rings made of the same pipe as the hub tube is made from and the same rod as the rollers. The outer thrust roller ring should be welded to the axle. The inner one is held on by the cart body or the clamps which hold the axle to the body. You can put one or more thrust rollers in.

One more advantage of this method of axle construction is that you do not need to drill any holes.

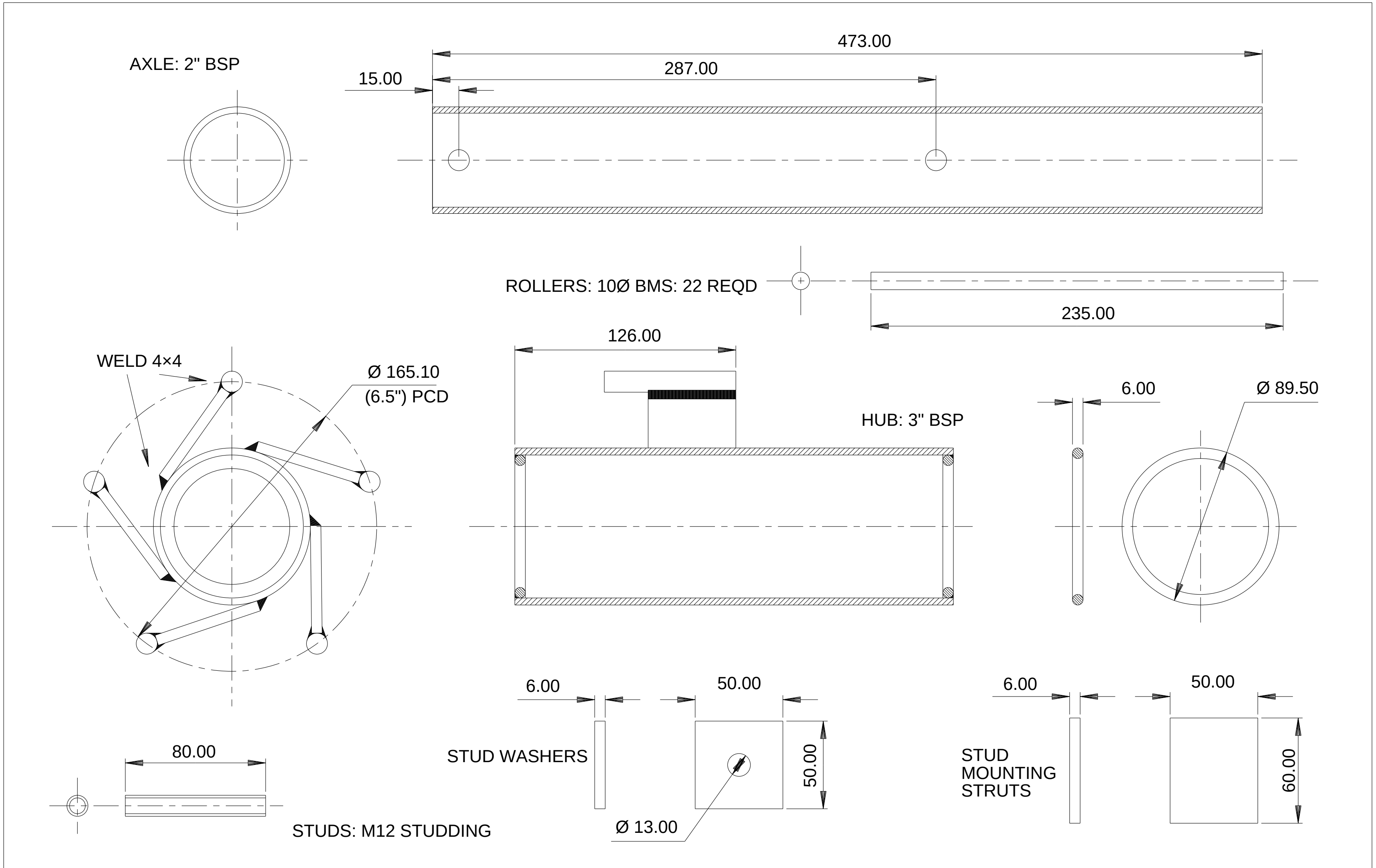
Other DTU cart developments

Other methods of hub design using aluminium castings, for example, which might need no machining, and others which need only limited machining are under development at Warwick and wheel designs in steel sheet, cast aluminium and timber are also in manufacture or under development. We have also been working on a variety of other bearing types including pressed cup-and-cone ball bearings. A range of designs for donkey and ox carts made of steel and wood, is also available, some of which are in production in Nigeria.

If you are interested we can send you more information.



					Scale		Title PIPE AND ROLLER AXLE FOR OX CARTS	Drawn by	CEO
					Date	17-3-93		Dwg No.	1/2



PIPE AND ROLLER AXLE FOR OX-CARTS	Scale		Title COMPONENTS		CEO
	Date	17-3-93		Dwg No.	2/2

DTU

Animal Cart Programme

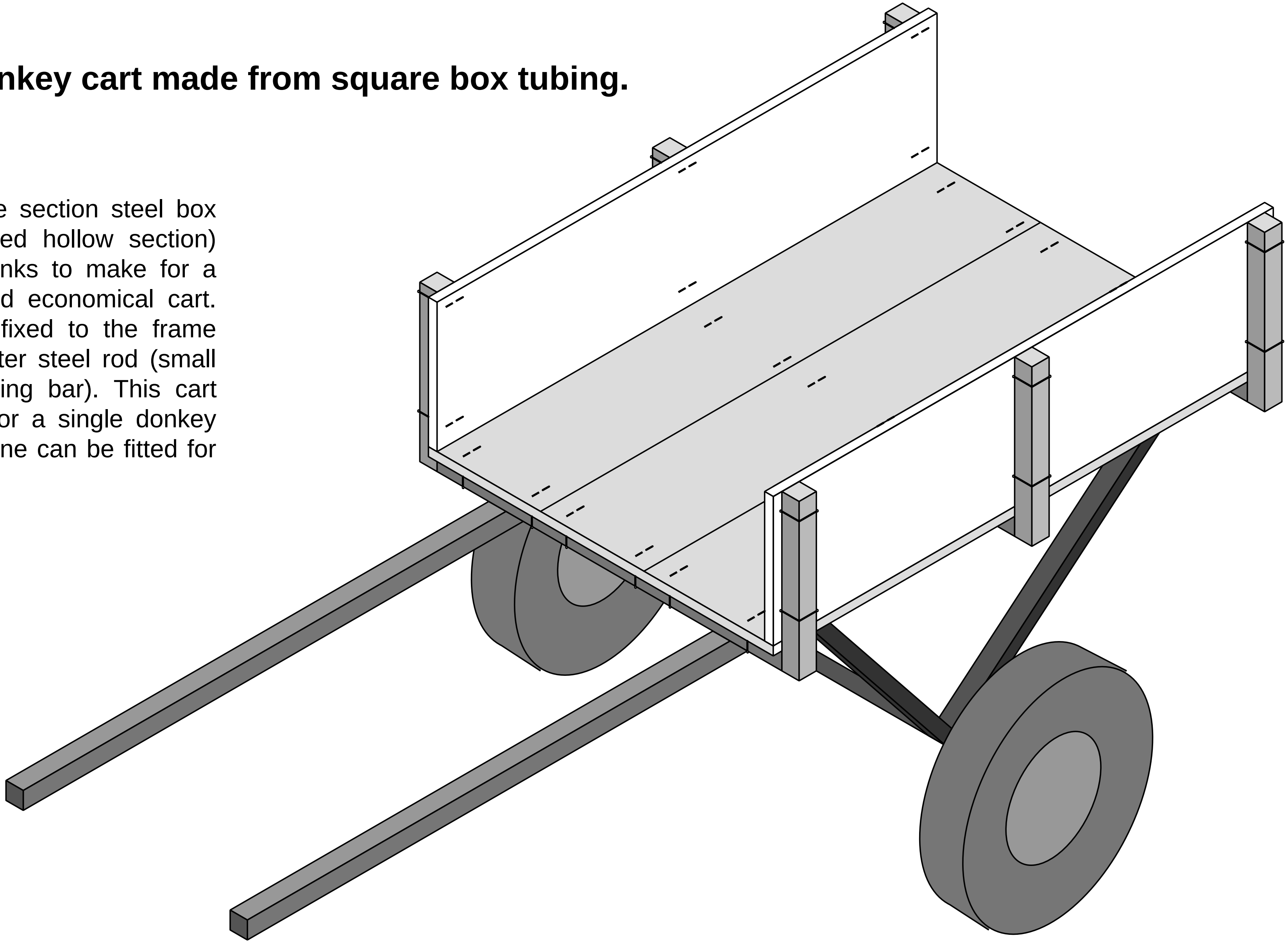
TECHNICAL
23
RELEASE

STEEL FRAME AND WOOD DONKEY CART

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@uk.ac.warwick.eng

Figure 1: donkey cart made from square box tubing.

Cart uses square section steel box tubing (RHS rolled hollow section) plus wooden planks to make for a quickly made and economical cart. The planks are fixed to the frame with 6mm diameter steel rod (small concrete reinforcing bar). This cart has two shafts for a single donkey but one central one can be fitted for two animals.



Donkey Cart Body Made From Square Box Tubing and Timber Planks

Introduction

Not enough farmers in Africa have animal carts. Those who have carts can take their produce to places where they can get the best prices. They can also get into town and buy fertilizer and better seeds and move stuff around their farm easier. The trouble is that carts are too expensive for many farmers. The question is what can be done?

Carts are made in many different places. Some carts are made in factories in industrial countries and some are made in African factories, but most are made by local blacksmiths or carpenters using scrap car and Land-Rover axles. These people cannot get enough axles to meet the demand so the price is high. Even if they did have the axle, they still end up building heavy bodies that take ages to make. In another booklet in this series we have told you how you can make simple low-cost axles; in this booklet we tell you about a simple steel and timber body. You should find that you can make the body for about $\$_{US}$ 35 depending on the cost of the materials and labour. Once you get organised, two men can probably make two bodies per day.

What you need is a body which carpenters and fabricators can make with their simple tools. These people will probably be in the small market towns used by the farmers and they will have an electric welder and some basic handtools like a hacksaw. Experts think that having the cart maker close to the farmer is a good thing because they can talk to each other easily and sort out any problems. And of course if the cart is made locally, it can be repaired locally, so there should not be any problems with spare parts.

Idea Behind Design

The idea behind the design of donkey cart described in this technical release is to allow construction without lots of special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a welder, a hacksaw, and a drill able to make a 6mm hole in wood. In fact you can make the drill yourself. You might find that a couple of 4" or a 5" G clamps (or something like it) are useful too. (The symbol " means inches here, so that 4" is about 100mm since there are about 25mm in an inch.)

You will see that there are no mitres and funny angles to cut in the square tubing so you save time when making the cart. Also the exact lengths of the components are not very fussy - again it saves a little time. But you will find that the carts look better if you take trouble to get things square and even etc.

These carts have been tested a bit in Nigeria, but we have not tested them enough. The only problems which we have found so far have been breaking of the animal shafts and we have cured this by welding some strengtheners (bits of 8mm, 10mm or 12mm round bar or re-bar) to the top and bottom of these shafts. (Re-bar means concrete reinforcing bar). The construction tends to be a bit light in Nigeria, because they sell very thin wall square tubing (much less than 1mm thickness) there. If you used tubing with a wall thickness of 2.5mm or more you probably would not need to put these strengtheners on. Really to get a cart at a reasonable cost you need to experiment a bit to see how the farmers treat their carts and what they expect them to stand. It's no good saying it must be strong enough so that they cannot ever break it - somebody will always break anything - and it will be very expensive to make it nearly unbreakable. At least like this you can repair it easily and cheaply.

Cutting list and costs

Table 1 shows a cutting list for a complete cart. Recent prices of materials in Nigeria are shown converted to $\$_{US}$. The square box tube (sometimes called rolled hollow section or RHS) is nominally 2" or 50mm on one side. You can often buy it in a variety of wall thicknesses. It's best with a with a wall thickness of 2.5mm (12 gauge) or more, but we have used wall thicknesses down to 1.6mm (16 gauge) as we mentioned earlier.

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete. Start by cutting the square box tube into the right lengths, as in the cutting list above. Then cut the various bits of re-bar or whatever you are going to use. You could cut the timber also at this stage, but it might be easiest to leave it till last.

2) Next weld the three U-shaped frames together. You might find the method shown in Figure 2 the easiest way to support the components during welding. It's quick and if you do not tighten the clamps too tight to start with, you can tap the bits with a hammer until everything is square and

TABLE 1: cutting list for steel framed donkey cart.

component	material	number of lengths and length required [no.×mm]	total material in cart [mm]	materials' cost in Nigeria [$us]
animal shafts	50×50 RHS	2×3150	6300	8.94
body frame bottoms	50×50 RHS	3×1100	3300	4.68
body frame sides	50×50 RHS	6×300	1800	2.55
axle struts	50×50 RHS	4×600	2400	3.40
shaft strengtheners	8mm to 12mm round bar[1]	8×600	4800	1.02
axle strut braces	8mm to 12mm round bar	2×850	1700	0.36
axle fixing studs	M12 threaded rod or bolts	2×100	200	0.64
axle fixing loops	6mm dia re-bar or similar	2×200	400	0.04
plank fixing staples	6mm dia re-bar or similar	30×250	7500	0.80
tray bottom planks	1"×12" or similar timber	4×1800	7200	4.26
tray side planks	1"×12" or similar timber	2×1800	3600	2.13
tray ends	1"×12" or similar timber	2×1100	2200	1.30
TOTAL->				30.12

1 The round bar can be anything actually - it doesn't even have to be round, so deformed or high-yield re-bar is fine. You could even use flat strip as long as its more than say 8mm thick.

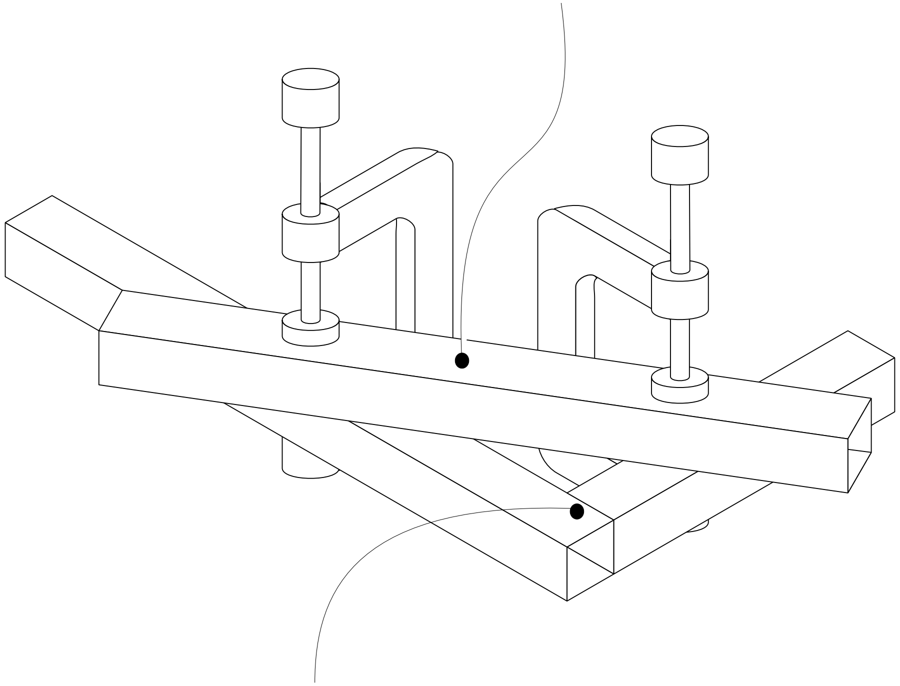


Figure 2: supporting components during welding.

straight. Then tighten the clamps before you weld.

3) Then take two axle support strut pieces and weld them together to make an L shape. When you do this you need to weld only two edges together to leave a space for the axle as Figure 3 shows. You can then weld this L piece to an animal shaft.

Repeat the process using the first sub assembly as a pattern for the second, but remember that the second assembly must be a mirror image of the second. In other words you will need to put the first assembly upside down on the ground and assemble the second one on top of it, as shown in Figure 4. make sure you do not weld the two assemblies together!

4) It may be easiest to fix the axle retaining bolts and loops on at this stage so that you can use them in the next stage.

5) Now you can weld the three U-frames on top of the animal shafts after you have checked for squareness etc.

6) Nearly there! Now you need to bend the staples to hold the wood onto the square tube. You could make up a jig for this with some bolts in a piece of wood or you can just bend them in a vice. They do not have to be very accurate, but as usual the more accurate the better. To fit them, drill a hole both sides of the tubing and push both legs of the U through the wood using a hammer if necessary. When it's through, hold a hammer against the bottom of the U whilst you knock the protruding legs over with another hammer. You might find this easier with someone to help you. Then clench the legs by knocking them into the surface of the wood to leave the surface flush. Once you are happy that

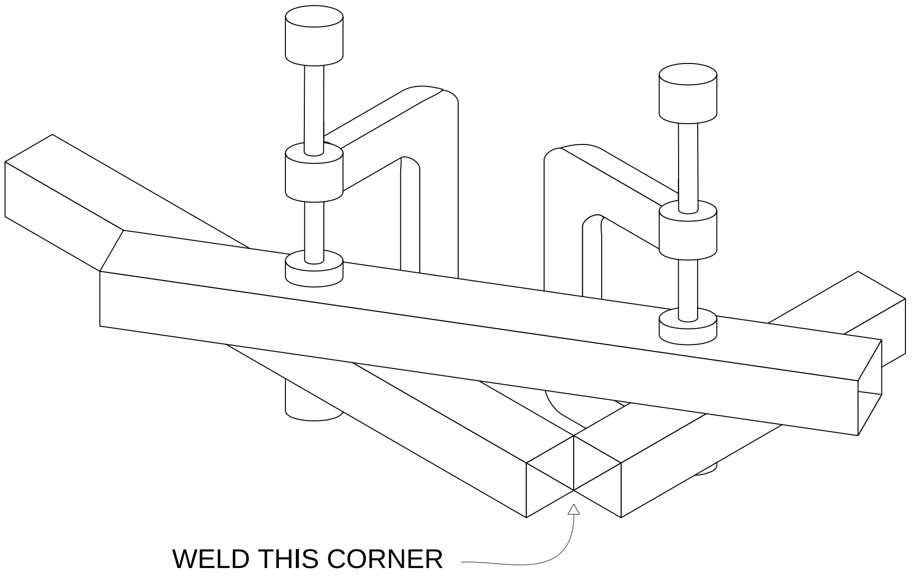


Figure 3: welding axle supports.

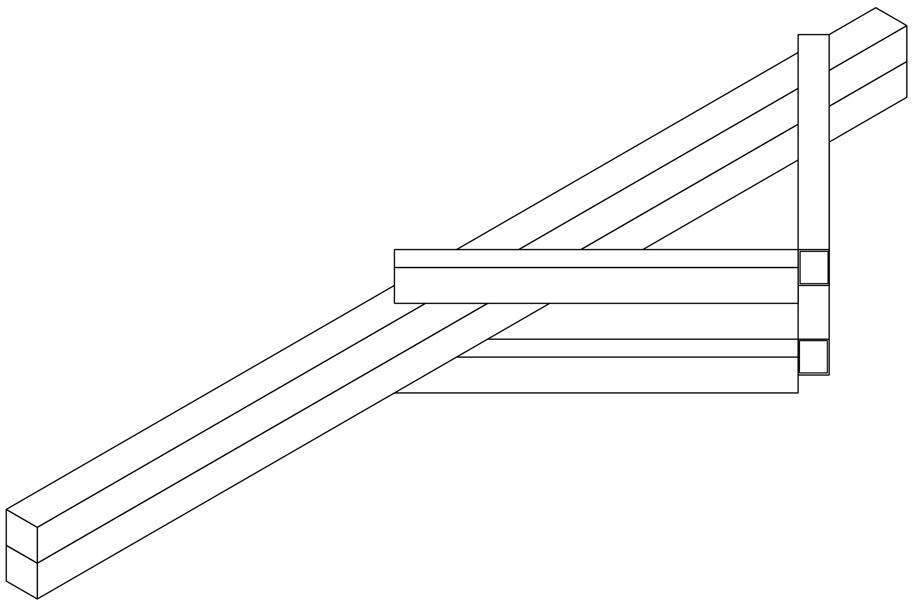


Figure 4: welding animal shaft and axle support struts assemblies.

all is in the right place, weld the staple to the square tubing with a couple of substantial tack welds, as shown in Figure 5. Then put the rest of the staples in in the same way.

7) Paint the cart. You've finished it!

Modifications

There are many different versions of this cart. This one has sides, for example, but you could make these removable by not using staples to fix the sides. A good way to fix the side planks might be to leave the steel uprights in place and tie the planks to the inside of these with bits of truck inner tube as giant bungees or elastic bands. This is an appropriate way because it is cheap and very easily repairable, but the farmers may want some flashy looking thing which will be very expensive to make. You will probably find that things like latches take as much time as the basic parts of the cart.

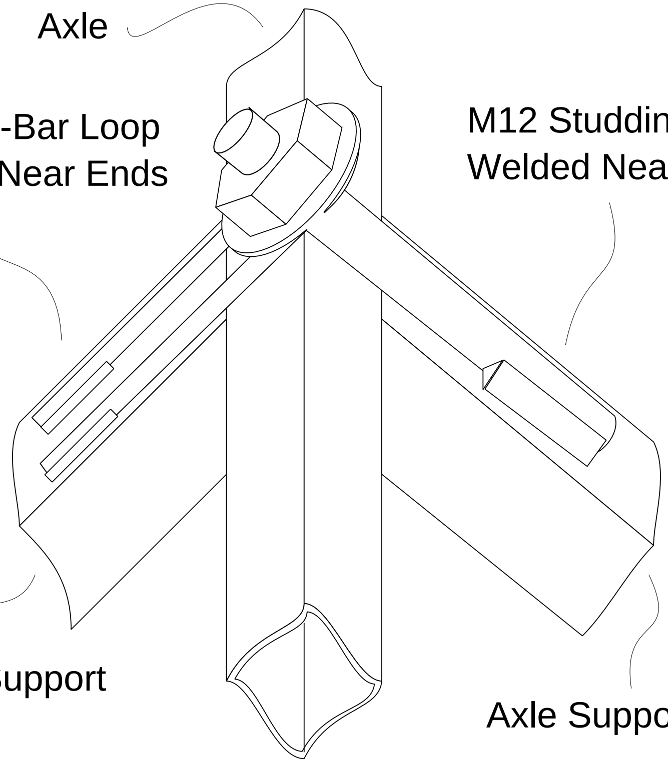


Figure 5: method of fixing axle to axle supports.

You can try longer and shorter carts and you can make them wider or narrower. When you do this, check the length and width of the planks of wood that you will use - you do not want to find that you are two inches short of being able to get two

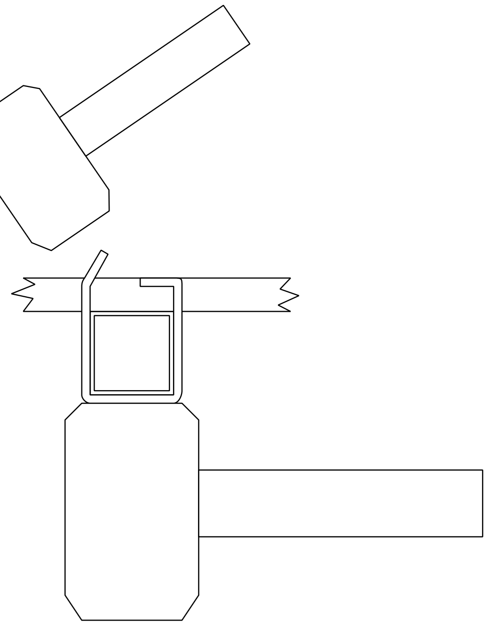


Figure 6: Using two hammers to clench wood fixing staple.

runs of plank out of one piece of timber, or that its just too narrow and you have to fiddle about and waste time filling the gap with an extra little strip.

Another thing is the height of the cart. No-one seems to know why some carts are made high and others low. Its better for the animals if the load tray is low particularly if the carts will be operated a lot over rough ground. But you may find that farmers want a high load tray to keep loads dry if they are fording rivers a lot, or it may be a status thing (if you are wealthy enough to own a cart, your eye level must be above a pedestrian's). Or it may be that it's too tiring to load a low cart - if, for example, you have to bend your back twice for each bundle - once to pick it up off the ground and again to put it down onto a low load tray. We have found that farmers usually want the body to come out over the wheels so they can load on lots of straw or light materials.

Other DTU cart developments

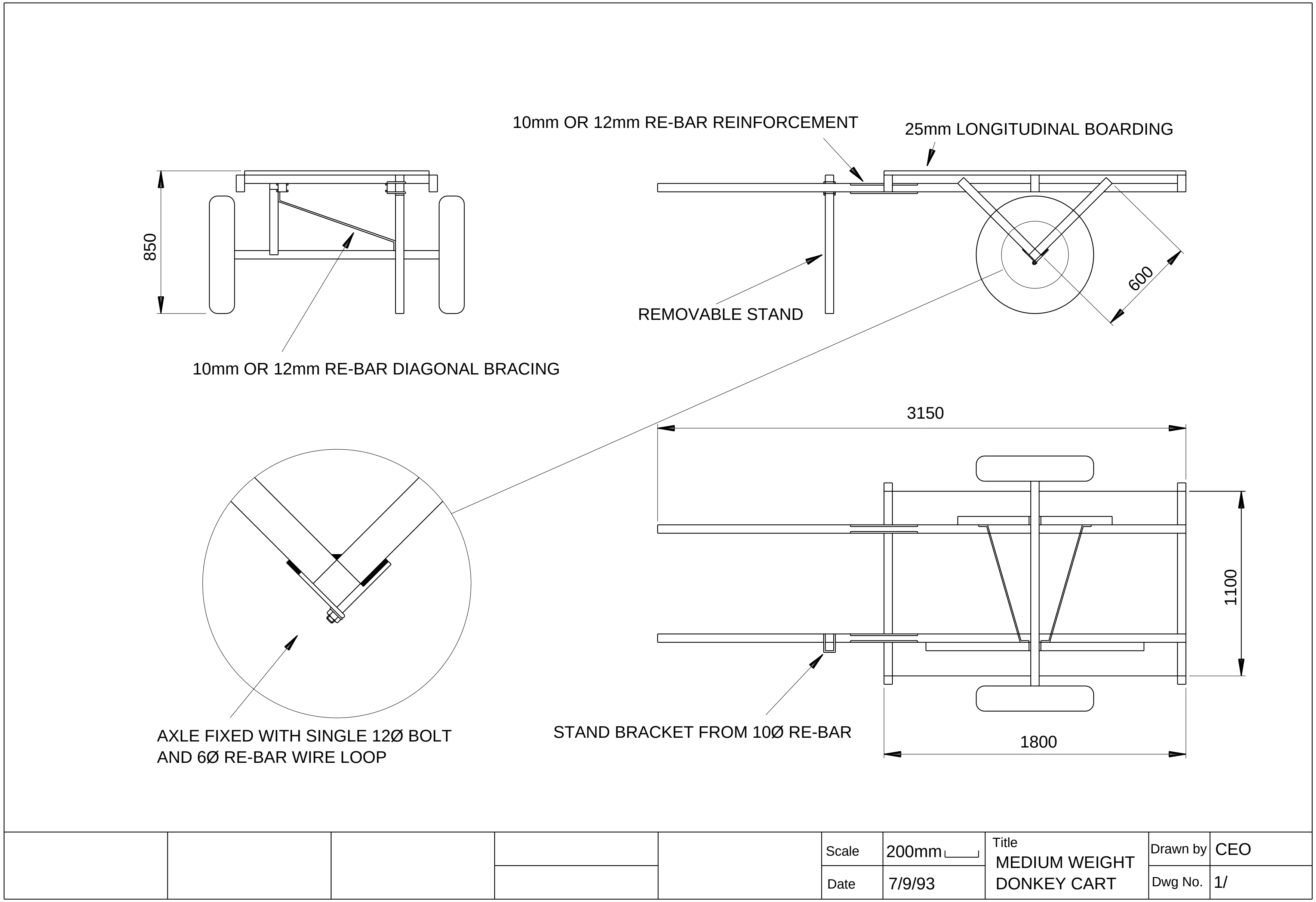
The DTU has been working on a range of cart body types for use with both donkeys and oxen. It has designs for both wooden and steel framed types. The wooden types are cheaper in material terms, but the steel framed ones are easier to make because the joints are more straightforward - nevertheless you can make either type of cart in only a few hours, if you are reasonably set up with tools and materials.

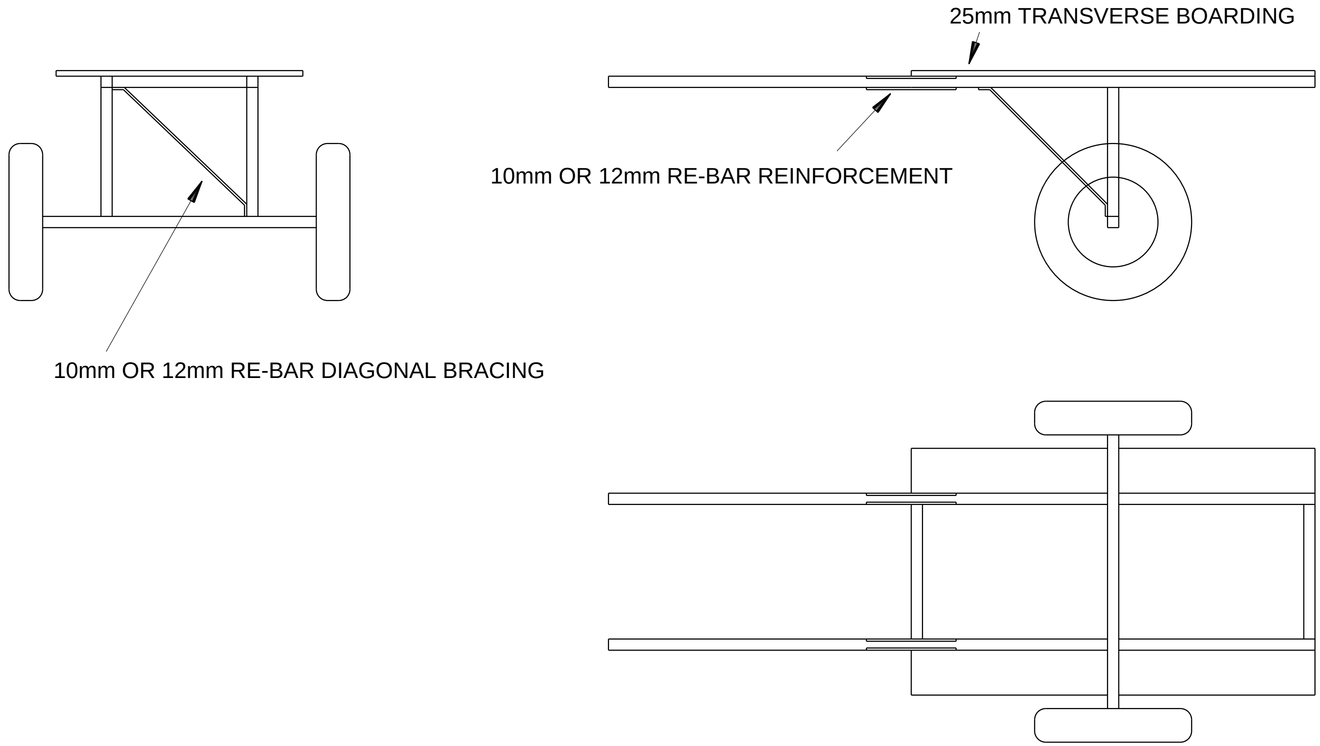
The DTU has also been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. For example it has pioneered a system of hubs using water pipe which do not need machining to make a roller bearing hub. Obviously friction is very low with these hubs and they usually give good milage before being worn out too - we usually get 15 000 km before they are very badly worn, but they may need cleaning and relubrication several times before they get this far. Still they are reasonably cheap - we can make them in Nigeria for about $\$_{US}20$, they only take one man a day to make, and they do not need any special tools.

Other hub designs using, for example aluminium castings, are in production in Nigeria and we are trying to reduce or eliminate the machining in these. Also wheel designs in steel sheet, cast aluminium and timber are in manufacture or under development.

Cart Drawings

Two drawings of carts are shown in the following pages. The first one is simple, but quite strong and easy to make, and is the one for which we have shown the list of materials. The second cart is lighter and even easier to make. You can use this cart with small donkeys.


10mm OR 12mm RE-BAR REINFORCEMENT
25mm LONGITUDINAL BOARDING
850
REMOVABLE STAND
600
10mm OR 12mm RE-BAR DIAGONAL BRACING
3150
1100
AXLE FIXED WITH SINGLE 12Ø BOLT
AND 6Ø RE-BAR WIRE LOOP
STAND BRACKET FROM 10Ø RE-BAR
1800
Scale
200mm
Date
7/9/93
Title
MEDIUM WEIGHT
DONKEY CART
Drawn by
CEO
Dwg No.
1/



Scale	200mm	Title LIGHT WEIGHT DONKEY CART	Drawn by	CEO
Date	13-7-93		Dwg No.	1/

DTU

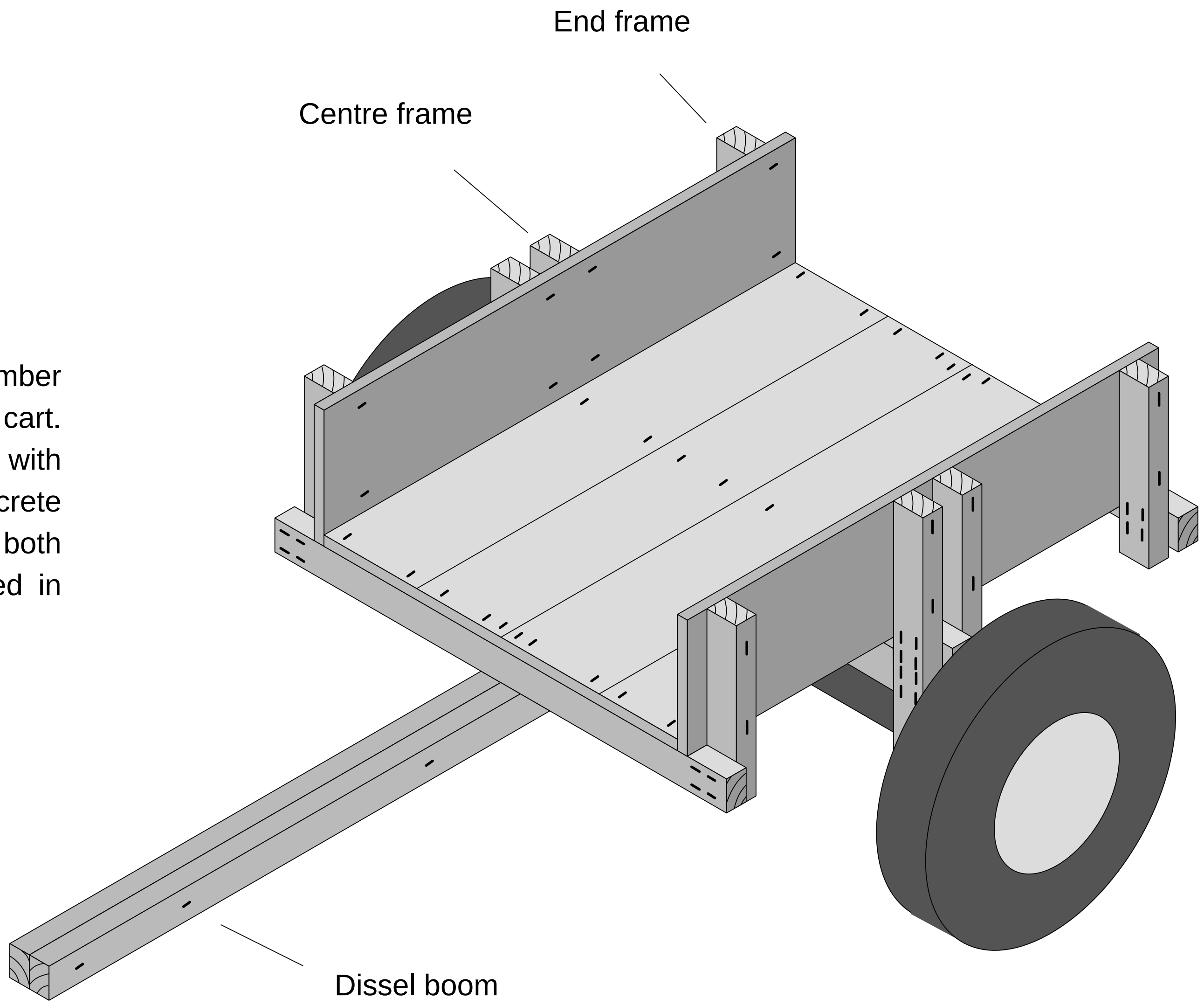
Animal Cart Programme

TECHNICAL
24
RELEASE

SIMPLE LOW-COST WOODEN OX CART

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@uk.ac.warwick.eng

Figure 1: wooden oxcart



This cart uses wooden planks and timber for a quickly made and economical cart. The planks are fixed to the frames with 6mm diameter steel rod (small concrete reinforcing bar) clenched over at both ends a bit like copper nails are used in boat building.

Ox Cart Body Made From Timber

Introduction

Not enough farmers in Africa have animal carts. Those who have carts can take their produce to places where they can get the best prices. They can also get into town and buy fertilizer and better seeds and move things around their farm easier. The trouble is that carts are too expensive for many farmers. The question is what can be done about it?

What you need is a body which carpenters can make quickly with their simple tools. These people will probably be in the small market towns used by the farmers. Experts think that having the cart maker close to the farmer is a good thing because they can talk to each other easily and sort out any problems. And of course if the cart is made locally, it can be repaired locally, so there should not be any problems with spare parts.

Carts are made in many different places. Some carts are made in factories in industrial countries and some are made in factories in Africa, but most are made by local blacksmiths or carpenters using scrap car and Land-Rover axles. These people cannot get enough axles to meet the demand so the carts are expensive. Even if they did have the axles, they still end up building heavy bodies that take ages to make. In another booklet in this series we have told you how you can make simple low-cost axles; in this booklet we tell you about a simple timber body. You should find that you can make the body for about $\$_{US}20$, depending on the cost of the materials and labour. Once you get organised, two men can probably make two bodies per day. This is quite a lot faster than most carts can be made and it follows from the simplifications which we have made to the design. We've designed it to be easy to make.

Idea Behind Design

The idea behind the design of oxcart described in this technical release is to allow construction without lots of special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a woodsaw, a hacksaw or cold chisel, a hammer and a drill able to make a 6mm hole in wood. In fact you can make the drill yourself if you have to. If you need to make a drill read the section **Making a flatbit** below - it's not too difficult, and its quite handy sometimes to be able to make very long drills. You might find that a couple of 4" or a 5" G clamps (or something like it) are useful too. (The symbol " means inches so 4" means about 100mm because there are about 25mm in an inch.)

The way that all the parts of this cart are joined together is like the one that used to be used to fix small boats together. It's called clench nailing and is a bit like riveting. What you do is make a hole right through the wood to be joined, and then put a straight piece of 6mm diameter re-bar (concrete reinforcing bar) right through so it sticks out about 25mm both sides. Then you just knock one end over with a hammer so it lies on the surface of the wood. Next you bend the other end over. Then you put a big hammer or something hard and heavy like a piece of steel against one of the ends and then hit the other with another hammer. What happens is that you tighten the two bits of wood together and you get quite a strong joint. If you put some washers or something like them made from sheet steel on the re-bar before you bend it over it will make the joint a bit stronger still. It does not make a very rigid joint, but you might find that the flexibility gives the cart some resilience so that it takes knocks better.

You will see that there are no mitres and complicated angles to cut in the timber so you save time when making the cart. Also the exact lengths of the components are not very critical - again it saves a little time, but you will find that the carts look better if you take a little trouble to get things square and even etc.

These carts have been tested a bit in Nigeria, but we have not tested them enough. We think that they are strong enough, but we cannot be sure. Really to get a reasonable price you need to experiment a bit to see how the farmers treat their carts and what they expect their carts to stand. It's no good saying it must be strong enough so that they cannot ever break it - somebody will always break anything - and it will very expensive to make it nearly unbreakable. At least you can repair these carts easily and cheaply.

Cutting list and costs

Table 1 shows a cutting list for a complete cart - Recent prices of materials in Nigeria are shown converted into $\$_{US}$.

TABLE 1: cutting list for wooden oxcart.

component	material	number of lengths & length required [No.×mm]	total material in cart [mm]	materials cost in Nigeria [$us]
animal shaft/ boom	75×50 or bush pole	2×3700	7400	2.39
body frame bottoms	75×50 roughsawn timber	3×1100	3300	1.06
body frame sides	75×50 roughsawn timber	6×300	1800	0.58
axle struts	75×50 roughsawn timber	4×600	2400	0.77
tray bottom planks	25×300 or similar timber	4×1800	7200	4.64
tray side planks	25×300 or similar timber	2×1800	3600	2.32
tray ends	25×300 or similar timber	2×1100	2200	1.42
plank fixing staples	6mm dia re-bar or similar	30×250	7500	0.80
body frame staples	6mm dia re-bar or similar	12×150	1800	0.19
axle fixing studs	M12 threaded rod or bolts	2×100	200	0.64
axle fixing loops	6mm dia re-bar or similar	2×200	400	0.04
TOTAL->				14.86

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete. Start by cutting the 75×50 timber into the right lengths, as in the cutting list, and then you can cut the bottom and side planks. Then cut the 6mm dia re-bar for the fixings.

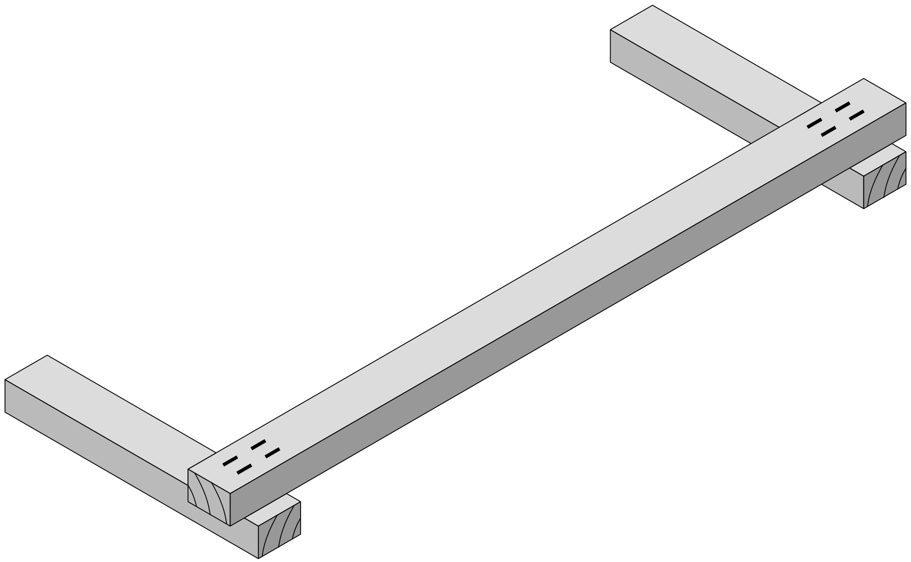


Figure 2: a finished end frame.

2) Next make up the two U-shaped front and back frames (endframes). If you have a G clamp you can use it to hold two pieces of the frame together during drilling and clenching. It's quick and you can tap the bits with a hammer until everything is square and straight and then drill the holes. You might also find that leaving the G clamp on makes it easier to get the re-bar fixings through.

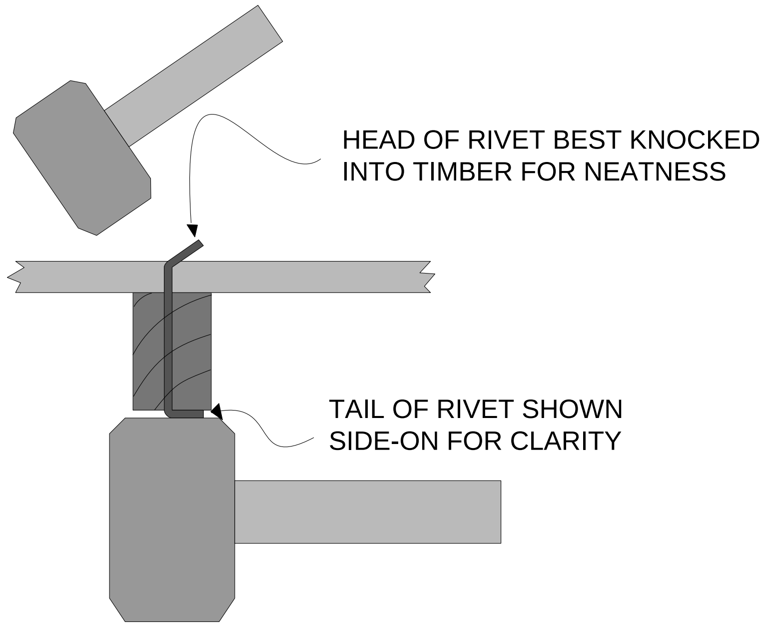


Figure 3: Using two hammers to clench rivet.

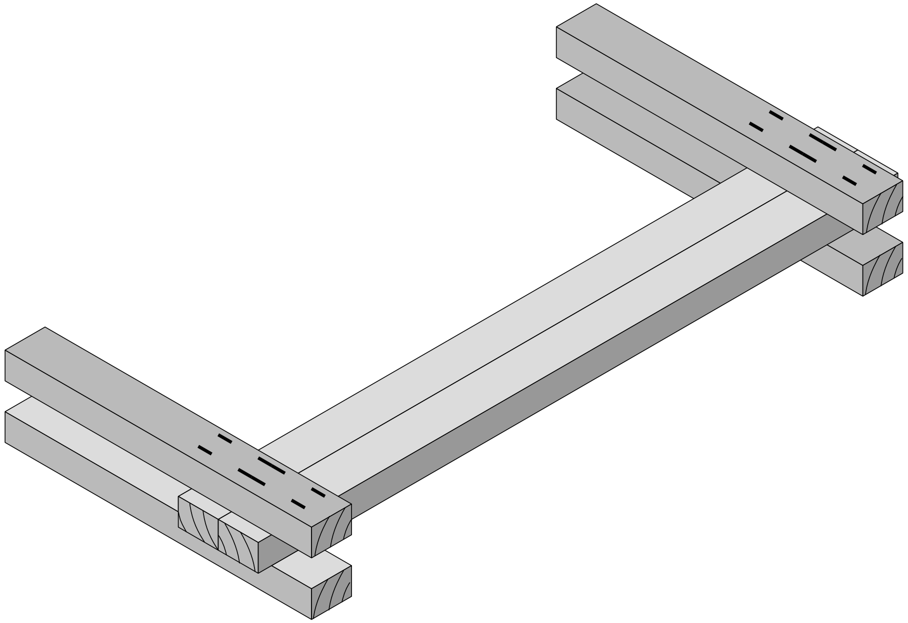


Figure 4: a finished midframe/ axle support frame including spacer.

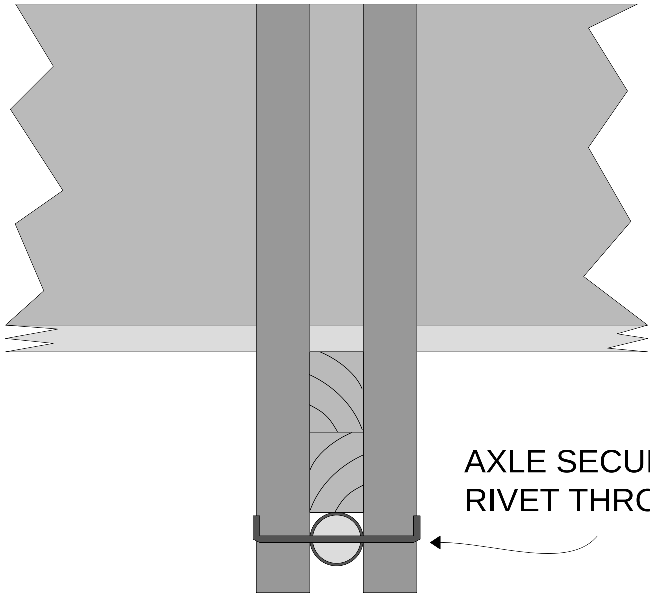


Figure 5: method of fixing axle to axle supports.

3) Then you can make up the frames that go in the middle - the ones that support the axle. You can start with the frame bottom and then fix the uprights to it either side with four rivets. When you've done both ends you will end up with an H shaped assembly.

4) Next you can fit the side and the bottom planks to the end frames and then the middle frame with more rivets.

5) Nearly there! Fix the axle with a rivet right through the middle of it where it goes between uprights of the middle frame - obviously you need to drill through the axle to do this. Another way is to bolt the axle in with a bolt right underneath the axle, but you need a long bolt to do this.

6) If you want to make it so that the ends of the load tray can be removed easily you can do in the way we have shown in

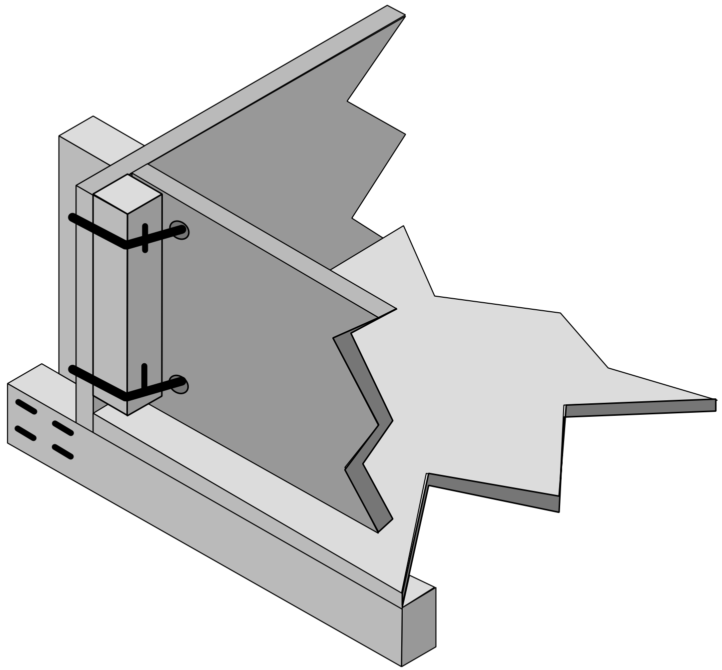


Figure 6: method of fixing tray ends with rubber or rope

7) Paint or creosote the cart. You've finished it!

Modifications

There are many different versions of this cart. You can try longer or shorter carts and you can make them wider or narrower. When you do this, check the length and width of the planks of wood that you will use - you do not want to find that you are two inches short of being able to get two runs of plank out of one piece of timber, or that its just too narrow and you have to fiddle about and fit in a narrow strip.

This cart design does not have ends to the load tray, but if your farmers really want ends you can make them from some more plank. A good way to fix the end planks might be to fix them to the inside of these with bits of truck inner tube as giant bungees or elastic bands. This is an appropriate way because it is cheap and very easily repairable, but the farmers may want some flashy looking thing which will be very expensive to make. You will probably find that things like latches take longer to make than the rest of the cart. Explain to the farmers that they will cost extra too!

Other DTU cart developments

The DTU has been working on a range of cart body types for use with both donkeys and oxen. It has designs for wooden and steel framed types. The wooden types are cheaper in material terms, but the steel framed ones are easier to make because the joints are more straightforward - nevertheless you can make either type of cart in only a few hours, if you are reasonably set up with tools and materials.

The DTU has also been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. For example it has pioneered a system of hubs using steel pipe such as water pipe which do not need machining to make a roller bearing hub. Obviously friction is low with these hubs and they usually give good milage before being worn out too - we usually get 15 000 km before they are very badly worn, but they may need cleaning and relubrication several times before they get this far. Still they are reasonably cheap - we can make them in Nigeria for about $\$_{US}20$, they only take one man a day, and they do not need any special tools.

Other hub designs using, for example aluminium castings, are in production in Nigeria and we are trying to reduce or eliminate the machining in these. Also wheel designs in steel sheet, cast aluminium and timber are in manufacture or under development.

Making a flatbit

Flatbits for drilling wood are easy to make and quite useful because you can make them very long and drill holes in quite awkward places.

To make the bit, get some round steel bar of the same size as the hole you want to make, or a little bit smaller. Then hammer the end to flatten it a little (a bit like a screwdriver). The drawing shows what we mean. In fact if you can harden the cutting edges you can use the flat bit to drill holes in metal too as long as you do not want to drill deep holes. To get hard cutting edges you will need to use either 'silver steel' or say spring steel or even to case harden the edges - though you will lose the hardness as soon as you re-sharpen the drill if it's just case hardened.

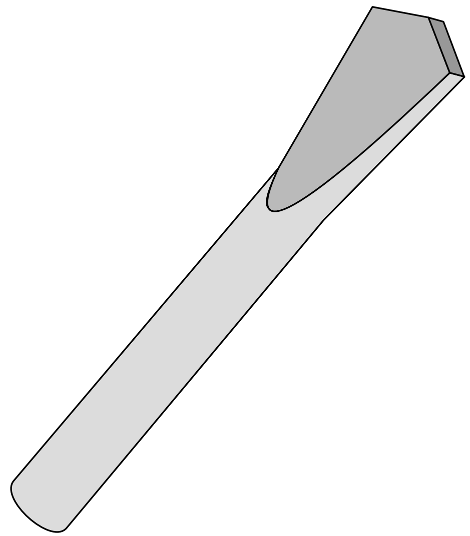


Figure 7: flat bit for drilling holes

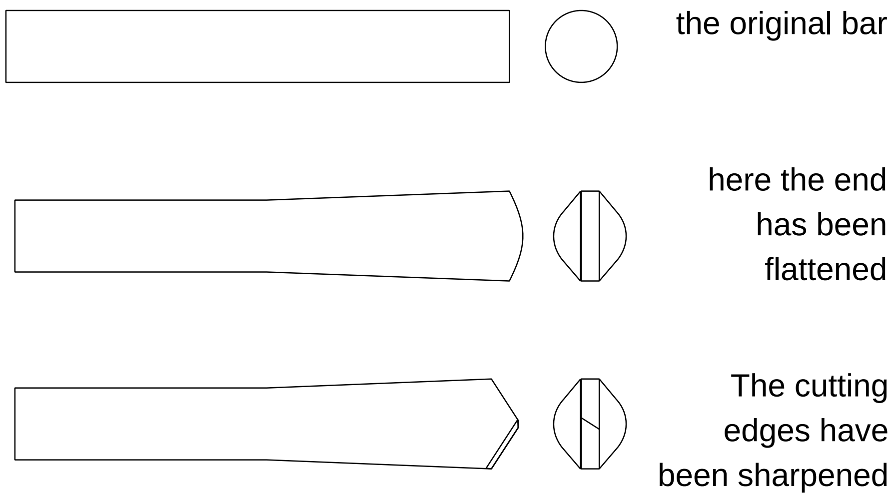
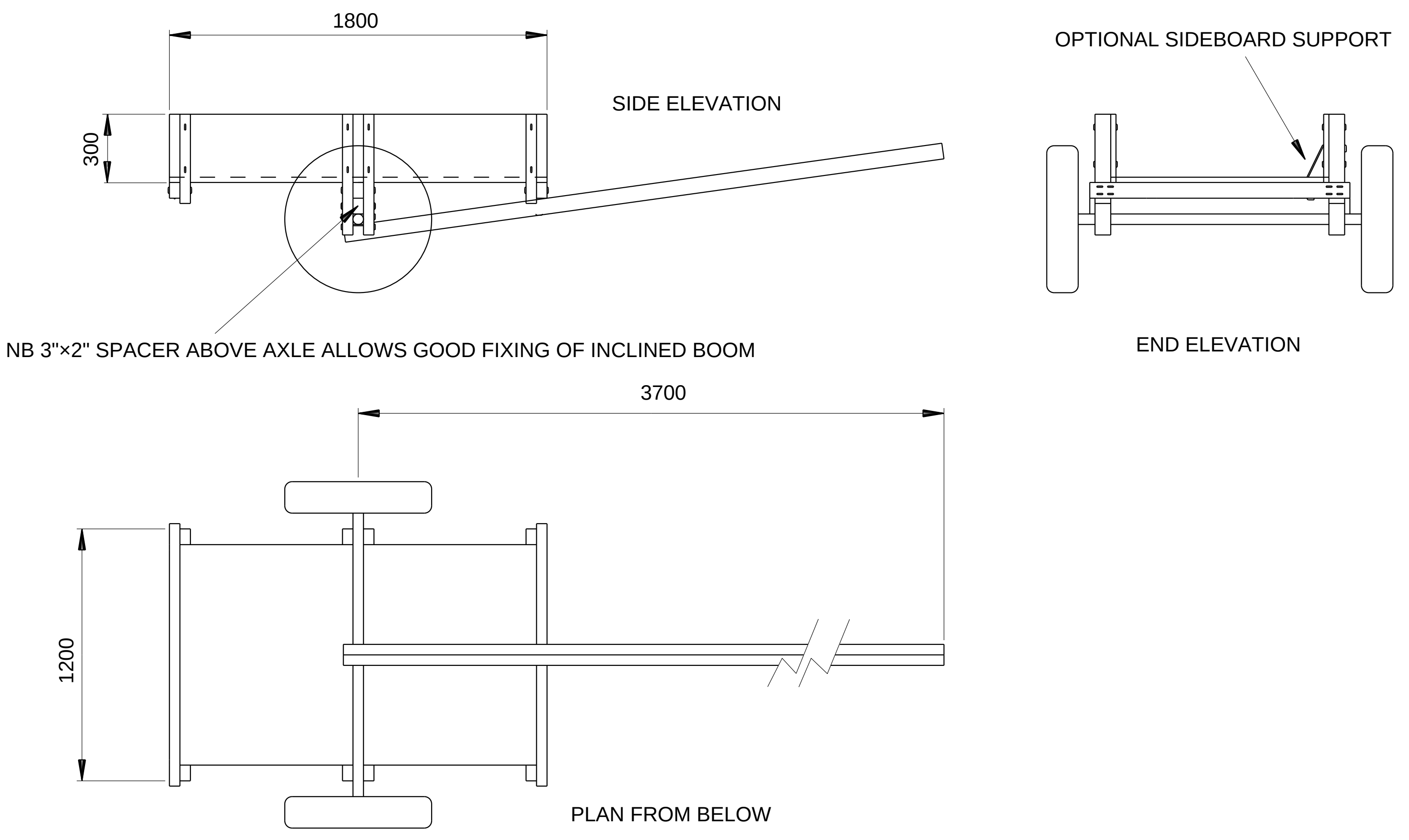


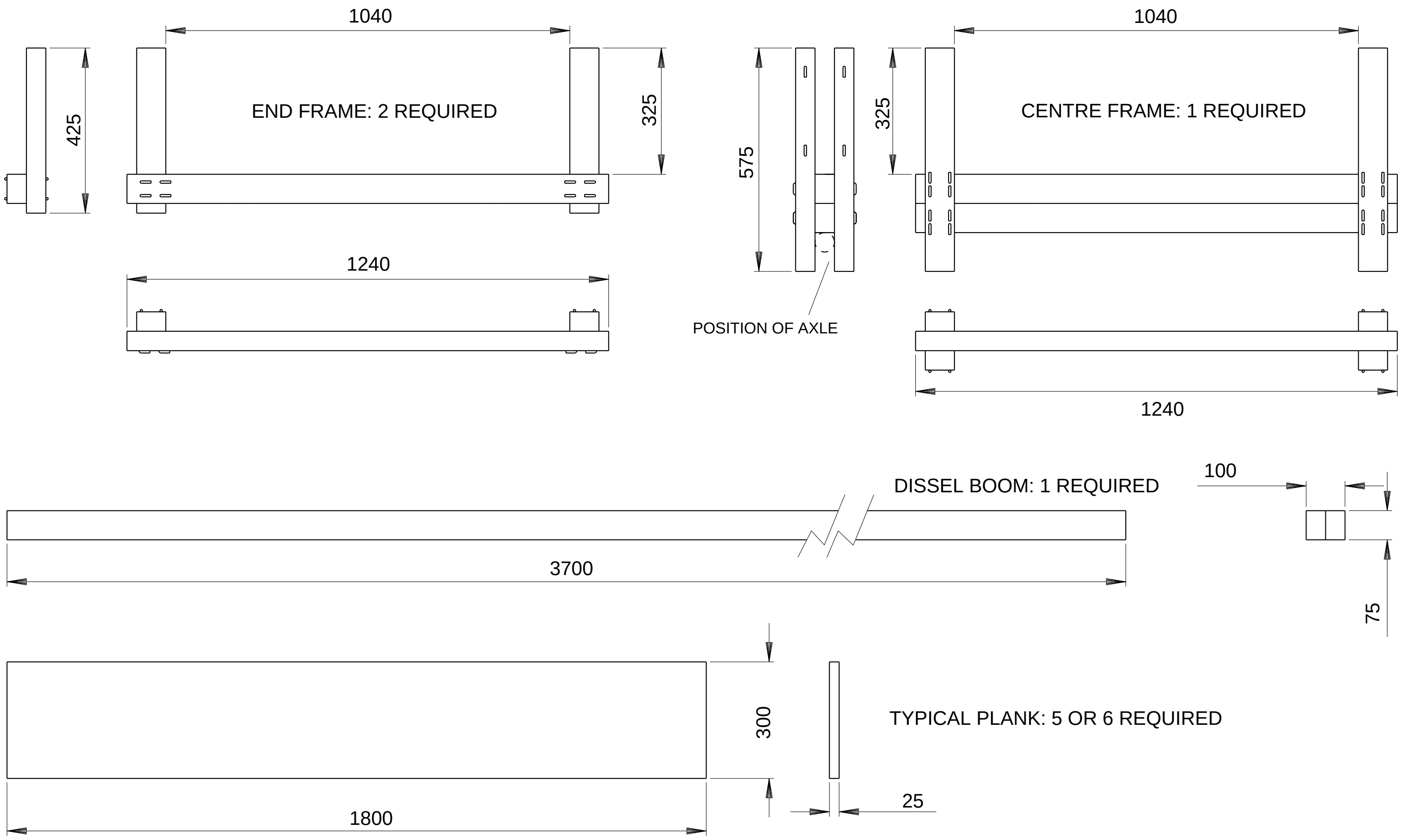
Figure 8: stages in the making of a flat bit for drilling holes.

Cart Drawings

You will find two drawings below, the first one gives a general view of the cart and the second, a view of the main components. As we have said you can vary the size of the cart quite a bit and even make it much longer if you add extra frames. You could even make a four wheeled cart like this!



Scale	200mm	Title LIGHTWEIGHT OX CART FROM WOOD	Drawn by	CEO
Date	6/10/92		Dwg No.	1/2



					Scale	100mm	Title OX CART COMPONENTS	Drawn by	CEO
					Date	16/7/94		Dwg No.	2/2

DTU

Animal Cart Programme

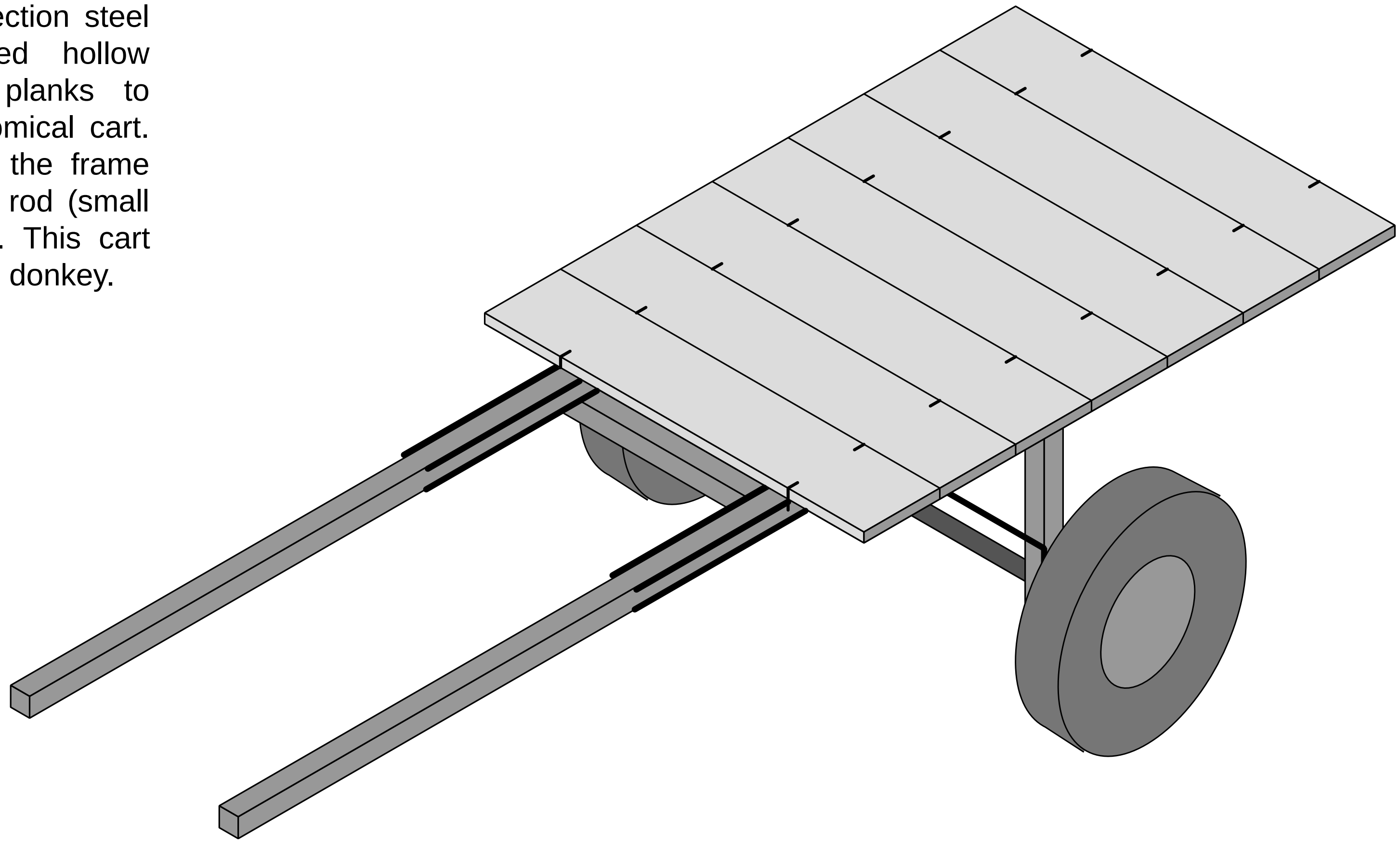
TECHNICAL
25
RELEASE

LIGHT STEEL AND WOOD DONKEY CART

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk

Figure 1: donkey cart made from square box tubing and wooden planking.

This cart uses square section steel box tubing (RHS rolled hollow section) plus wooden planks to make a quick and economical cart. The planks are fixed to the frame with 6mm diameter steel rod (small concrete reinforcing bar). This cart has two shafts for a single donkey.



TECHNICAL RELEASE 25

Lightweight Donkey Cart Body Made From Square Box Tubing and Timber Planks

Introduction

Not enough farmers in developing countries have animal carts. Those who have carts can take their produce to places where they can get good prices. They can also get into town and buy fertilizer and better seeds and move things around their farm easier. The trouble is that carts are too expensive for many farmers. What can be done?

Carts are made in many different places. Some carts are made in factories in industrial countries and some are made in African factories, but most are made by local blacksmiths or carpenters using scrap car and Land-Rover axles. These people cannot get enough axles to meet the demand, so the price is high. Even if they did have the axle, they still end up building heavy bodies that take ages to make. In another booklet in this series we have told you how you can make simple low-cost axles; in this booklet we tell you about a simple steel and timber body. You should be able to make the body for about $\$_{US}$ 30 depending on the cost of the materials and labour. Once you get organised, two men can probably make two bodies per day.

What you need is a cart body which carpenters and fabricators can make with their simple tools. These people will probably be in the small market towns used by the farmers and they will have an electric welder and some basic handtools like a hacksaw. Experts think that having the cart maker close to the farmer is a good thing because they can talk to each other easily and sort out any problems. Also if the cart is made locally, it can be repaired locally, so there should not be any problems with spare parts.

Idea Behind Design

The idea behind the design of donkey cart described in this technical release is to allow construction without lots of special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a welder and a hacksaw. You might also find that a couple of 4" or a 5" G clamps (or something like it) are useful too. (The symbol " means inches here, so that 4" is about 100mm since there are about 25mm in an inch.) A wheelbrace or carpenters brace is also useful - you can make the drill bit yourself.

You will see that there are no mitres and unusual angles to cut in the square tubing so you save time when making the cart. Also the exact lengths of the components are not very fussy. But you will find that the carts look better if you take trouble to get things square and straight.

These carts have been tested in Nigeria, but we would like to test them more. The only problems which we have found so far have been breaking of the animal shafts and we have fixed this by welding some strengtheners (bits of 8mm, 10mm or 12mm round bar or re-bar) to the top and bottom of the chassis. (Re-bar means concrete reinforcing bar). The construction tends to be a bit light in Nigeria, because they sell square tubing with very thin walls (much less than 1mm thickness). If you used tubing with a wall thickness of 2.5mm or more you probably would not need to put these strengtheners on. But really to get a cart at a reasonable cost you need to experiment a bit to see how the farmers treat their carts and what they expect them to stand. It's no good saying it must be strong enough so that they cannot ever break it - somebody will always break anything - and it will be very expensive to make it nearly unbreakable. At least with this design you can repair it easily and cheaply.

Cutting list and costs

Table 1 shows a cutting list for a complete cart. Recent prices of materials in Nigeria are shown converted to $\$_{US}$. The square box tube (sometimes called rolled hollow section or RHS) is about 2" or 50mm on one side. You can often buy it in a variety of wall thicknesses. It's best with a wall thickness of 2.5mm (12 gauge) or more, but we have used wall thicknesses down to 1.6mm (16 gauge) as we mentioned earlier.

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete. Start by cutting the square box tube into the right lengths, as in the cutting list shown in Table 1. Then cut the various bits of re-bar or whatever you are going to use. You could cut the timber also at this stage, but it might be easiest to leave it till last.

TABLE 1: cutting list for light steel framed donkey cart.

component	material	# lengths & length reqd	total material in cart	materials cost in Nigeria
[#*mm]				
animal shafts	50×50 RHS	2×3150	6300	8.94
body cross pieces	50×50 RHS	3×700	2100	2.98
axle struts	50×50 RHS	2×600	2400	3.40
shaft strengtheners	8mm to 12mm round bar[1]	8×600	4800	1.02
axle strut braces	8mm to 12mm round bar	3×850	2550	0.54
axle fixing studs	M12 threaded rod or bolts	2×125	250	0.85
axle fixing loops	6mm dia re-bar or similar	2×330	660	0.07
plank fixing studs	6mm dia re-bar or similar	14×75	1050	0.11
tray planks	1"×12" or similar timber	6×1100	6600	3.90
TOTAL =				21.82

[1] The round bar can be anything actually - it doesn't even have to be round, so deformed or high-yield re-bar is fine. You could even use flat strip as long as its more than say 8mm thick.

2) Next weld the axle supports to the animal shafts. The method shown in Figure 2 is probably the easiest way to support the components during welding the first shaft and support. It's quick and if you do not tighten the clamps too

tight to start with, you can tap the parts with a hammer until everything is square and straight. Then tighten the clamps before you weld.

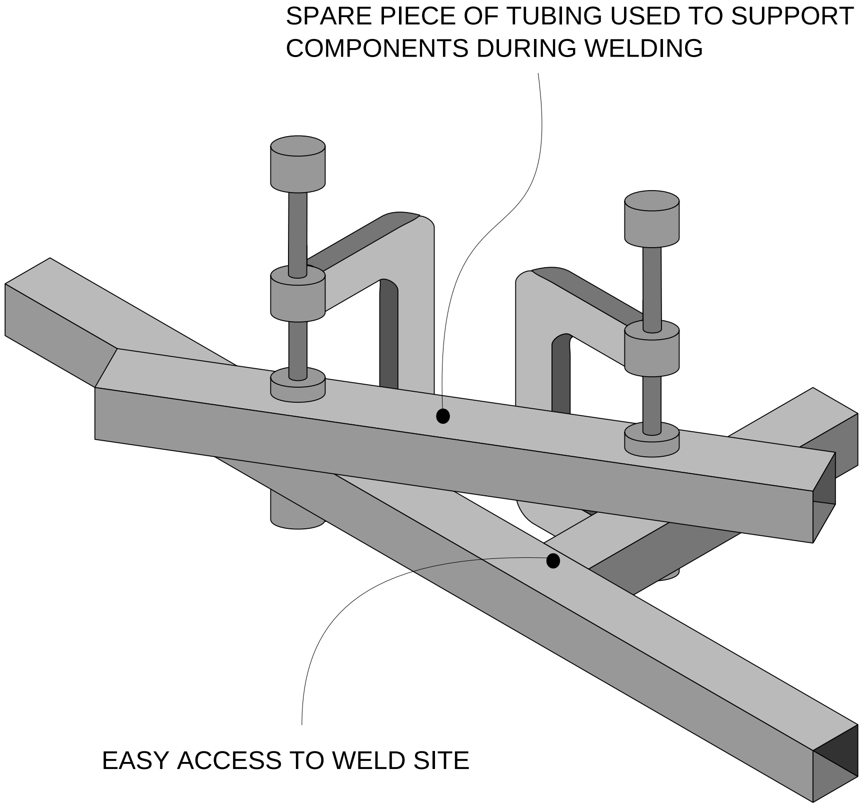


Figure 2: supporting components during welding.

Repeat the process using the first shaft and support as a pattern for the second as shown in Figure 3. Make sure you do not weld the two assemblies together!

3) It may be easiest to fix the axle retaining bolts and loops on at this stage so that you can use them in the next stage. Figure 4 shows a good way to fix a square axle on.

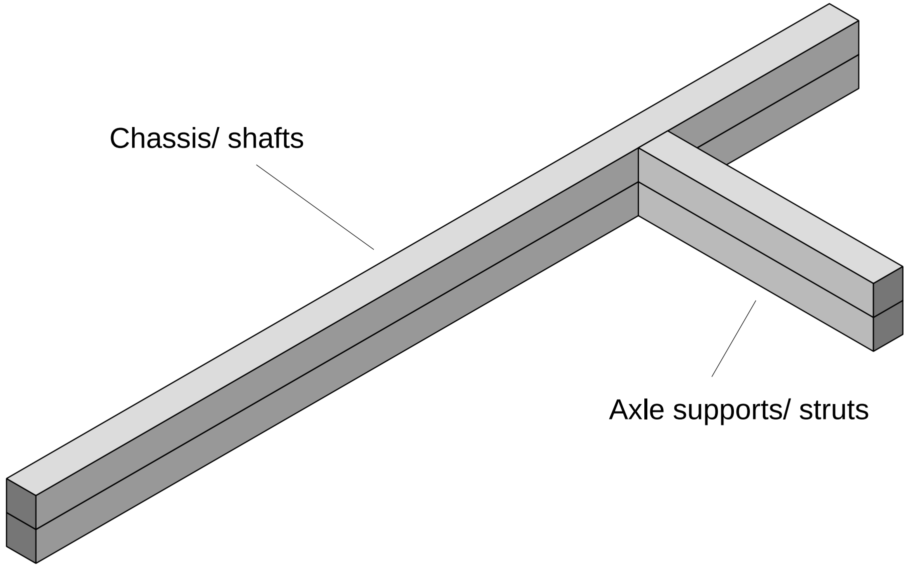


Figure 3: welding animal shaft and axle support strut assemblies.

4) Now you can weld the three cross pieces under the animal shafts after you have checked for squareness etc.

5) Next you need to weld on the various reinforcements to the square tube. These are the axle braces and the pieces where the shafts might get broken - over the axle supports and at the front of the load tray/ wooden planking. Figure 5

shows the frame nearly ready for fixing the planking.

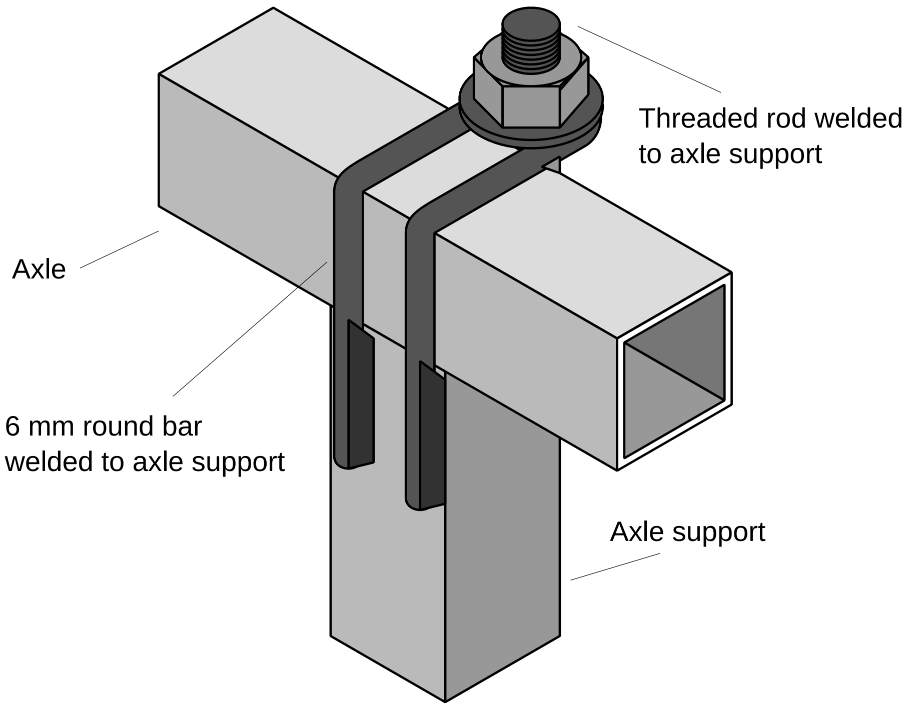


Figure 4: method of fixing axle to axle supports.

6) Nearly there! Now you need to fix the wood onto the square tube. You can do this with staples or studs. Staples use more steel but studs need more welding. Figure 6 shows the choices - staples are like big U s, and studs are just pieces of 6 mm wire sticking up which you bend over the wood.

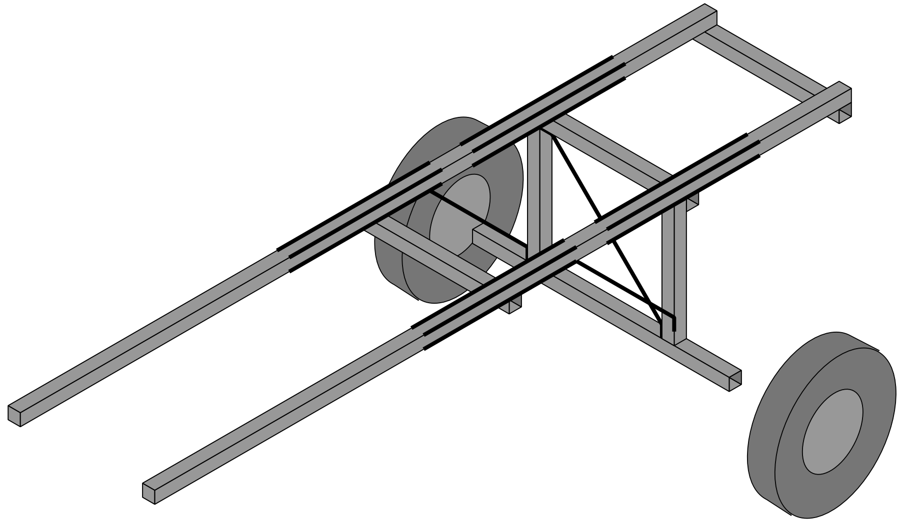


Figure 5: cart frame before planking.

To make staples you can make a jig from some bolts in a piece of wood or you can just bend the staples in a vice. They do not have to be very accurate, but as usual the more accurate the better.

To fit the staples, drill a hole in the plank both sides of the tubing and push both legs of the U through the wood using a hammer if necessary. When it's through, hold another hammer or something heavy against the bottom of the U whilst you knock the protruding legs over with another hammer. You might find this easier with someone to help you. Then clench the legs by knocking them into the surface of the wood to leave the surface flush. Once you are happy that all is in the right place, weld the staple to the square tubing as shown in Figure 6. Then put the rest of the staples in in the same way.

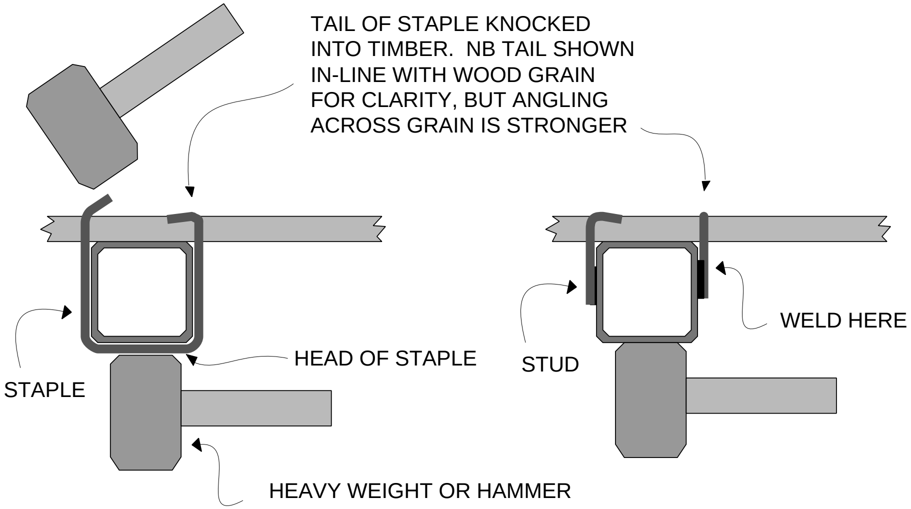


Figure 6: fixing wood with staples or studs.

If you want to use studs, weld them on to the side of the square tube as shown in Figure 6. You can either drill holes for them as described earlier for staples, or you can bring them up past the edge of the planks, which saves drilling holes.

7) Paint the cart. You've finished it!

Modifications

There are many different versions of this cart. The one shown in this document has no sides, because it is for one donkey and is as light as possible. It is good for everything except loose materials such as sand, but you can always put this into sacks. To fix ropes for tying firewood or sorghum stoba etc, you can cut slots or notches in the ends of the planks as shown in Figure 7, which shows two ways of doing it.

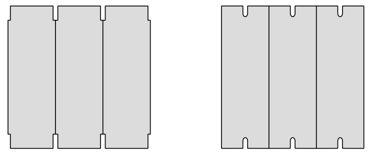


Figure 7: notched ends in planks to tie rope to.

You could put side planks on a cart like this using the same 6mm wire/ round bar/ re-bar to tie them on. This would be cheap and very easily repairable.

A danger with building carts is that the farmers will want something flashy looking which will be very expensive to make. You would probably find that things like hinges and latches take as much time to make as the basic parts of the cart.

If you want a cart with permanent sides you could build our medium weight donkey cart.

You can try longer and shorter carts and you can make them wider or narrower. When you do this, check the length and width of the planks of wood that you will use - you do not want to find that you are two inches short of being able to get two runs of plank out of one piece of timber, or that its just too narrow and you have to fiddle about and waste time filling the gap with an extra little strip.

Another thing is the height of the cart. No-one seems to know why some carts are made high and others low. Its better for the animals if the load tray is low particularly if the carts will be operated over rough ground. But you may find that farmers want a high load tray to keep loads dry if they are fording rivers a lot, or it may be a status thing (if you are wealthy enough to own a cart, your eye level must be above a pedestrian's). Or it may be that it's too tiring to load a low cart - if, for example, you have to bend your back twice for each bundle - once to pick it up off the ground and again to put it down onto a low load tray. We have also found that farmers usually want the body to come out over the wheels so they can load on lots of straw or light materials.

Other DTU cart developments

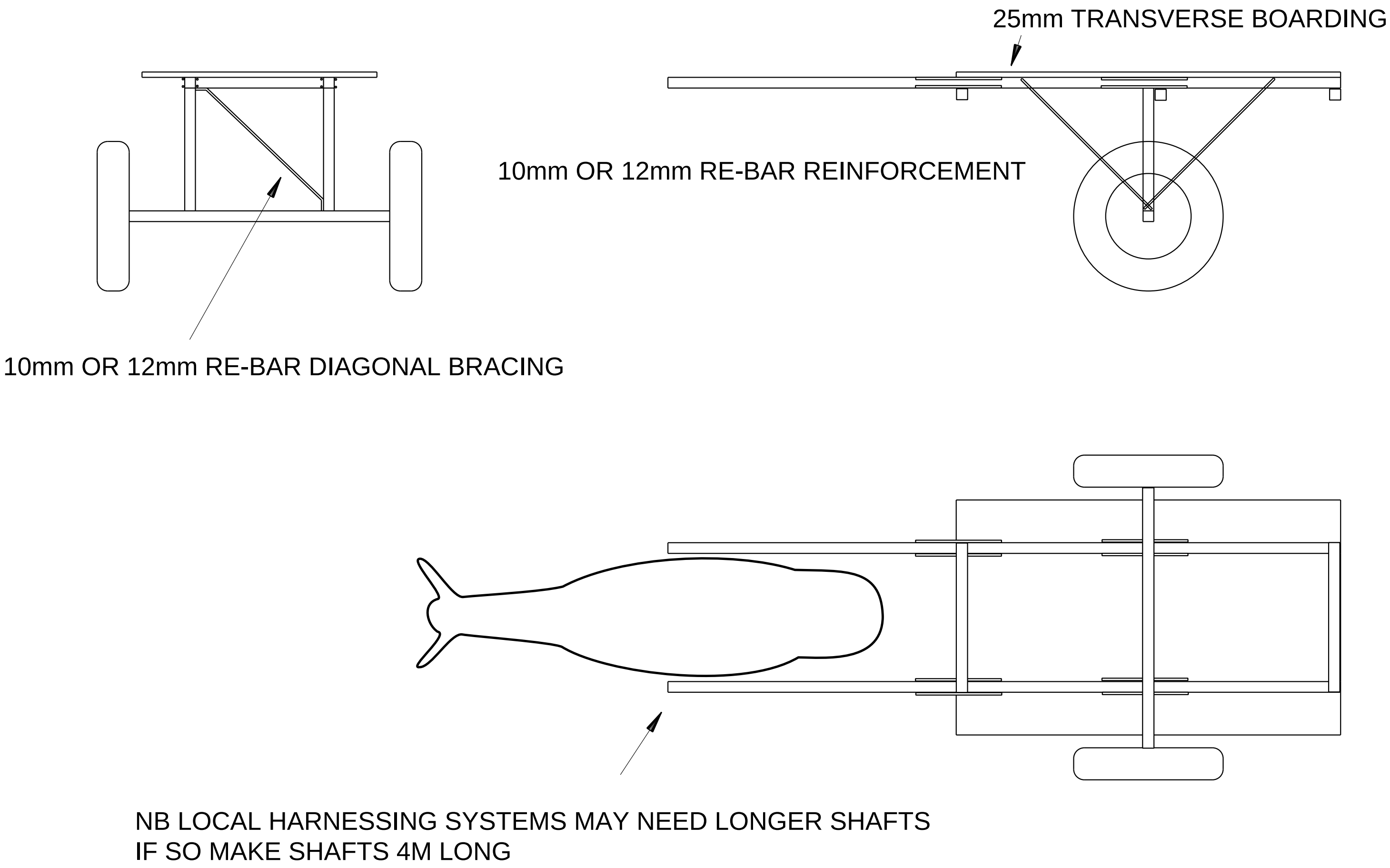
The DTU has been working on a range of cart body types for use with both donkeys and oxen. It has designs for both wooden and steel framed types. The wooden types are cheaper in material terms, but the steel framed ones are easier to make because the joints are more straightforward - but you can make either type of cart in only a few hours, if you have all the tools and materials you need before you start.

The DTU has also been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. For example it has worked on a system of hubs using water pipe which do not need machining to make a roller bearing hub. Friction is low with these hubs and they usually give good milage before being worn out too - we usually get 15 000 km before they are very badly worn, but they may need cleaning and relubrication several times before they get this far. But they are reasonably cheap - we can make them in Nigeria for about $\$_{US}$20. They only take one man a day to make and they do not need any special tools.

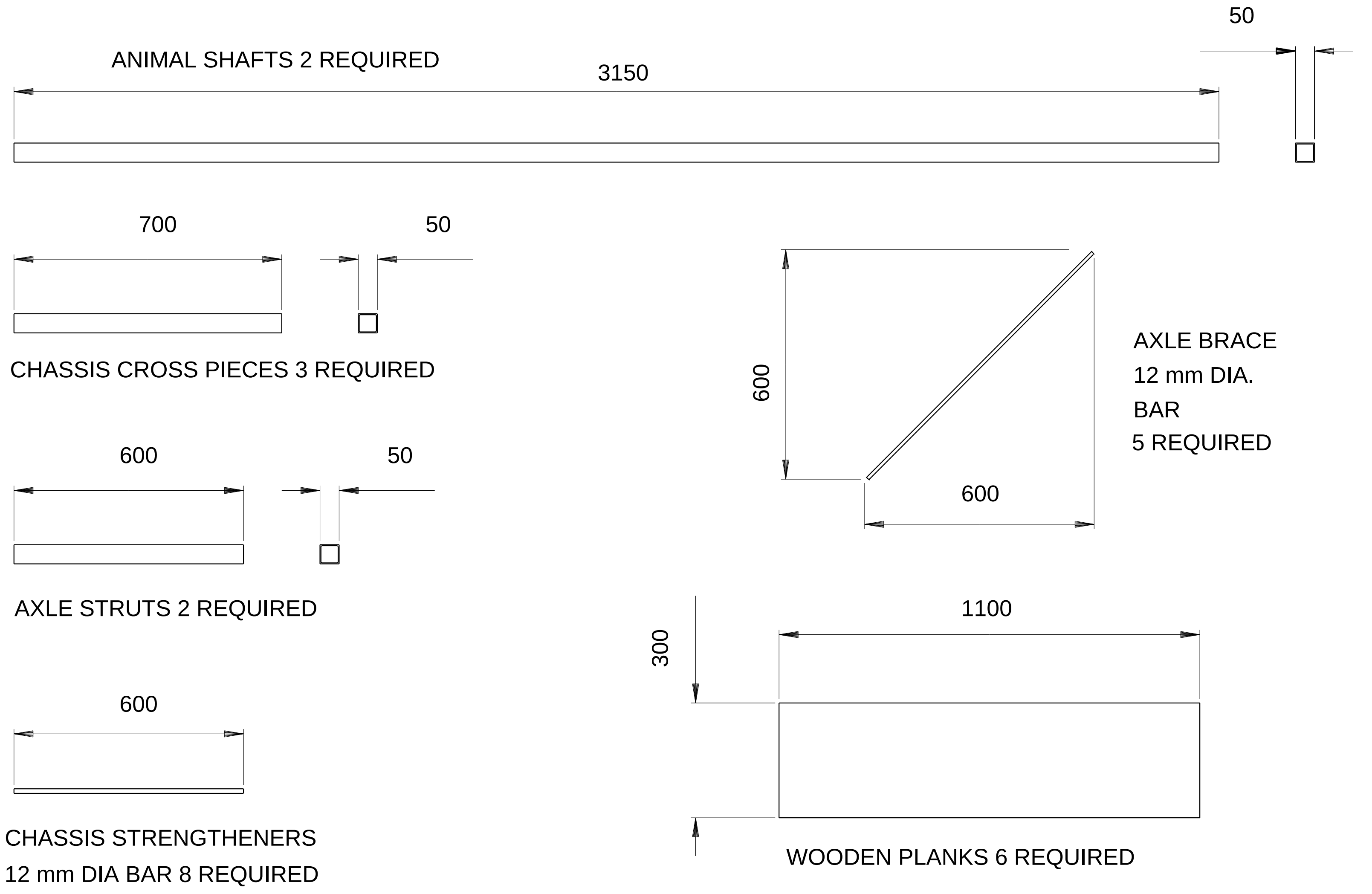
Other hub designs using, for example aluminium castings, are in production in Nigeria and we are trying to reduce or eliminate the machining in these. Also wheel designs in steel sheet, cast aluminium and timber are in manufacture or under development.

Cart Drawings

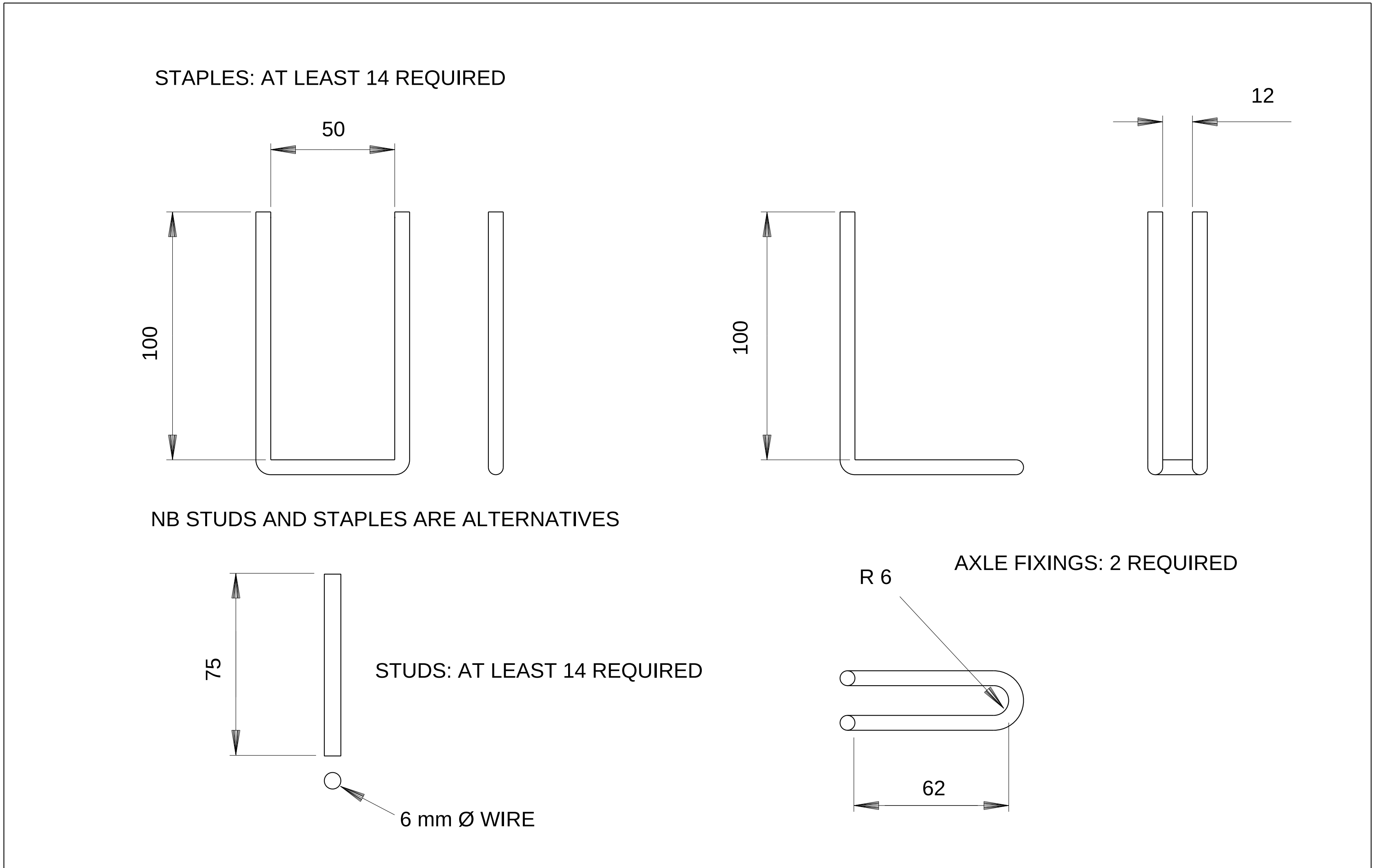
The drawing for the cart is shown on the following page and the list of materials has been shown on a previous page.



Scale	200mm	Title: LIGHT WEIGHT DONKEY CART	Drawn by	CEO
Date	13-7-93		Dwg No.	1/3



					Scale	100mm	Title CART COMPONENTS	Drawn by	CEO
					Date	13-7-93		Dwg No.	2/3



Scale	10mm	Title CART COMPONENTS	Drawn by	CEO
Date	13-7-93		Dwg No.	3/3

DTU

Animal Cart Programme

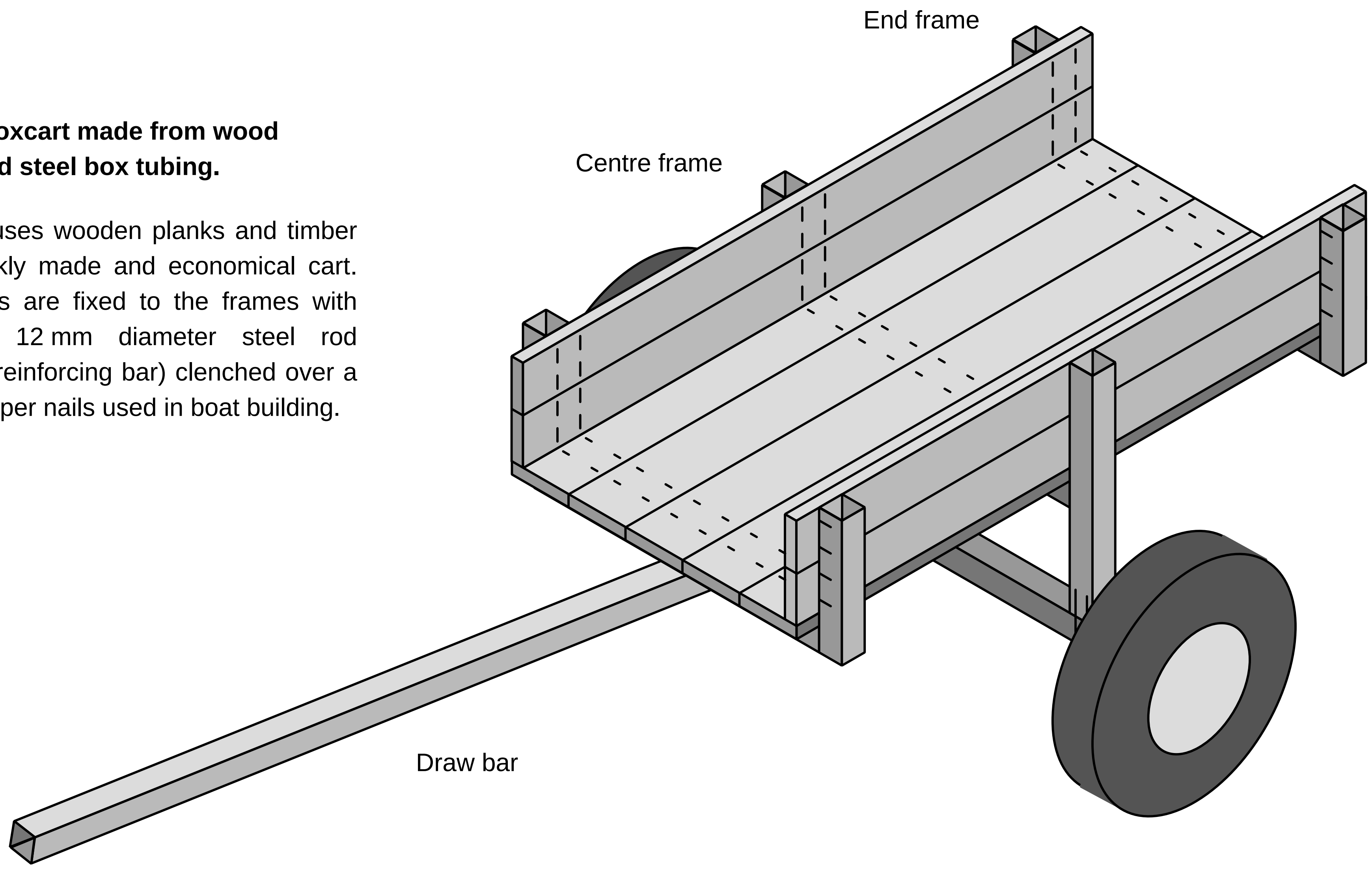
TECHNICAL
26
RELEASE

LOW-COST STEEL & WOOD OX CART

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)1203 523523 extn 2339, fax: +44 (0)1203 418922, email: dtu@eng.eng.warwick.ac.uk

Figure 1: oxcart made from wood planks and steel box tubing.

This cart uses wooden planks and timber for a quickly made and economical cart. The planks are fixed to the frames with 6 mm to 12 mm diameter steel rod (concrete reinforcing bar) clenched over a bit like copper nails used in boat building.



Ox Cart Body Made From Steel Box Tubing and Timber

Introduction

Not enough farmers in Africa have animal carts. Those who have carts can take their produce to places where they can get the best prices. They can also get into town and buy fertilizer and better seeds and move things around their farm easier. The trouble is that carts are too expensive for many farmers. The question is what can be done about it?

What you need is a body which fabricators and carpenters can make quickly with simple tools. These cart makers will probably be in the small market towns used by the farmers. Experts think that having the cart maker close to the farmer is a good thing because they can talk to each other easily and sort out any problems. And of course if the cart is made locally, it can be repaired locally, so there should not be any problems with spare parts.

Carts are made in many different places. Some carts are made in factories in industrial countries and some are made in factories in Africa, but most are made by local blacksmiths or carpenters using scrap car and Land-Rover axles. These people cannot get enough axles to meet the demand so the carts are expensive. Even if they do have the axles, they still end up building heavy bodies that take ages to make. In another booklet in this series we have told you how you can make simple low-cost axles; in this booklet we tell you about a simple steel and timber body. You should find that you can make the body for about $\$_{US}60$, depending on the cost of the materials and labour. Once you get organised, two men can probably make one body in a day. This is quite a lot faster than most carts can be made and it follows from the simplifications which we have made to the design. We've designed these carts to be easy to make.

Idea Behind Design

The idea behind the design of oxcart described in this technical release is to allow construction without lots of special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a simple welder, a woodsaw, a hacksaw, a hammer and a drill able to make a 10 mm or 12 mm hole in wood. (In fact you can make the drillbit yourself if you have to - read our booklet **Making a flatbit** - it's not too difficult.) You might find that a couple of 4" or a 5" G clamps (or something like it) are useful too. (The symbol " means inches so 4" means about 100mm because there are about 25mm in an inch.)

The cart frames are fixed together by welding and the wooden planks are fixed to the frames with clenched steel bar. This is a bit like the way small boats used to be fixed together. It's called clenching. What you do is make a holes through the wood to be fixed, put a giant staple made from 6 mm, 8 mm or 10 mm diameter re-bar (concrete reinforcing bar) over the steel box tubing and through the wood so it sticks out about 50 mm, and then knock the ends over with a hammer so they lie on the surface of the wood. You can tighten the joint more by putting a big hammer or something hard and heavy under the staple as shown in Figure 5 and then hitting the ends harder. If you put some washers (or something like them made from sheet steel) on the re-bar before you bend it over it will make the joint a bit stronger still. It does not make a very rigid joint, but you will find that the flexibility gives the cart some resilience so that it takes knocks better.

You will see that there are no mitres or complicated angles or joints to cut so you save time when making the cart. Also the exact lengths of the components are not very critical - again it saves a little time, but you will find that the carts look better if you take a little trouble to get things square and even etc and welding is easier with good square ends.

These carts have been tested a bit in Nigeria, but we have not tested them enough. We think that they are strong enough, but we cannot be sure. Really to get a reasonable price you need to experiment a bit to see how the farmers treat their carts and what they expect their carts to stand. It's no good saying it must be strong enough so that they cannot ever break it - somebody will always break anything. It is very expensive to make something unbreakable. At least you can repair these carts easily and cheaply.

Cutting list and costs

Table 1 shows a cutting list for a complete cart - Recent prices of materials in Nigeria are shown converted into $\$_{US}$.

TABLE 1: cutting list for steel and wood oxcart.

component	material	number of lengths & length required [No.×mm]	total material in cart [mm]	materials cost in Nigeria [$us]
animal draw bar	80×80 box tubing	1 × 3 600	3 600	12.77
frame bottoms	80×80 box tubing	3 × 1 000	3 000	10.64
end frame sides	80×80 box tubing	4 × 415	1 660	5.89
centre frame sides	80×80 box tubing	2 × 750	1 500	5.32
tray bottom planks	30 × 150 or similar timber	6 × 2 000	12 000	3.87
tray side planks	30 × 150 or similar timber	4 × 2 000	8 000	2.58
tray ends	30 × 150 or similar timber	2 × 1 000	4 000	1.29
plank fixing staples	8 mm dia re-bar or similar	60 × 400	24 000	2.55
axle fixing studs	M12 threaded rod or bolts	4 × 150	600	2.55
axle fixing loops	8 mm dia re-bar or similar	4 × 550	2 200	0.23
TOTAL->				47.69

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete. Start by cutting the 80 × 80 box section steel into the right lengths, as in the cutting list, then cut the bottom and side planks. Lastly cut the 6 mm or 10 mm dia or whatever re-bar for the fixings (the staples etc).

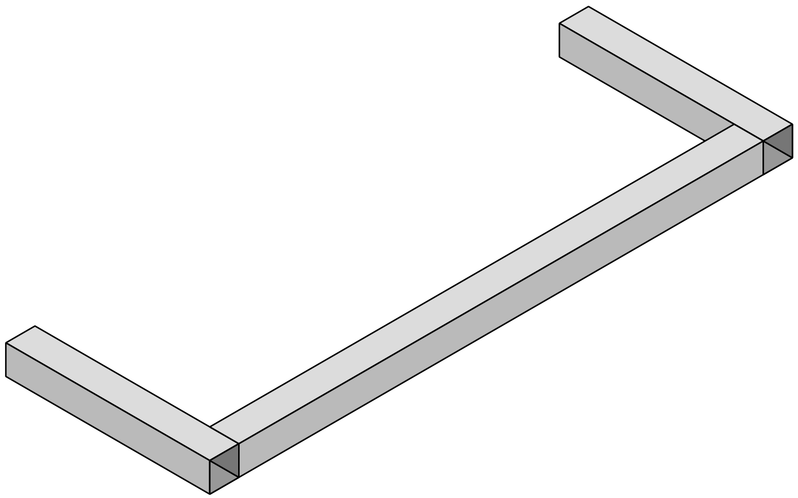


Figure 2: finished end frame.

2) Next make up the two U-shaped front and back frames (endframes). If you have a couple of G clamps you can use them to hold two pieces of the frame together during welding as shown in Figure 3. It's quick and you can tap the parts with a hammer until everything is square and straight and then weld.

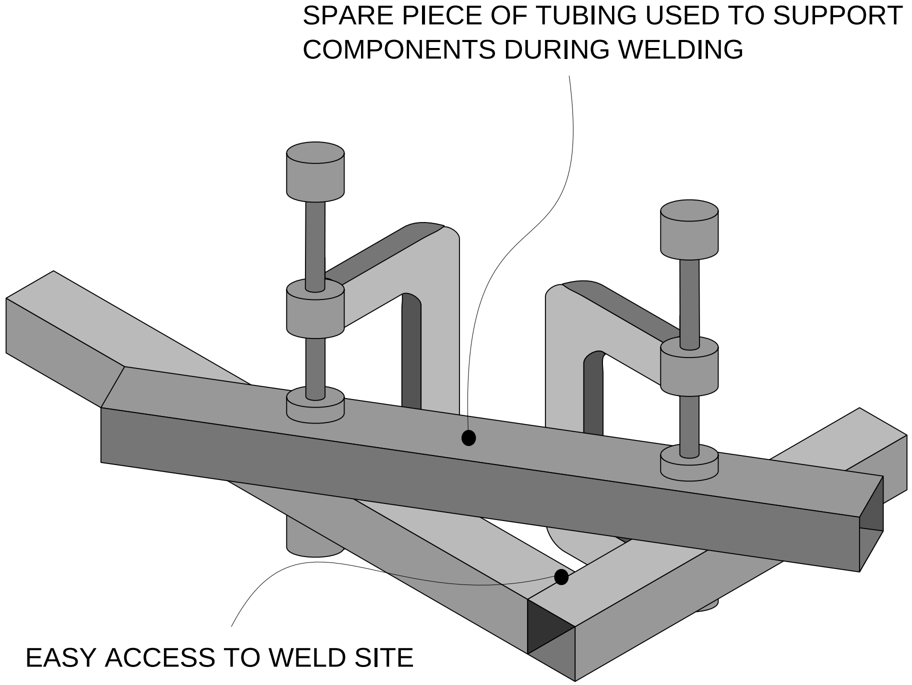


Figure 3: holding frame components during welding.

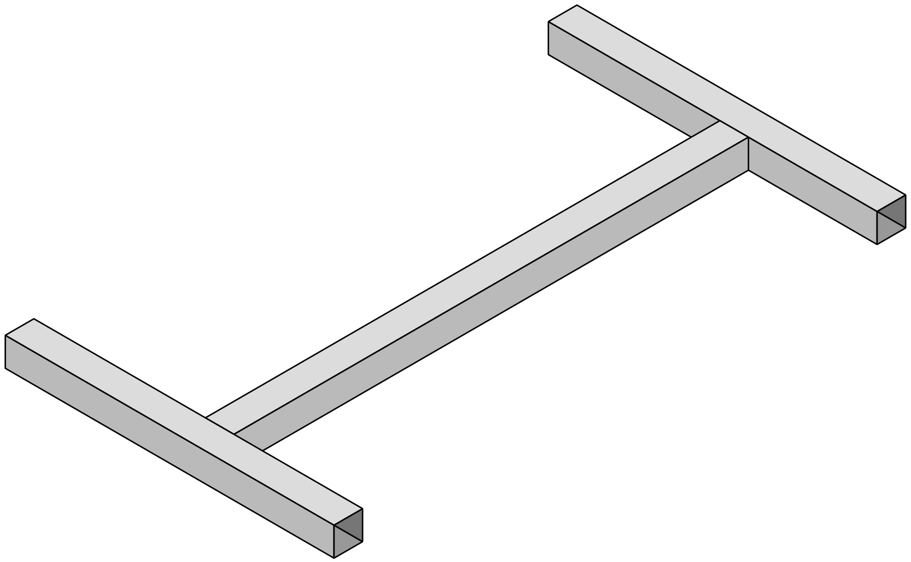


Figure 4: a finished centre frame.

3) Then make up the frame that goes in the middle (as shown in Figure 4) - the one which supports the axle.

4) Next you can fit the side and the bottom planks to the end frames and then the middle frame with staples or studs. Figure 5 shows how these staples and studs can be tightened with a hammer and a weight or another hammer. You need to fix the staples in the right place with a small weld. Studs are another way of fixing the planks. Studs are just short lengths of round bar welded to the sides of the box section as shown in the right of Figure 5. This saves round bar but means more welding.

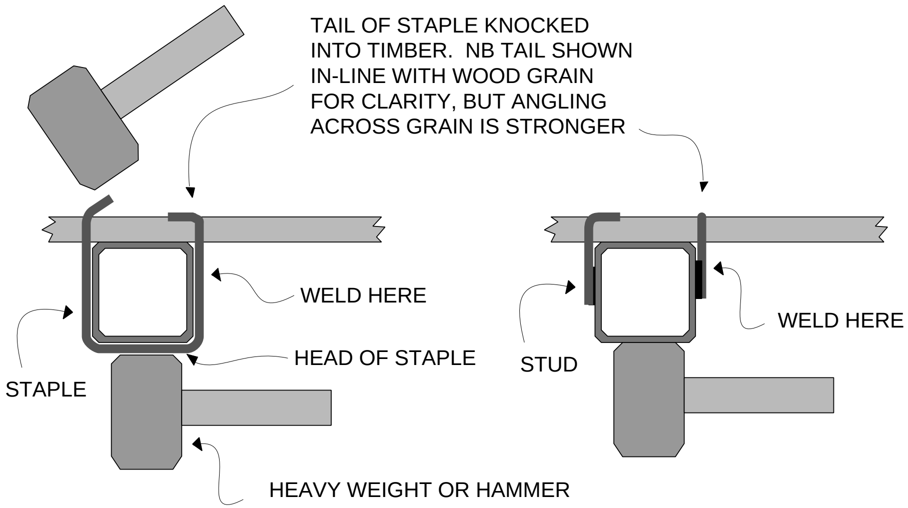


Figure 5: tightening staple or welded stud.

When you bend the end of the stud or staple over you can either bend it in line with the grain of the wood or across the grain, as shown in Figure 6. Bending it in line as shown on the left lets it go into the wood nicely and looks neat, but bending it over across the grain gives a stronger joint.

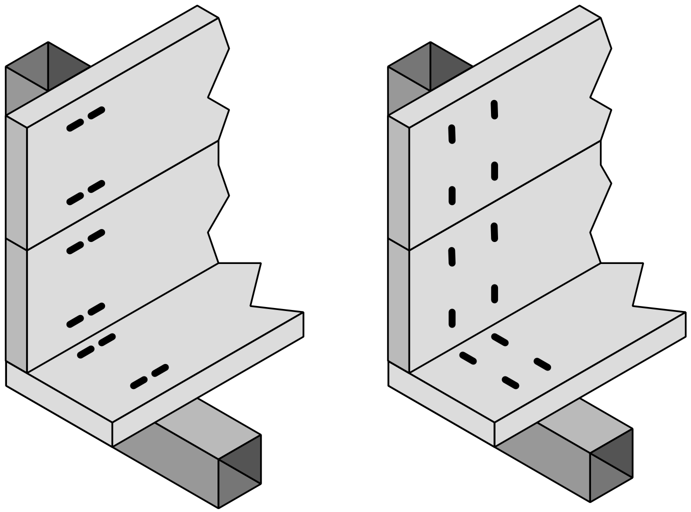


Figure 6: studs or staples bent in line with grain (left) or across it (right).

One thing we have not tried is bending the staple or fixing over the edge of a plank instead of putting it through a hole.

5) Next fix the axle with a one or two loops of round bar and some threaded rod and nuts and washers as shown in Figure 7. If you can get big threaded rod (say 16 mm and some 10 mm plain rod, you can fix the axle with one loop and threaded rod, but if you have only 12 mm then you must use two loops as shown.

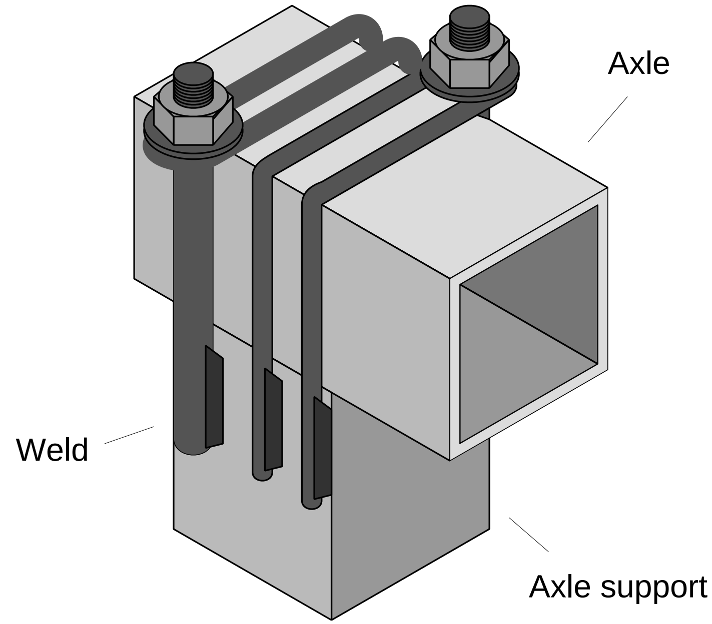


Figure 7: method of fixing axle to axle supports.

6) Nearly there! Now you need to fix the draw bar (or pole or dissel boom - there are so many names for this!). It is best to fix the draw bar to the body so it can be taken off and replaced if it gets damaged. A good way to do this is with stops made of short lengths of box tubing, and round bar welded on as shown in Figure 8. The stops carry the main loads and it is easy to cut through the re-bar hoops if you need to change the draw bar. You will need to put new hoops on of course when you put the new draw bar on.

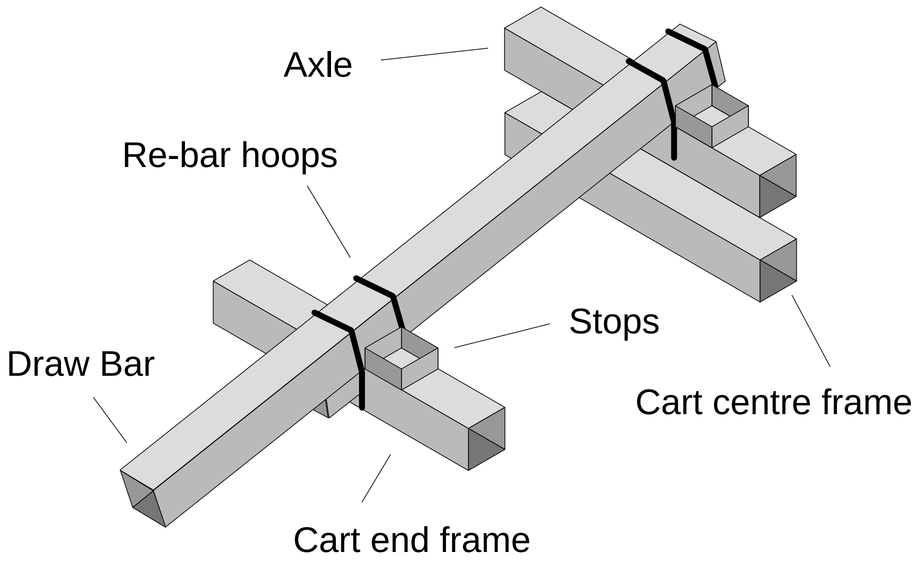


Figure 8: method of fixing draw bar to body. (View of cart upside down.)

7) If you want to make it so that the ends of the load tray can be removed easily you can do so in the way we have shown in Figure 9. Here a piece of the box section steel is fixed to the side planks by clench bars or bolts and stops the end being able to fall out. It is better if box does not touch the bottom planks because then it is easier to clean the corners. The end can be tied to these end stops with rope or inner tube rubber. This is a good way because it is cheap and very easily repairable, but the farmers may want some flashy looking thing which will be very expensive to make. You will probably find that things like latches take longer to make than the rest of the cart. Explain to the farmers that they will cost extra too!

8) Paint or creosote the cart. You've finished it!

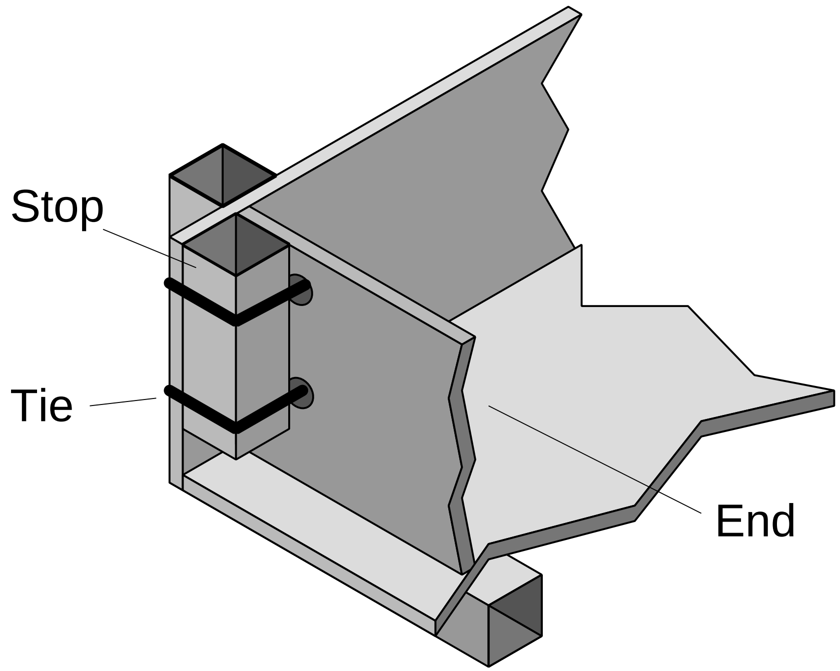


Figure 9: method of fixing ends with rubber or rope

Modifications

There are many different versions of this cart. You can try longer or shorter carts and you can make them wider or narrower. When you do this, check the length and width of the planks of wood that you will use - you do not want to find that you are two inches short of being able to get two runs of plank out of one piece of timber, or that its just too narrow and you have to fiddle about and fit in a narrow strip.

Other DTU cart developments

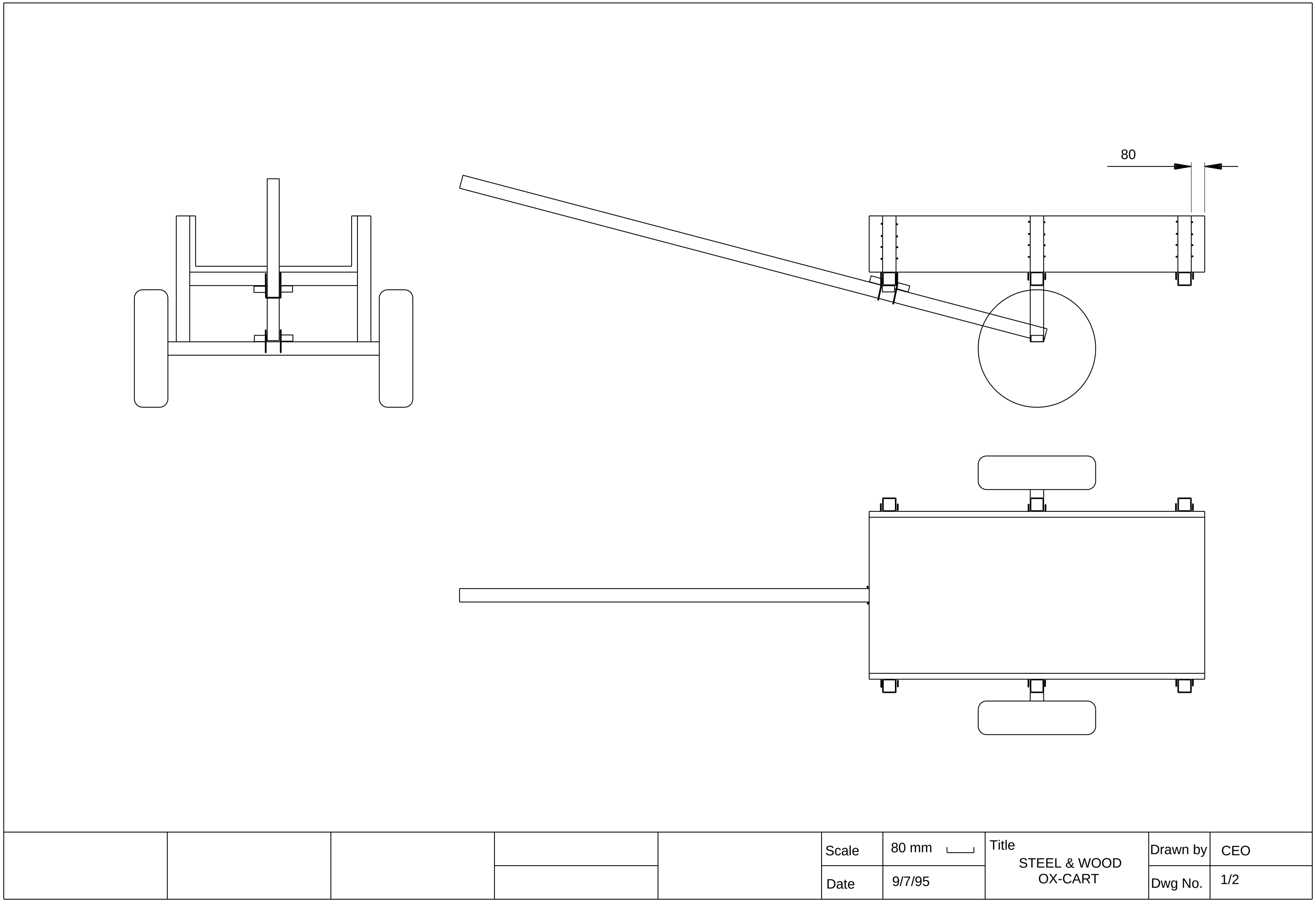
The DTU has been working on a range of cart body types for use with both donkeys and oxen. It has designs for wooden and steel framed types. The wooden types are cheaper in material terms, but the steel framed ones are easier to make because the joints are more straightforward - nevertheless you can make either type of cart in only a few hours, if you are reasonably set up with tools and materials.

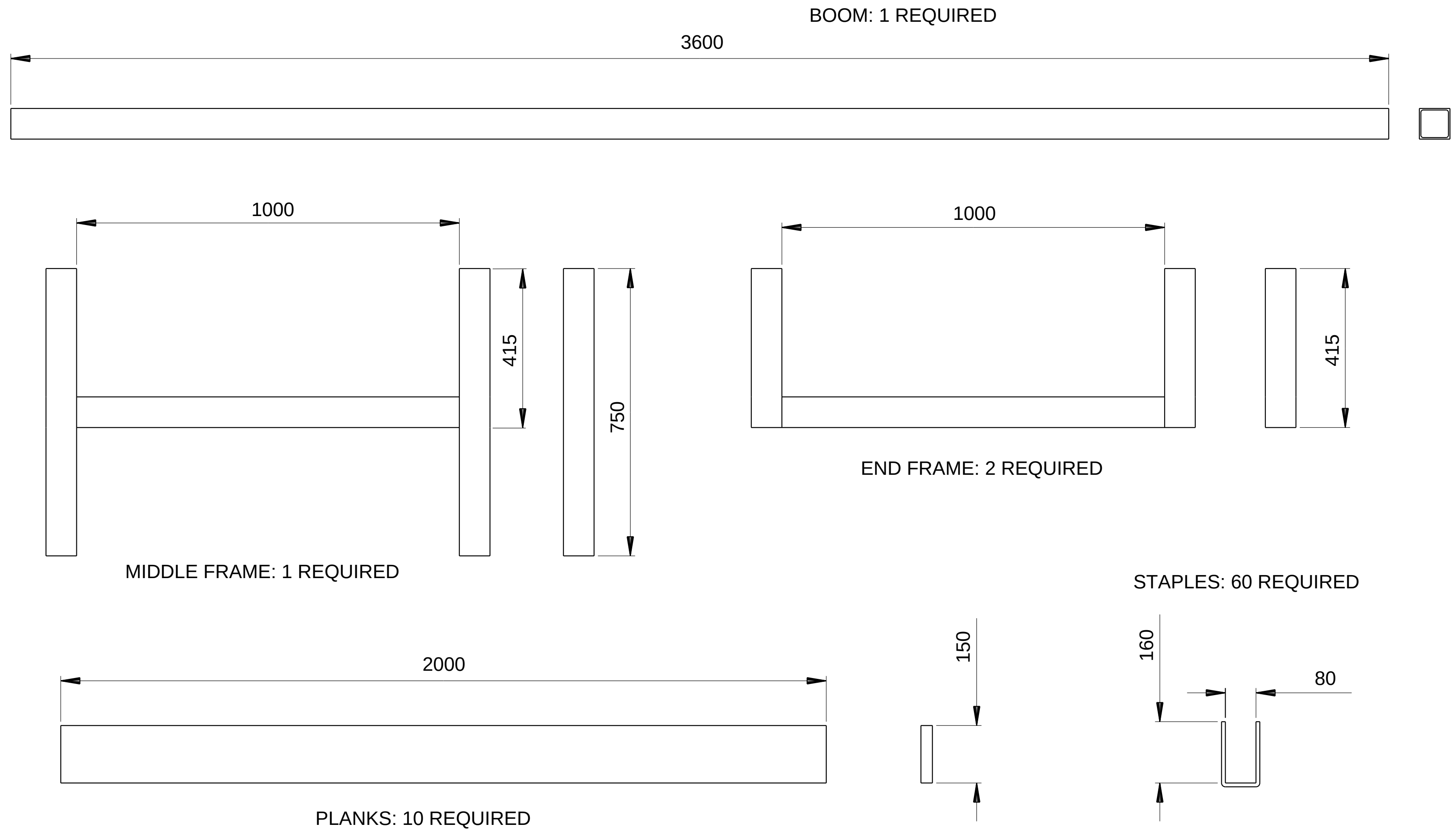
The DTU has also been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. For example it has pioneered a system of hubs using steel pipe such as water pipe which do not need machining to make a roller bearing hub. Obviously friction is low with these hubs and they usually give good milage before being worn out too - we usually get 15 000 km before they are very badly worn, but they may need cleaning and relubrication several times before they get this far. Still they are reasonably cheap - we can make them in Nigeria for about $\$_{US}40$, they only take one man a day, and they do not need any special tools.

Other hub designs using, for example aluminium castings, are in production in Nigeria and we are trying to reduce or eliminate the machining in these. Also wheel designs in steel sheet, cast aluminium and timber are in manufacture or under development.

Cart Drawings

You will find two drawings on the next pages, the first one gives a general view of the cart, and the second gives a view of the main components. As we have said you can vary the size of the cart quite a bit and even make it much longer if you add extra frames. You could even make a four wheeled cart like this!


80
Scale
80 mm
Title
STEEL & WOOD
OX-CART
Drawn by
CEO
Date
9/7/95
Dwg No.
1/2



					Scale	80 mm	Title COMPONENTS: STEEL & WOOD OX-CART	Drawn by	CEO
					Date	9/7/95		Dwg No.	2/2

KENDAT

DTU

Animal Cart Programme

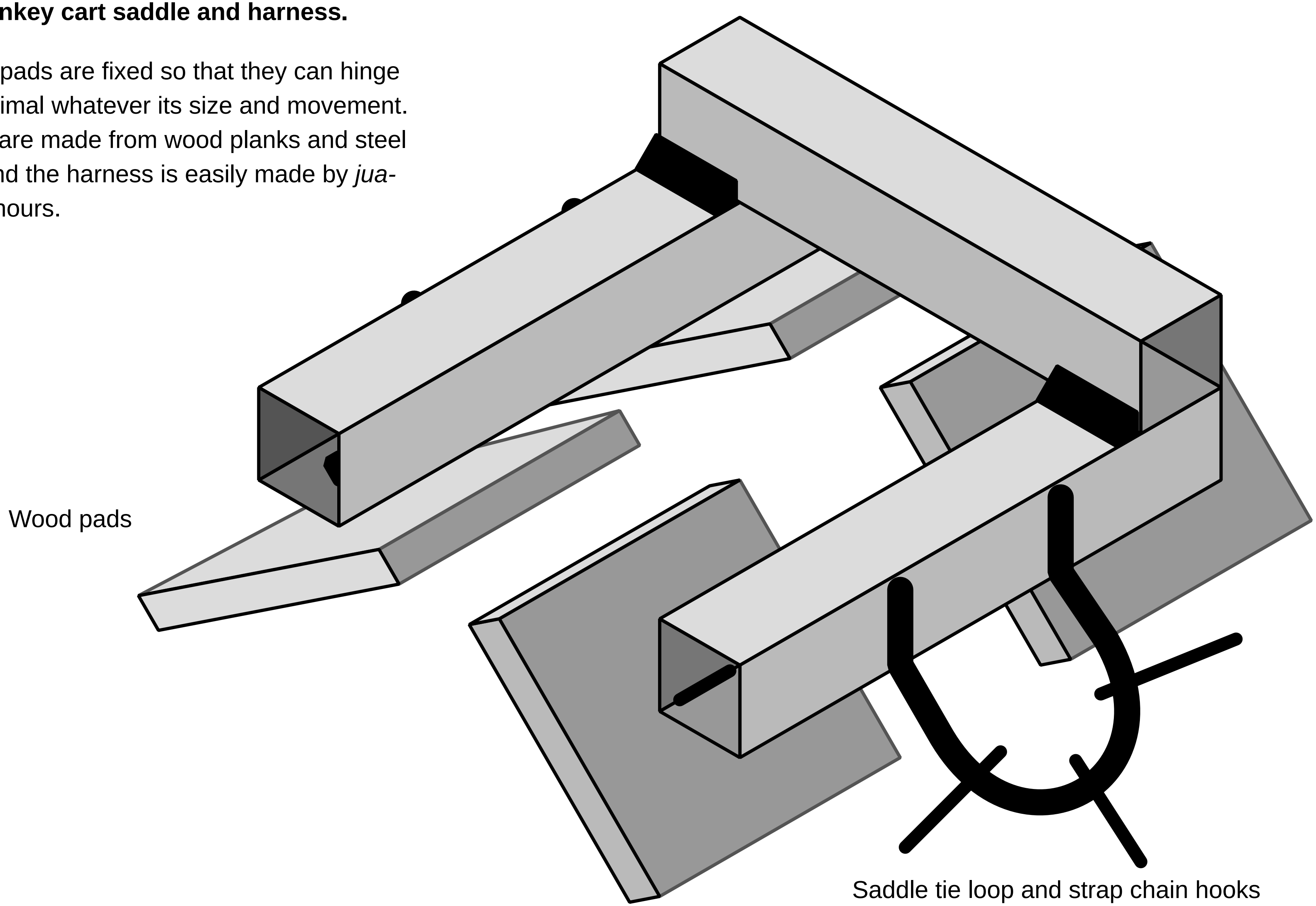
TECHNICAL
27
RELEASE

Single Donkey Harness for Cart Pulling

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)1203 523523 extn 2339, fax: +44 (0)1203 418922, email: dtu@eng.eng.warwick.ac.uk
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: donkey cart saddle and harness.

The wooden pads are fixed so that they can hinge and fit the animal whatever its size and movement. The saddles are made from wood planks and steel box tubing and the harness is easily made by *jua-kali* in a few hours.



Donkey Harness for Carts Made From Steel Box Tubing, Timber and Canvas/ Sacking

Introduction

This Technical release tells you how to make a saddle and harness system for one donkey to pull a cart with two shafts. Another Technical Release tells you how to make a saddle and yoke harness for two donkeys and a cart with a single draw pole.

You should find that you can make the whole harness for less than £$_{UK}$5, depending on the cost of the materials and labour. Once you get organised, two men can probably make a complete harness in two hours - we have designed this harness to be easy to make.

Other booklets in this series tell you how to make simple low-cost axles and carts: we have designs for steel framed and wooden framed carts and for many different kinds of axle. All carts and axles can be made without special tools - even drilling metal is not required.

Idea Behind Design

Saddles are used in many countries to hitch animals to carts. Our saddles use a system of hinged pads which swivel to fit any animal in any reasonable condition. As the animal moves and changes condition the saddle still fits. Using this harnessing method carts can be pulled, steered and braked, and stabilised if the load is too far back on the cart body.

These harness has been tested in Kenya and work well - we have even had donkeys jump over a hedge pulling a cart with this harness! But we would like to test them for a year or two more to see how the animals react.

Special tools and jigs and hard-to-get materials are not required to make the harness. The only tools which you must have are a simple welder, a woodsaw, a hacksaw and a hammer.

The saddle frame is welded and the wooden pads are fixed to the frames with nails which are put through holes in the steel

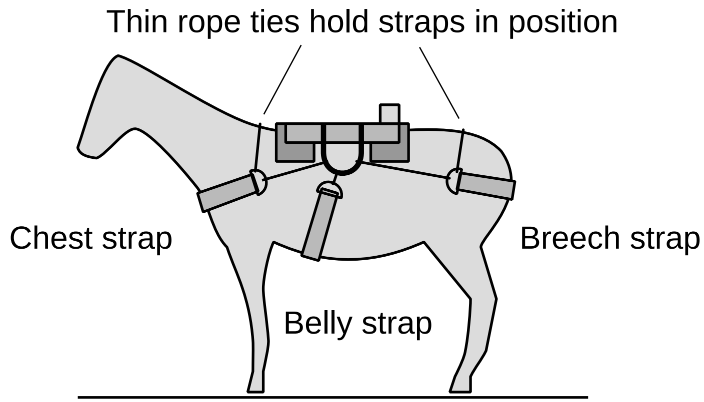


Figure 2: saddle secured to donkey with straps.

frame and welded so that they are loose and allow the pads to follow the shape of the animal.

Cutting list and costs

Table 1 shows a cutting list for a complete harness - recent prices of materials in Kenya are shown converted into £$_{UK}$.

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete.

2) Make up the U-shaped frame as shown in Figure 4. If you have a G clamp you can use it to hold two pieces of the frame together during welding.

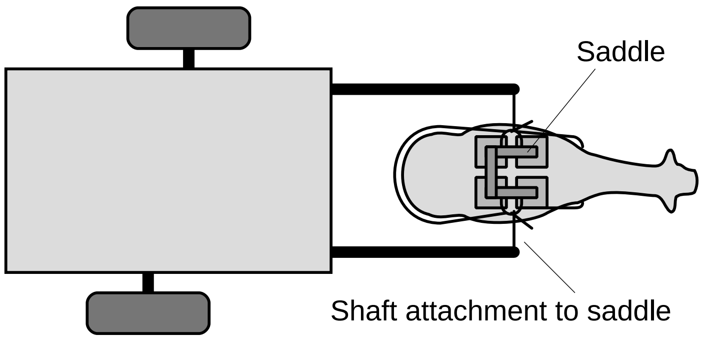


Figure 3: donkey harnessed to cart.

3) Then weld the tie loops and the yoke attachment stub and loop onto the U frames so that the frame looks as shown in Figure 5.

4) Next cut the wooden load pads and round off all the edges so that there are no sharp corners to stick into the donkey.

   Hammer two nails through each of the pads in the positions shown in the drawings. With some timbers you may need to drill holes for the nails to avoid splitting or burn the holes with a hot nail. Then cut the nails so that about 30 mm projects from the timber as shown in Figure 9.

5) Now mark the position of the holes required to accommodate the pad nails in the steel tubing. These holes

TABLE 1: harness materials cutting list.

component	material	# components	total mat [mm]	mat cost [£$_{UK}$]
main frames	50×50 box tubing	3×325	1800.00	1.06
strap loops	12 mm re-bar	2×300	1200.00	0.19
load pad pivots	12 mm re-bar	4×20	160.00	0.03
load pads	25×150 timber	4×150	1200.00	0.14
pad fix nails	nail or 6mm re bar	8×50	800.00	0.04
strap rings	6mm re bar	6×180	2160.00	0.11
strap clenchers	6mm re bar	6×120	1440.00	0.07
strap hooks	6mm re bar	6×150	1800.00	0.09
straps	CC5 canvas	3×4×65	1560.00	1.97
strap chains	dog chain	3×300	900.00	0.70
			TOTAL =	**4.40**

should be 15 mm and 65 mm from the ends of the square tube as shown in Figure 7. Blow the holes through with the welder at maximum current setting or use an angle grinder or file or hacksaw.

6) Next you can weld on the pad pivots blocks as shown in Figure 7.

7) Now put the nails through the blown holes and weld a piece of nail across the ends of the nails as shown in Figure 10. Welding down inside the tube looks difficult but skilled workers can weld the pads in about one minute.

An alternative way of doing it is to cut slots 70 mm long along the corners where the holes would be as shown in Figure 8. The slots should be 8 mm wide so that the nails are very loose in them. Make up the pads as shown in Figure 9, put the nail loop into the slot in the right place and weld the pad pivot blocks into place across the slot so that they are in the same place as in Figure 7.

8) Next you need to make up the six straps to hold the saddles onto the donkeys. The D rings at the end of the straps can be made from 6 mm diameter concrete reinforcing bar as shown in Figure 11. A separate piece of the re-bar is clenched over the strapping using hammer blows to fix the D rings to the ends of the straps as shown.

The straps themselves can be made from heavy canvas or hessian sacking. You should use three or four thicknesses

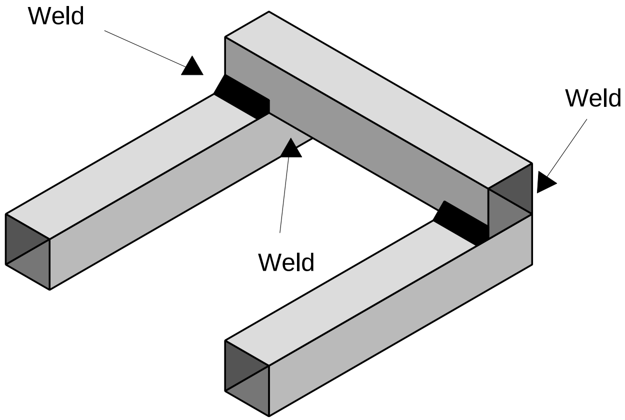


Figure 4: welding of frame cross piece.

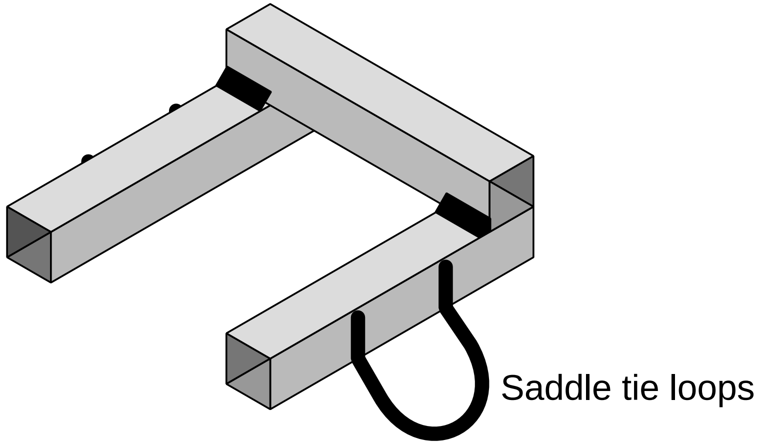


Figure 5: tie loop welded to U-frame.

of material for them to make them strong enough and soft enough not to hurt the donkey.

9) Make the strap chain hooks from more 6 mm re-bar as shown in Figure 12 and fit the fixed ends to the saddle tie loops.

10) Paint and creosote the saddle. You've finished it!

Method of harness use

1) First put a blanket or two folded hessian or jute sacks (not plastic) onto the donkey's back to protect it.

Remember that protecting the donkey will save money because it can work harder if it is comfortable and will not get sick from skin wounds.

2) Put the saddle on so that the cross beam is towards the animal's rear. Position the fronts of the wood pads about 100 mm behind the animal's shoulder blades. This means that the saddle should never come near parts of the animal's back which move.

3) Next hook the breaching strap to the loops hanging from the side of the saddle. It should be tight enough to tend to pull the saddle a little rearwards. Make sure that the breaching strap is pulled up high so that it does not rub the backs of the legs. But it should not be so high that the animal cannot defecate. Tie a piece of thin rope across the animal's back between the rings of the strap to hold the

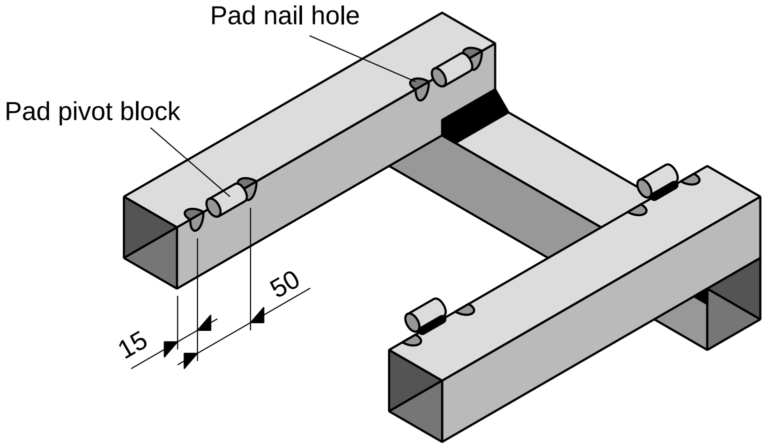


Figure 6: pad nail hole positions and pad pivots.

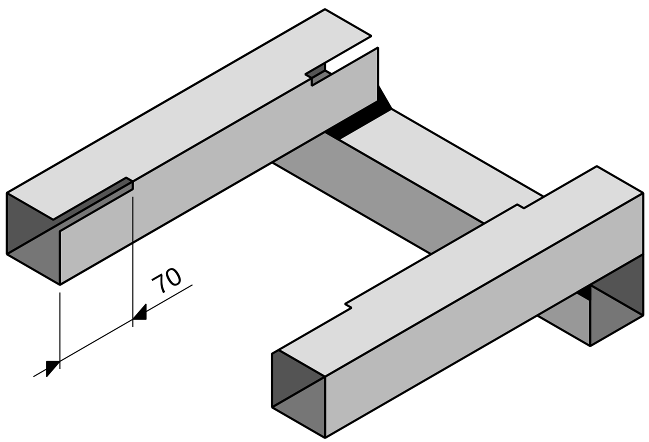


Figure 7: pad nail slots.

strap up.

4) Now hook the chains for the belly strap onto the hooks fixed to the saddle tie loops. The strap should be 50 mm or 100 mm behind the front legs - check that the legs do not rub on the strap when the animal walks. Tighten the strap so that you can just get a couple of fingers under it between the strap and the animal. This will be much tighter than the other straps.

5) Hook the chest strap to the loop and adjust the tension so that it is a little loose. Use another short piece of rope to hold the chest strap up so that it is just below the windpipe. The strap goes tight when the animal pulls really hard. We have noticed that the belly strap and breaching strap are nearly enough without the chest strap and so we leave the chest strap a bit loose.

6) You are ready to go! You should be able to saddle an animal in only a few seconds when you get practised.

Saddle Drawing

You will find drawings of the saddle and yoke on the last pages of this Technical Release.

Other DTU cart developments

The DTU has been working on a range of cart designs for use with both donkeys and oxen. It has designs for wooden and

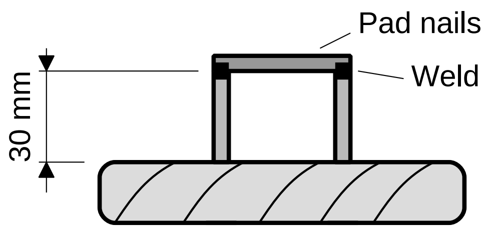


Figure 8: welded pad nails.

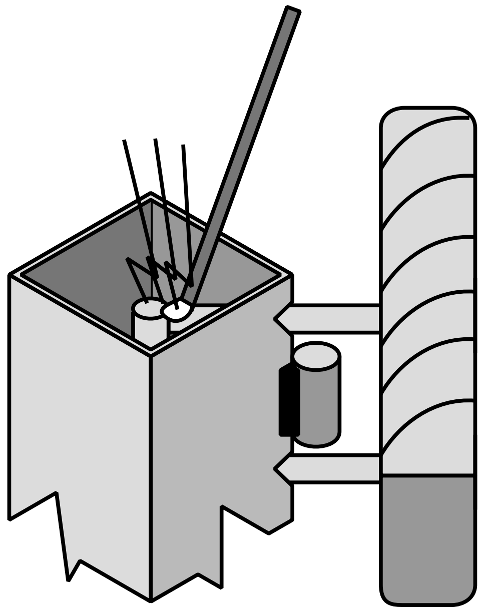


Figure 9: welding re-bar to frames for wooden load pads.

steel framed types. You can make either type of cart in only a few hours, if you are reasonably set up with tools and materials.

The DTU has also been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. It has a system of axles with bearings made from PVC pipe, another with wooden bearings and a third using scrap ball bearings. None of these axles need machining and they only take two men a day to make.

Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.

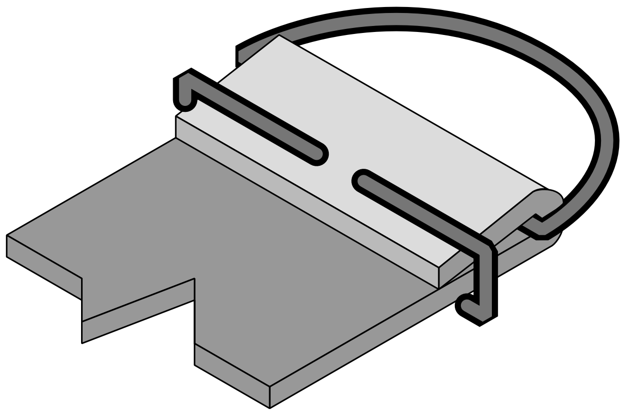


Figure 10: D rings for straps made from re-bar.

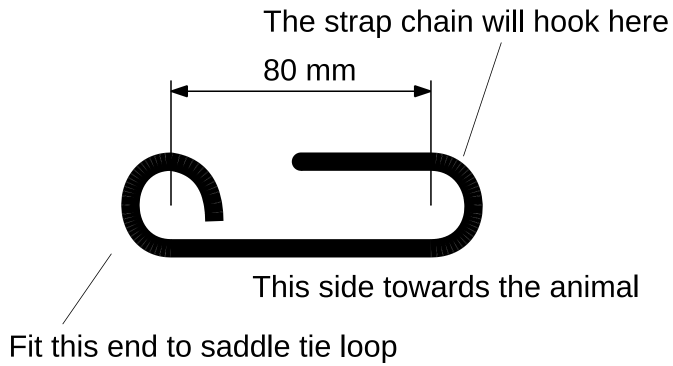
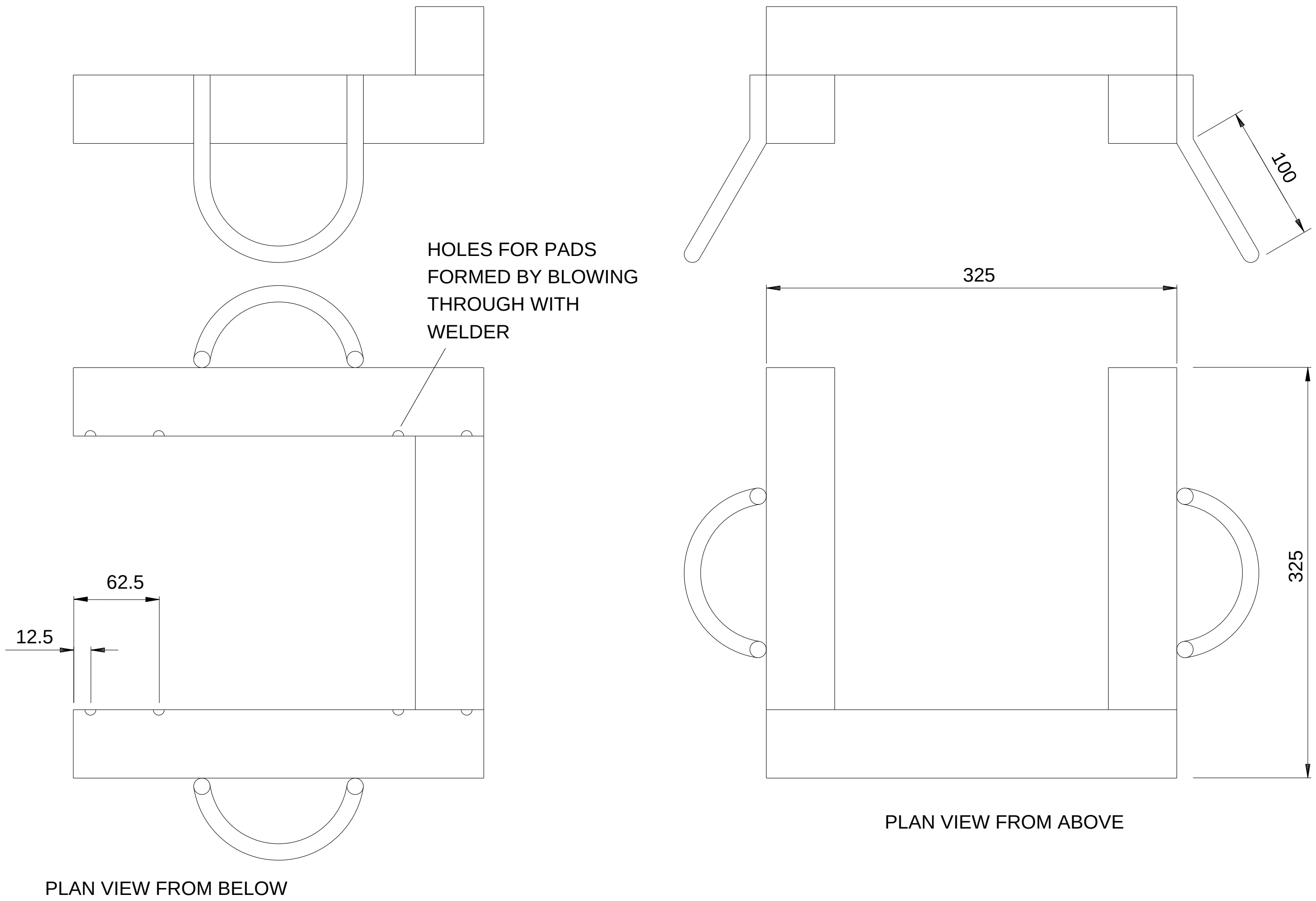
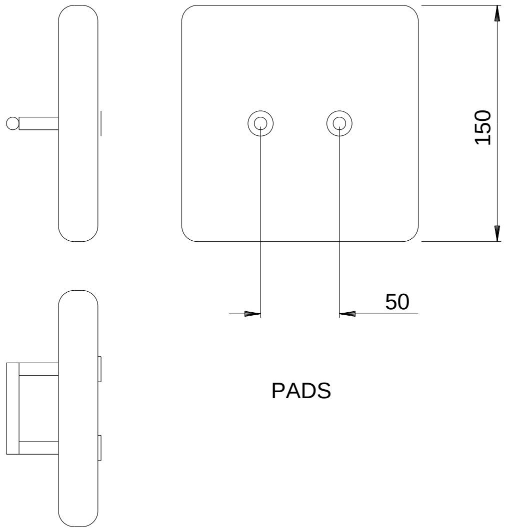
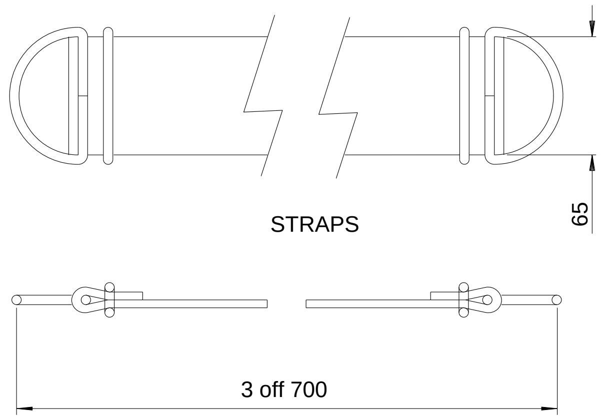


Figure 11: chain hooks for straps.



Scale	50 mm	Title Saddle for donkey carts - frame	Drawn	CEO
Date	15/4/99		Dwg #	1/2





					Scale	50 mm	Title	Drawn	CEO
					Date	15/4/99	Saddle pads & straps	Dwg #	2/2

DTU

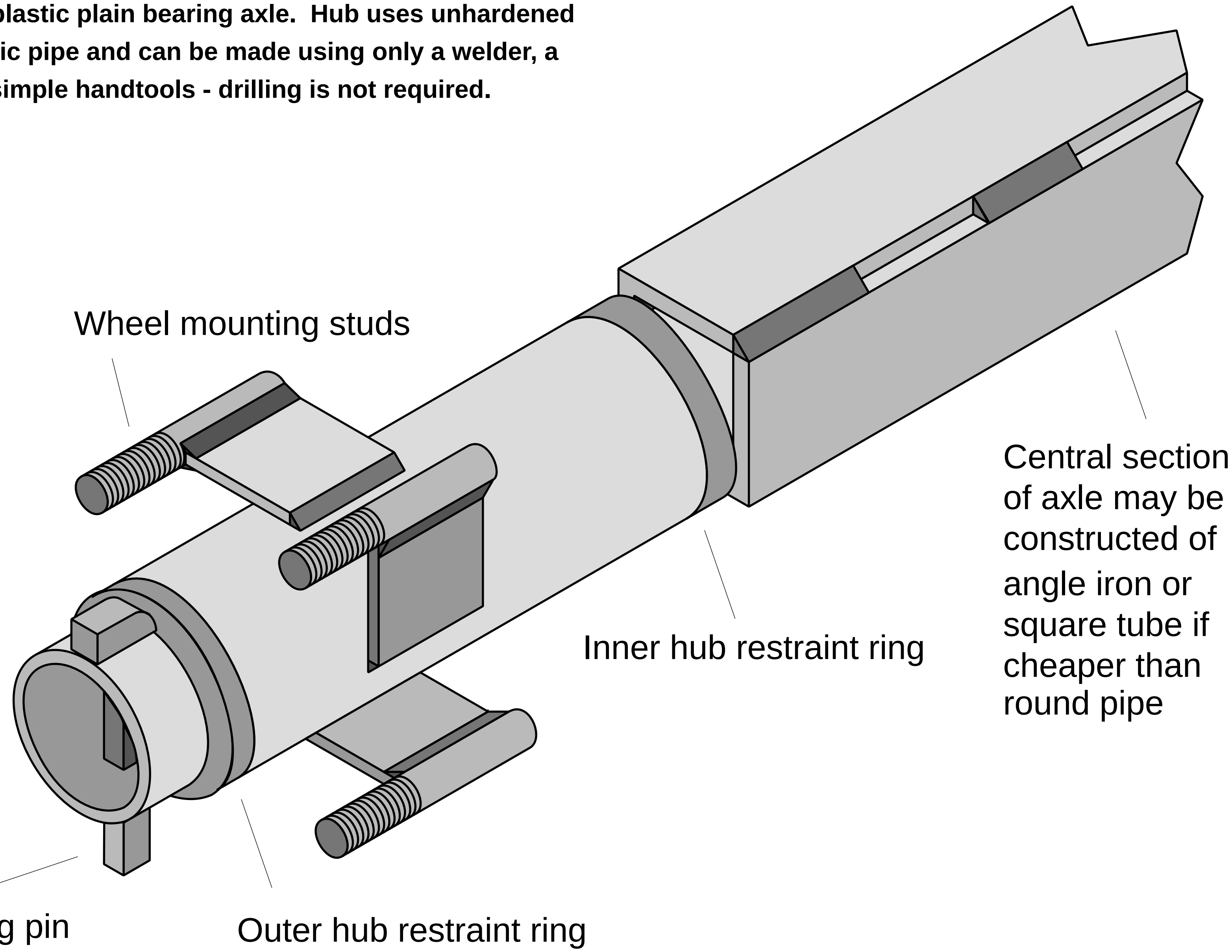
Animal Cart Programme

TECHNICAL
28
RELEASE

PIPE PLAIN BEARING DONKEY CART AXLE

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk

Figure 1: pipe and plastic plain bearing axle. Hub uses unhardened mild steel and plastic pipe and can be made using only a welder, a hacksaw and very simple handtools - drilling is not required.



Pipe Plain Bearing Donkey-Cart Axle

Introduction

Not enough farmers in Africa have animal carts. Those who have carts can take their produce to places where they can get the best prices. They can also get into town and buy fertilizer and better seeds and move things around their farm easier. The trouble is that carts are too expensive for many farmers. The question is what can be done about it?

Carts are made in many different places. Some carts are made in factories in industrial countries and some are made in factories in Africa, but most are made by local blacksmiths or carpenters using scrap car and 4WD axles. In many countries these people cannot get enough axles to meet the demand so the price is high. Another problem is that the axles are often so worn that they do not last long. Lots of farmers take the (differential) unit out of the axle too, which makes the axle break sooner and lets the dirt in.

What you need is an axle which blacksmiths and fabricators can make with fairly simple tools - without having to get parts machined. There are usually blacksmiths and fabricators in the small market towns used by the farmers. Experts think that having the cart maker close to the farmer is a good thing because they can talk to each other easily and sort out any problems. And of course if the cart is made locally, it can be repaired locally, so there will not be problems with spare parts.

Idea behind design

The idea behind the design of axle described in this technical release is to allow construction without the use of machine tools (drills, lathes and milling machines), and to use materials which should be readily available. The materials can be used 'as bought' - no hardening of any of the components is needed. The only tools which you must have are a welder and a hacksaw, but a file and a vice are also very handy. Of course if you do have power tools - especially a power hacksaw or anglegrinder with cutoff wheel - things can be made much faster. This axle is suitable for a wide range of production methods so that if you have to make many of them you can make special tools to make it quicker.

The hubs in this design use 2"BSP pipe on $1^1/_2$" BSP axles so that they have a small diameter and will fit most scrap car wheels. Most wheels have a hole in the middle for the axle.

This hole is usually about 60mm diameter, or a bit bigger. Sometimes they are smaller and the wheel will not fit, but you can sometimes saw or file the hole bigger if you have to. Another way of avoiding the hole in the wheel problem is to use live axles as described in Technical Release 29.

The best way to make these axles is to use a piece of plastic pipe between the axle and the hub tube to make a bearing. The best plasic is probably polythene, but this is hard to get in short lengths so we have used PVC in Kenya. Search around the stores in your area to see what is available. If the only steel pipes you can find for the axle and hub tube do not have enough gap between them for a plastic bearing then you can still get good performance without it, but the axle will wear a bit faster and the cart will be a little harder to pull. Alternatively you can enlarge the hub tube a little see the **modifications** section.

You will see from Figure 1 that the wheels are fixed to the axles by struts rather than by thick metal discs as on most axles. Putting the fixing studs on struts like this is much easier and cheaper. It also means that you might be able to fit a slightly different wheel by bending the struts a bit to fit. Or if that does not work you can even cut nearly through the welds and then weld them in the right place. You could even cut the struts right off and weld on a different number if your wheels have a different number of holes.

Performance

We have not yet properly tested axles like these, but many farmers and water carriers in Kenya have used them on their carts for three years or more without even a plastic bearing. Of course you need to clean the axle and hubs out and regrease them every year at least and if you use them heavily you should grease them every day. Actually this is easy because all you need to do is remove the hub restraint pin through the end of the axle, take the wheel and hub off, put some grease on the axle and replace everything. It should only take a few seconds.

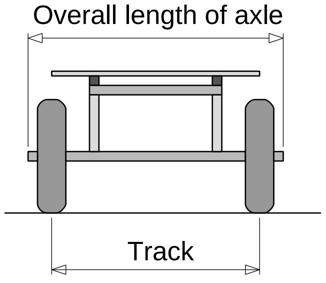


Figure 2: track width of cart.

Length of axle

You need to decide how long to make your axle. Of course you can make the axle any length you like to suit your cart, but most carts will need an overall axle length of between 1400 to 1600 mm. In the **cutting list and cost** section you will see that you can make the axle with a central section of some other material than the $1^1/_2$" BSP if this is very expensive. We have assumed that you will use square box tubing but you could use angle iron. To calculate how long to make the axle decide how big you want the **track** to be (see Figure 2) and add 280 mm. This distance will now be the **overall length of the axle** (see Figure 2 for this also). If you are going to use square box tubing or angle iron for the central section of the axle then you should subtract 240 mm from the **track** and make the central section this long.

Cutting list and costs

The table shows a cutting list for a complete axle - two wheel hubs with stub axles joined by a square tube section in the middle. We have shown this way because round pipe is very expensive in some countries. But if pipe is reasonably cheap where you are then make the whole axle out of one piece of pipe. Recent prices of materials in Kenya for the axle are shown in $\$_{US}$. The 2" BSP (British Standard Pipe) is about 61mm outside diameter, or a bit less, with a wall thickness of about 3.6mm. The 1½" BSP is about 49mm outside diameter, with a wall thickness of about 3.2mm.

component	material	number & length reqd [no.× mm]	total material in axle [mm]	materials cost in Kenya [$US]
central axle	2" square box tubing	1×1200	1200	3.20
hub stub axles	1½" BSP malleable iron pipe	2×350	700	2.60
hub outer tube	2" BSP malleable iron pipe	2×200	400	1.80
optional plastic brg	1½" or 2" plasic pipe	2×200	400	0.47
hub retaining rings	6mm or 8mm square bar	4×154	616	0.12
hub restraint pegs	6mm or 8mm square bar	90×2	180	0.04
wheel studs	50 × 12mm bolts	8×50	na	2.40
wheel stud struts	6×40 black or bright steel strip	8×37	296	0.60
TOTAL COST =				11.23

Construction step by step

1. The first and probably most difficult job, is to get some suitable pipes. Obviously the axle has to be strong enough to carry the cart, so it should be made from pipe bigger than about 40mm outside diameter. You must also make sure that the pipe has a wall thickness of more than about 2.5 mm.

The hub pipe also must have a wall thickness of 2.5mm or more. And it must have a bore (or inside diameter), which goes easily over the axle with enough room for the plastic bearing and some grease. There can be quite a lot of clearance (slackness or looseness) between the hub and the axle (say up to 3mm) - it does not have to be tight. If you cannot get steel pipes with enough clearance for a plastic pipe bearing then the axle will work quite well without it, but you should grease it more often.

These instructions deal with making an axle to the design shown in the drawings. If you find that you cannot get the right sizes of material you might still be able to make an axle with other sizes. See the **Modifications** section later in this booklet.

2. You will probably find that no plastic pipe fits properly between the axle pipe and the hub tube but all you need to do is slit a piece along its length and open it up a bit or close it down until it fits. A better way is to make a helical saw cut in the plastic pipe so it is like a spring. Then it will open up or close down to the size easily and it will not tend to wear the area around the slit.

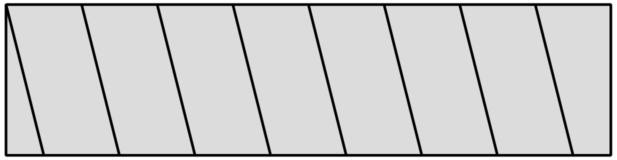


Figure 3: helical sawn cut in platic pipe bearing sleeve.

3. When you have got the right pipe sizes, you can cut the two hub pipes each 200 mm long and the axle pipe about 1500 mm or 1600 mm long or if you are going to do it with a central section in square pipe or angle iron you need to cut the central section about 1200 mm long plus the two stub axles each about 350mm long. If you need to support a simple one piece pipe axle an easy way is to weld short pieces of round bar onto it as shown in Figure 4. If you make these about 300 mm long they will strengthen the axle quite a lot as well.

   Figure 5 shows how you can weld the stub axle tube into the square box tubing. You will need to cut slots in the sides of the square tube so that it can bend in or out to accept the stub-axles. When you are welding the stub axles in, make sure you tack weld them in first and check that they are straight before you do the final weld.

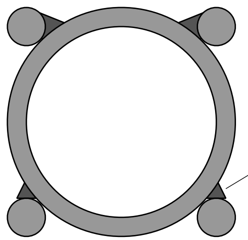


Figure 4: method of adding round bar to support pipe axle and strengthen it if needed.

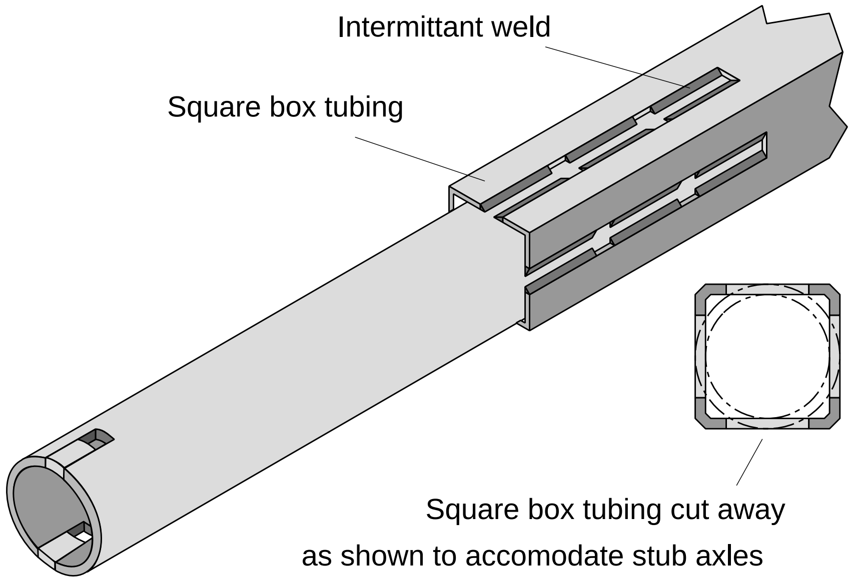


Figure 5: method of welding stub axles into square tube.

4. Next you need to make a hole near each end of the axle pipe for the hub restraint pin. To do this without using a drill cut a slot in each side of each end of the axle. Make the slots about 30 mm long to start with and about 10 mm wide. You want the slots just the right length so that with the hub restraint pins in place the hub tubes are not pinched tight. It does not matter of they are a bit loose.

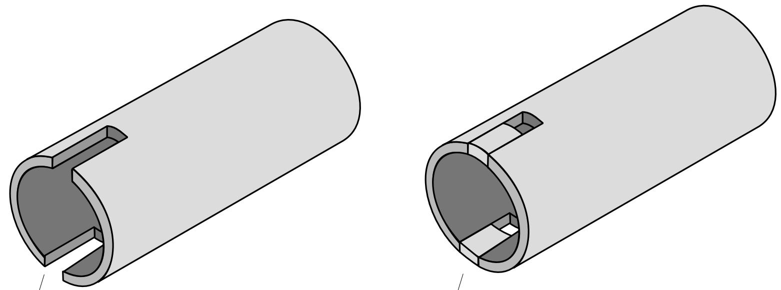


Figure 6: method of making hole for hub restraint pin without drilling.

5. The next step is to weld the stud bolts or bits of threaded rod onto the struts. If you have a vice it makes welding the bits together easier because you can clamp the pieces together while you tack weld them. You need to make one strut for each wheel stud unless you are using five or six

stud wheels on a single donkey cart. If so you could use only three studs per wheel if the cart user wants only light use from the cart.

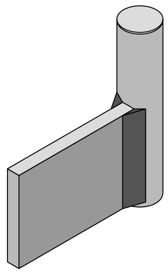


Figure 7: stud welded to the wheel struts.

We have used this method of fixing wheels successfully with the old VW wheels which have very large holes in the middle.

When you've made up the struts you can put a nut on each stud and then put the studs through the wheels, and put a second nut on each stud. Then get everything even and straight with the hub pipe in place as well in preporation for welding the struts to the hub tube. You want to get the middle of hub pipe level with the middle of the tyre, as is shown in Figure 7. Most car wheels need the studs to be about 40mm offset and this is what is on the drawings.

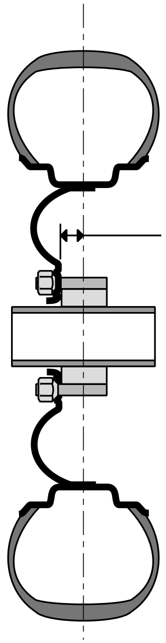


Figure 8: cross section of tyre wheel and hub tube showing centering of hub tube in wheel

Getting the hub in the middle of the wheel and tyre means that the bearings in the hub are evenly loaded. You might find that holding the hub tube in the wheel with some wedges as shown in Figure 8 is a good way to do it.

When you are happy, tack weld the struts to the hub pipe. Then remove the wheel and wedges so you've got room and finish the welding of the struts to the hub tube. Repeat this for the other hub. If you are going to make several axles you can make up a simple jig, rather than the wedges, to hold everything for welding. We have used a piece of plywood with a central hole to fit snugly over the hub tube and four holes for the studs. In other words its a bit like a dummy wheel.

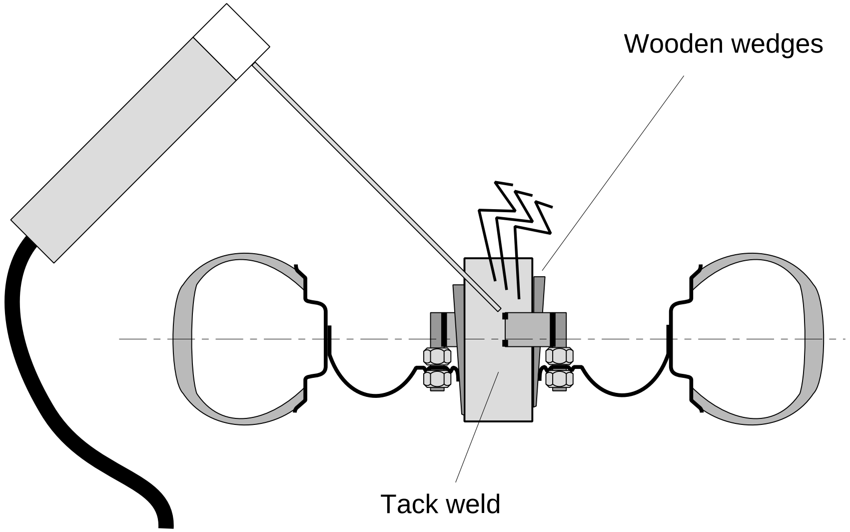


Figure 9: cross section of tyre wheel and hub tube during tack welding of stud support struts

It is best to check that the hubs, and stub-axles still go together when you've finished welding because sometimes weld contraction can pull it all out of shape and make it all too tight. You might need to file off some high spots inside the hub. If you can get it together without a hammer you'll be ok because it will wear to the right shape.

6. Nearly there! Now take the wheels off the hubs and make up four rings (called hub restraint rings on the drawing) from 6 mm or 8 mm square steel bar (or round bar if you cannot find square). One ring must be welded about 230 mm from each end of the axle pipe or from the ends of the stub axles to stop the hubs going too near the centre of the axle.

7. Now put it together! Smear some grease onto the plastic bearing pipe (if you are using one) and onto the axle and put the hub tube and plastic bearing onto the axle. Then put the outer hub restraint ring on and secure it with the hub restraint pin. Do the same for the other hub.

8. Paint everything!

9. You've done it!

Modifications

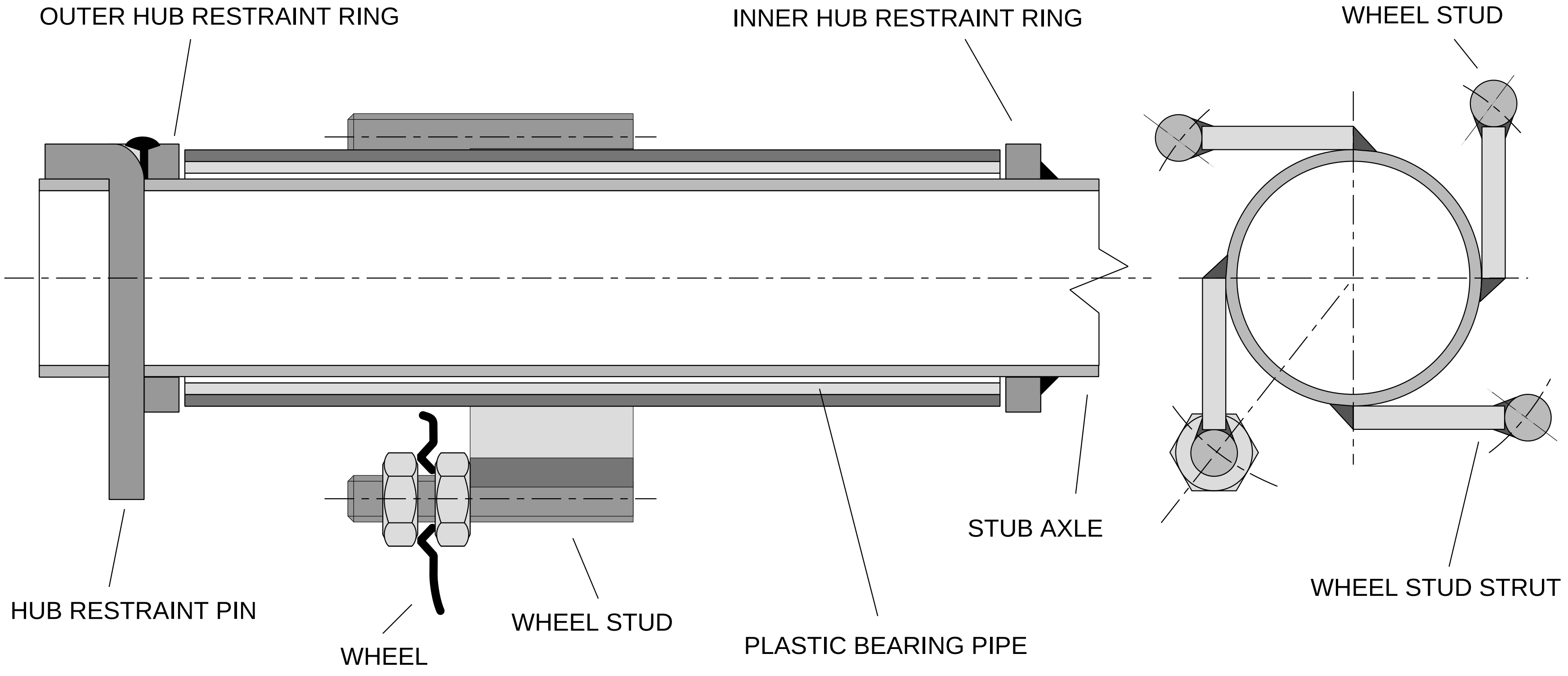
Pipes can be made slightly bigger (up to 1mm bigger) by forcing a short piece of round bar of the right diameter through them with a press. Another way to do it is to saw the pipe along its length and open it up to the right size and then weld it. You can also make it a bit smaller like this by cutting a wider slot and squashing the pipe down. Try to clean the weld back flush with the tube using a file if you can as this will give longer life. It does not matter if

If you find that you cannot get anything like the materials talked about in the cutting list then maybe you can adapt the design a bit. If the hole in the middle of the wheels is big then you stand a better chance of finding a combination of pipes and rollers that will fit. You can often cut a bit out of the middle of the wheel to make the hole bigger. The hole in Land Rover wheels is big and you can get 4" pipe into them. Of course the shaft does not have to be a pipe - it could be solid and then it could be a bit smaller, say 30mm diameter if the steel is high quality.

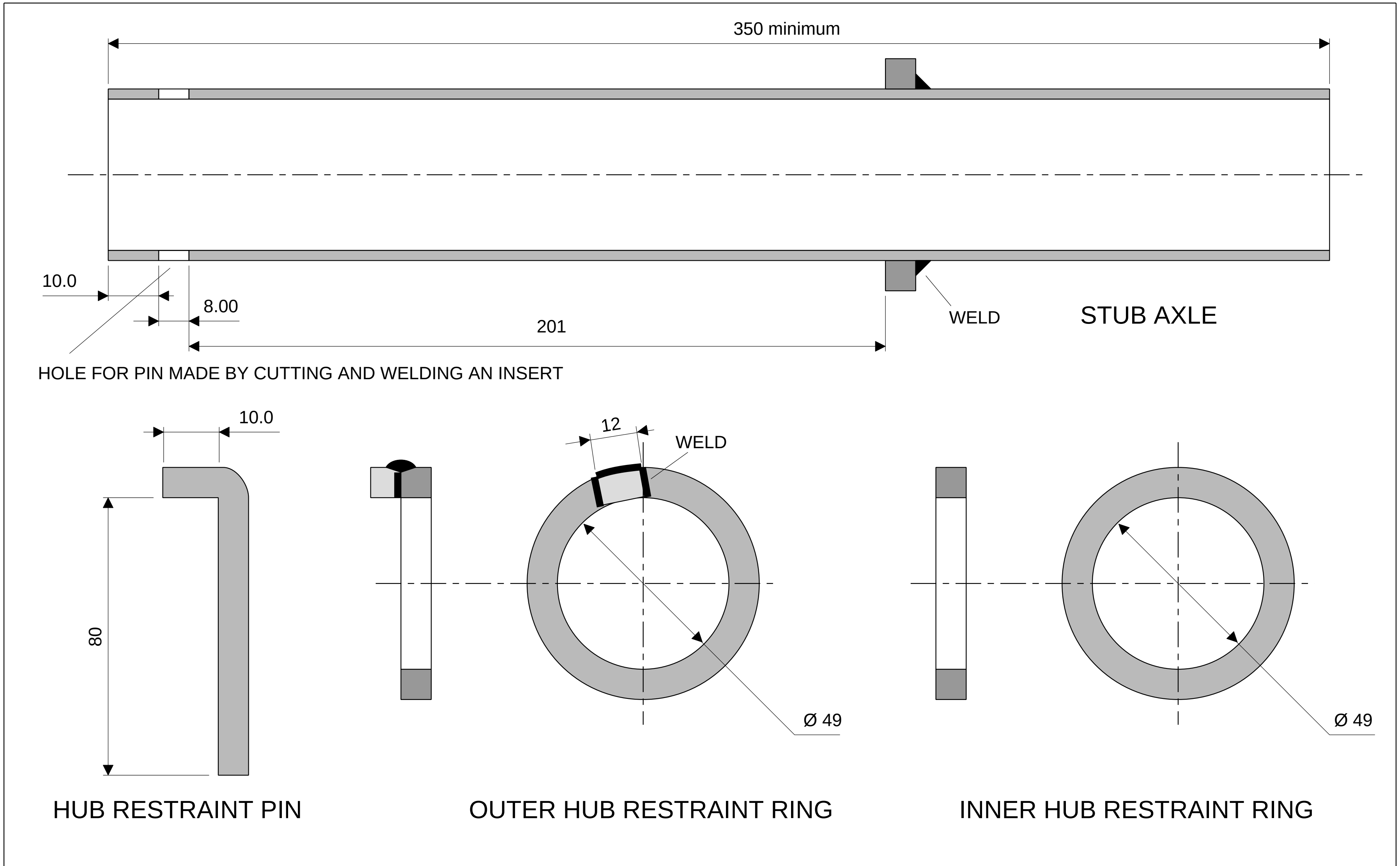
Another idea that we have tried is to use a hardwood as the hub and even the wheel. If you think about it, the wear on something which is rolling must be less than when something is sliding over it, so a wooden bearing should be better than a sliding one. Some bearings we have tried have had a steel ring fitted inside so that the rollers roll on this steel. We have also tried making these rings from round bar like wire so that it's like the rollers roll on the inside of a spring. This seemed to work quite well.

Other DTU cart developments

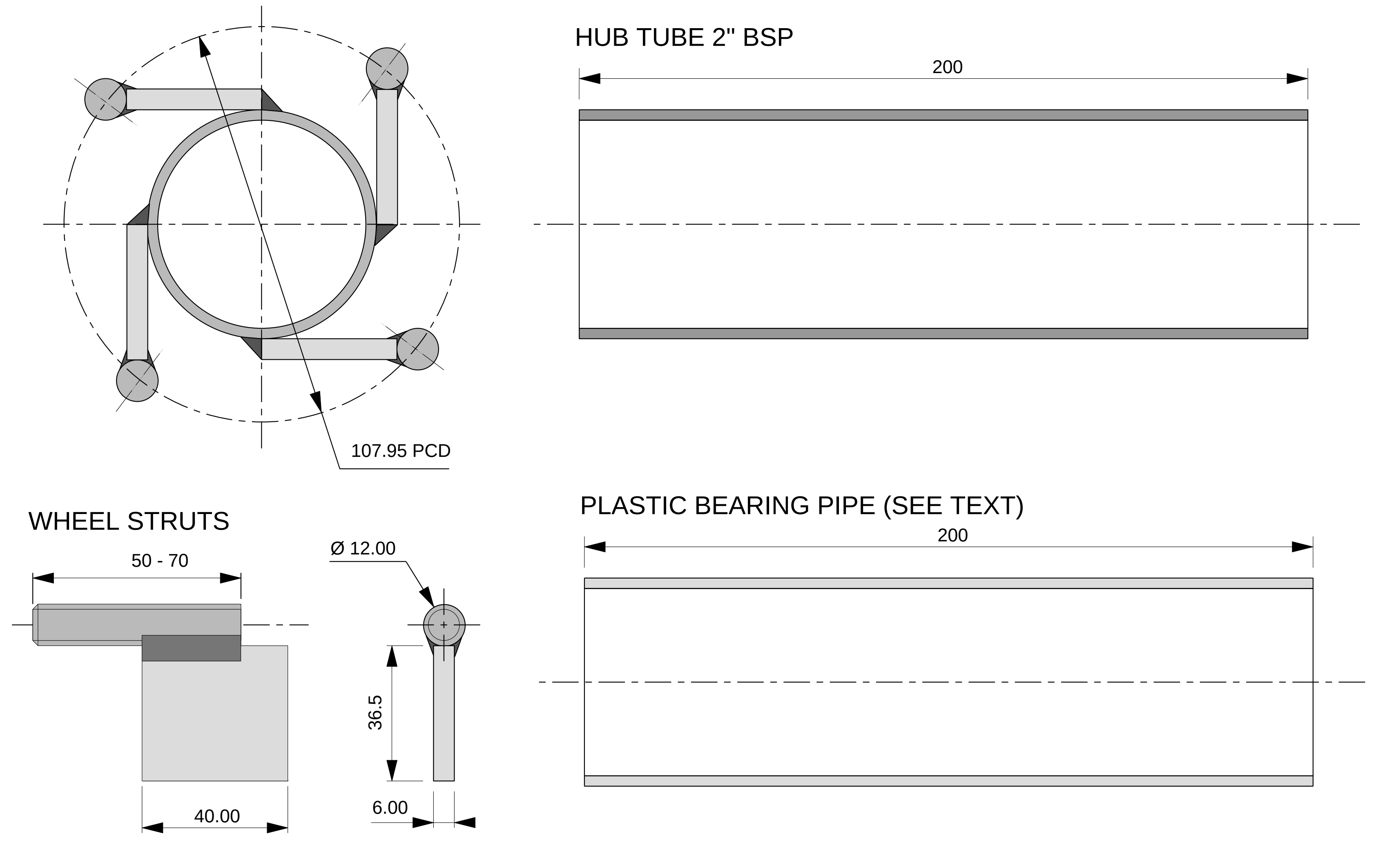
Other methods of hub design using aluminium castings, for example, which might need no machining, are under development at Warwick and wheel designs in steel sheet, cast aluminium and timber are also in manufacture or under development. A range of designs for donkey and ox carts made of steel and wood, is also available, some of which are in production in Kenya and Nigeria.



GENERAL ASSEMBLY DRAWING	Scale	10mm	Title	PIPE PLAIN BEARING DONKEY CART AXLE	Drawn by	CEO
	Date	1-1-97			Dwg No.	1/3



	HUB AND AXLE COMPONENTS	Scale	10mm	Title PIPE PLAIN BEARING DONKEY CART AXLE	Drawn by	CEO
		Date	29-12-96		Dwg No.	2/3



HUB TUBE, PLASTIC BEARING & WHEEL STUDS	Scale	10mm	Title PIPE AND PLAIN BEARING DONKEY CART AXLE	Drawn by	CEO
	Date	1-1-97		Dwg No.	3/3

KENDAT

DTU

Low-Cost Animal Cart Programme

TECHNICAL
29
RELEASE

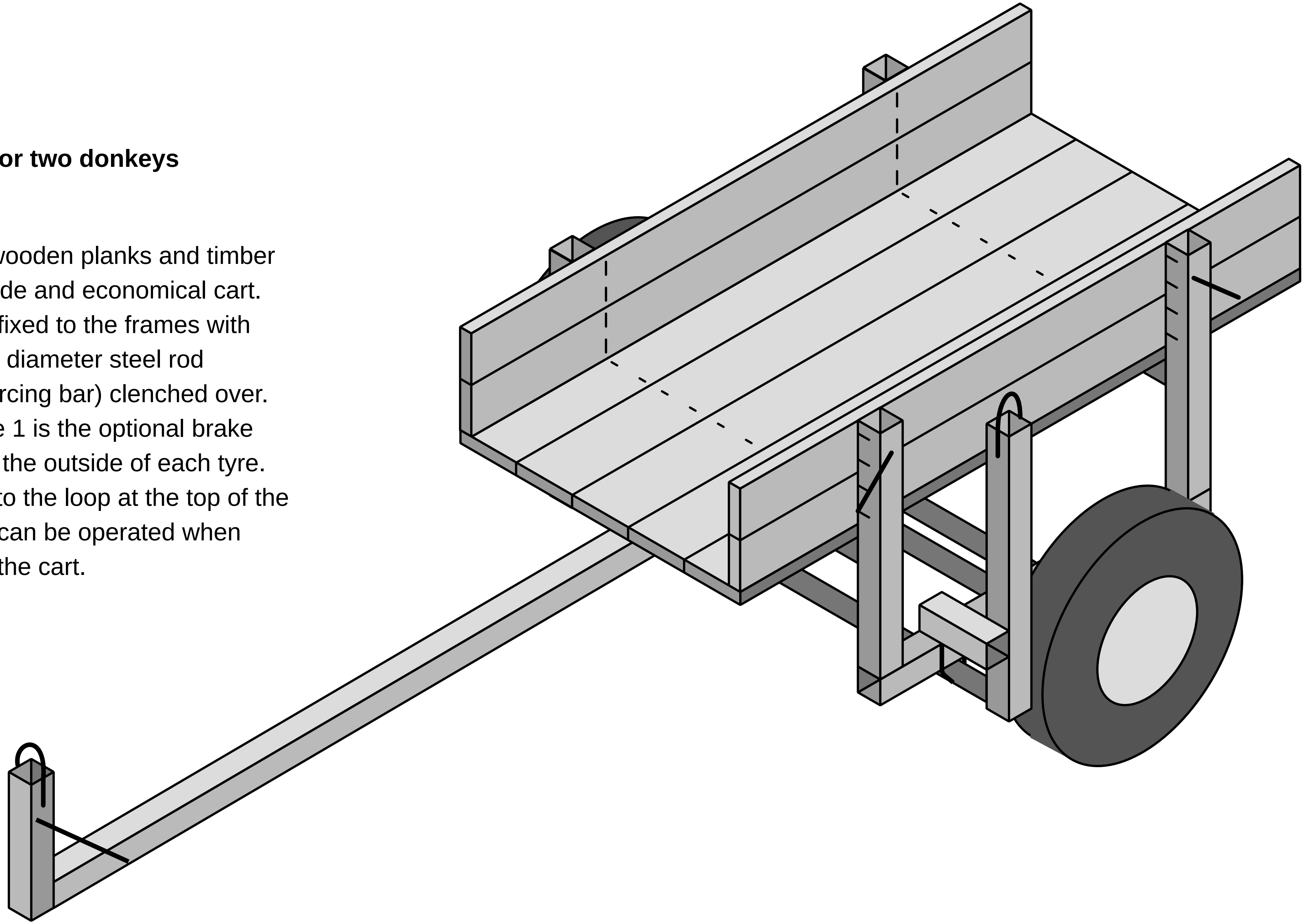
Low Cost Steel Framed Cart for Two Donkeys

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk

KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: cart for two donkeys with brake.

This cart uses wooden planks and timber for a quickly made and economical cart. The planks are fixed to the frames with 8 mm or 10 mm diameter steel rod (concrete reinforcing bar) clenched over. Shown in Figure 1 is the optional brake which works on the outside of each tyre. If a rope is tied to the loop at the top of the lever the brake can be operated when walking behind the cart.



Two Donkey Cart made from Steel Box Tubing and Timber.

Introduction

In this booklet we tell you how to make a simple cart from square steel tube and timber. This Technical Release does not tell you here how to make the axle - you will have to read another Technical Release to for this. We have designs for stub axles with PVC bearings and with needle roller bearings that you can make yourself and we have designs for twin offset axles using PVC, wood and scrap ball bearings. All axles can be made without machine tools - in fact you do not even need a drill!

You should find that you can make the cart itself for about £ 40 depending on the cost of the materials and labour. An axle plus wheels, tyres and tubes will cost another £ 50 - £ 60. Once you get organised, two men can probably make one cart in a day. We've designed these carts to be easy and quick to make.

Idea Behind Design

These carts are designed to be constructed without lots of special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a welder, a wood saw, a hacksaw, and a hammer. You might find that a couple of 4" or a 5" G clamps (or something like it) are useful too.

The cart frames are fixed together by welding and the wooden planks are fixed to the frames with clenched steel bar. You weld 8 mm diameter re-bar (concrete reinforcing bar) to the steel box

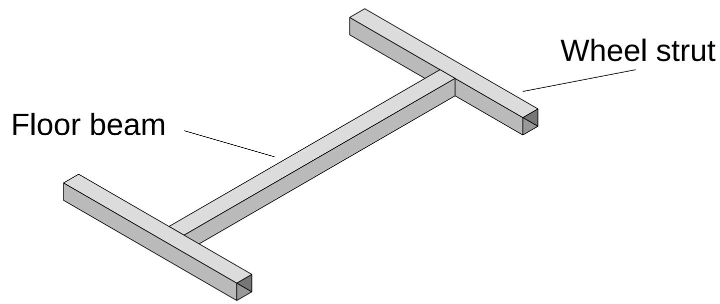


Figure 2: finished H frame.

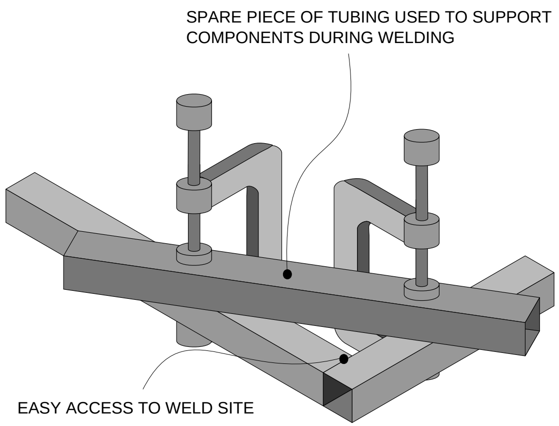


Figure 3: holding frame components during welding.

tubing so it sticks out about 20 mm beyond the surface of the planks, and then knock the ends over with a hammer so they lie on the surface of the wood.

You will see that there are no mitres or complicated angles or joints to cut when making the cart, so you save time. Also the exact lengths of the components are not very critical - again it saves a little time, but you will find that the carts look better if you take a little trouble to get things square and even etc and welding is easier with good square ends. It is much better to

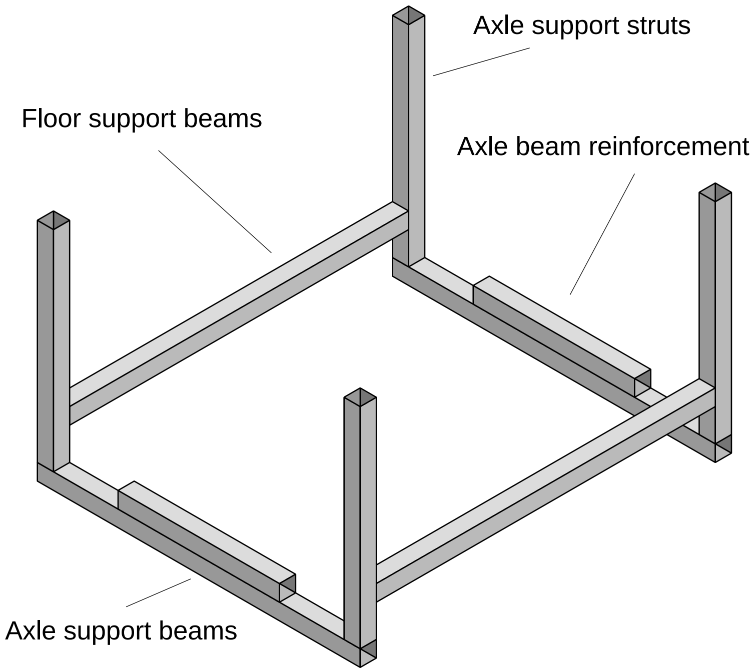


Figure 4: main frame assembly.

use a try square to mark the position of a cut than guess.

These carts have been tested in Nigeria, Kenya and Uganda and we have had only a few serious failures caused by poor welding or incorrect material. We think that they are strong enough, but we cannot be sure - somebody will always break anything.

Cutting list and costs

Table 1 shows a cutting list for a complete cart - Recent prices of materials in Kenya are shown converted into £$_{UK}$.

TABLE 1: 50×50 RHS vestigial frame donkey cart.

description	length m	#	total m	cost £$_{UK}$
50×50 RHS:			**11560.00**	27.71
floor beams 7 × 160 mm (planks 160 mm)	1120.00	2	2240.00	
wheel struts 3 × 160 + 25 + 60 + 240	805.00	4	3220.00	
axle support beams 1000 mm long	1000.00	2	2000.00	
draw pole	2700.00	1	2700.00	
draw pole upright extn	400.00	1	400.00	
draw pole reinforcement	1000.00	1	1000.00	
R8			**3900.00**	6.01
plank fixings each plank (13 off) takes 6	75.00	52		
R12			**1200.00**	0.33
yoke loop	400.00	1	400.00	
tie cleats for rope	200.00	4	800.00	
6"×1" timber			23.40	4.99
tray planks 13 off	1.80	13	23.40	
TOTAL COST =				39.04

Construction step by step

1) First get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete. Start by cutting the 50 × 50 box section steel into the right lengths, as in the cutting list, then cut the bottom and side planks. Lastly cut the 8 mm dia or whatever re-bar for the fixings ie the studs.

2) Next make up the two H-shaped frames shown in Figure 2. If you have a couple of G clamps you can use them to hold two pieces of the frame together during welding as shown in Figure 3. It's quick and you can tap the parts with a hammer until everything is square and straight and then weld.

3) Then stand the two H frames on the axle support beams as shown in Figure 4 and weld up the main frame. If you are using our PVC bearing system then weld on some axle beam reinforcement as shown in the figures but with the ball bearing and wooden bearing types you do not need this.

4) Next you can fit the side and the bottom planks to the

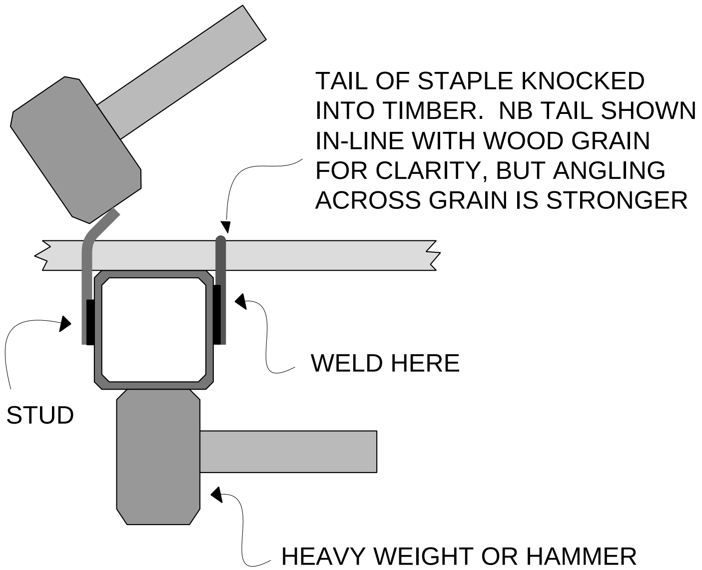


Figure 5: tightening welded stud.

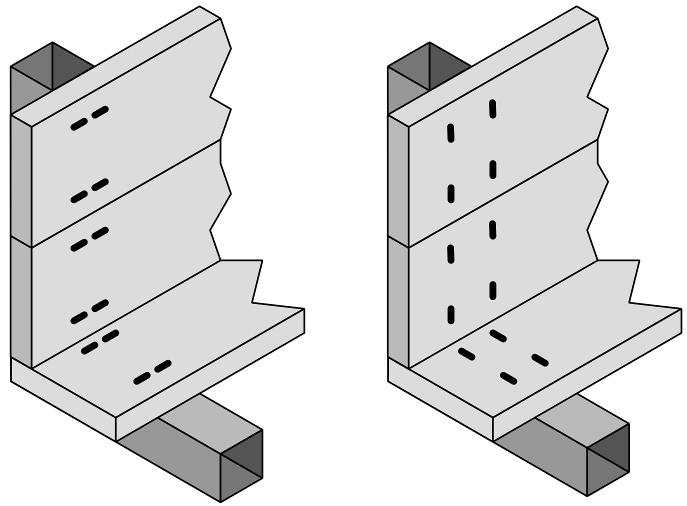


Figure 6: studs or staples bent in line with grain (left) or across it (right).

frames with studs. Studs are just short lengths of round bar welded to the sides of the box section as shown in Figure 5 (which also shows how these studs can be tightened with a hammer and a weight or another hammer).

Studs can either be put through holes in the planks or they can just be welded at the edge of each plank and then simply bent over the edge.

When you bend the end of the stud over you can either bend it in line with the grain of the wood or across the grain, as shown in Figure 6. Bending it in line as shown on the left lets it go into the wood nicely and looks neat, but bending it over across the grain gives a stronger joint.

5) Nearly there! Now you need to fix the draw pole. It is best to fix the draw pole to the body so it can be taken off and replaced if it gets damaged. A good way to do this is with short lengths of round bar welded on as shown in Figure 7. It is easy to cut through the re-bar hoops if you need to change the draw bar. You will need to use new re-bar of course when you put the new draw bar on. Also shown in Figure 7 is an easy way to reinforce the boom with an extra piece of steel tube welded to the main tube.

6) If you want to make it so that the ends of the load tray can be removed easily you can do so in the way we have shown in Figure 8. This is a good way because it is cheap and very easily repairable.

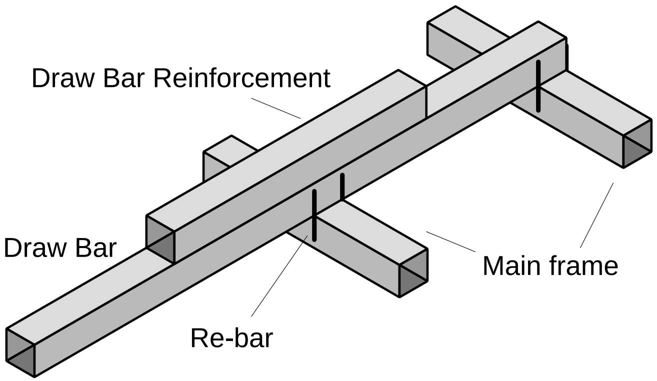


Figure 7: method of fixing draw bar to body. (View of cart upside down.)

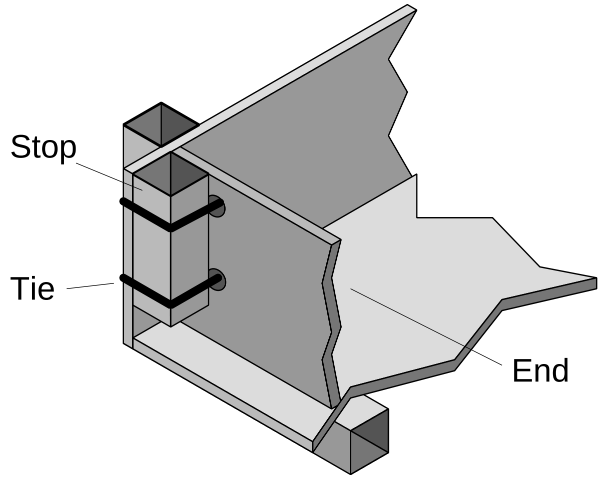


Figure 8: method of fixing tray ends with rubber or rope

7) Paint or creosote the cart. You've finished it!

Modifications

There are many different versions of this cart. You can try longer or shorter carts and you can make them wider or narrower. When you do this, check the length and width of the planks of wood that you will use - you do not want to find that you are two inches short of being able to get two runs of plank out of one piece of timber, or that its just too narrow and you have to fiddle about and fit in a narrow strip.

Other DTU cart developments

The DTU has been working on a range of carts for use with both donkeys and oxen. It has designs for wooden and steel framed bodies and for a range of wheels and axles. All steel framed carts can be fitted with a simple brake. The DTU also has designs for single and double donkey harness.

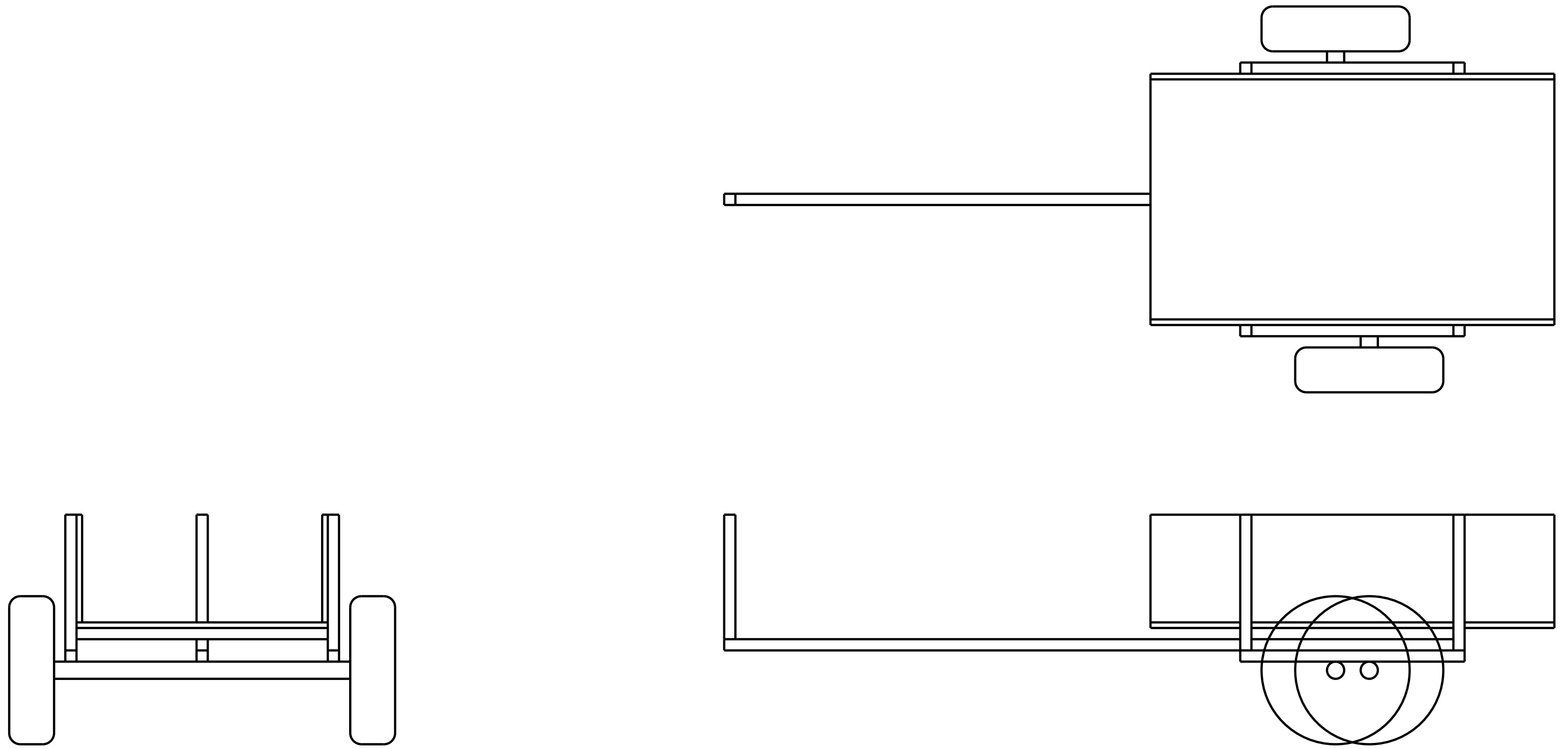
Cart Drawings

You will find two drawings on the next pages, the first one gives a general view of the cart, and the second gives a view of the main components. As we have said you can vary the size of the cart quite a bit and even make it much longer if you add extra frames. You could even make a four wheeled cart like this!

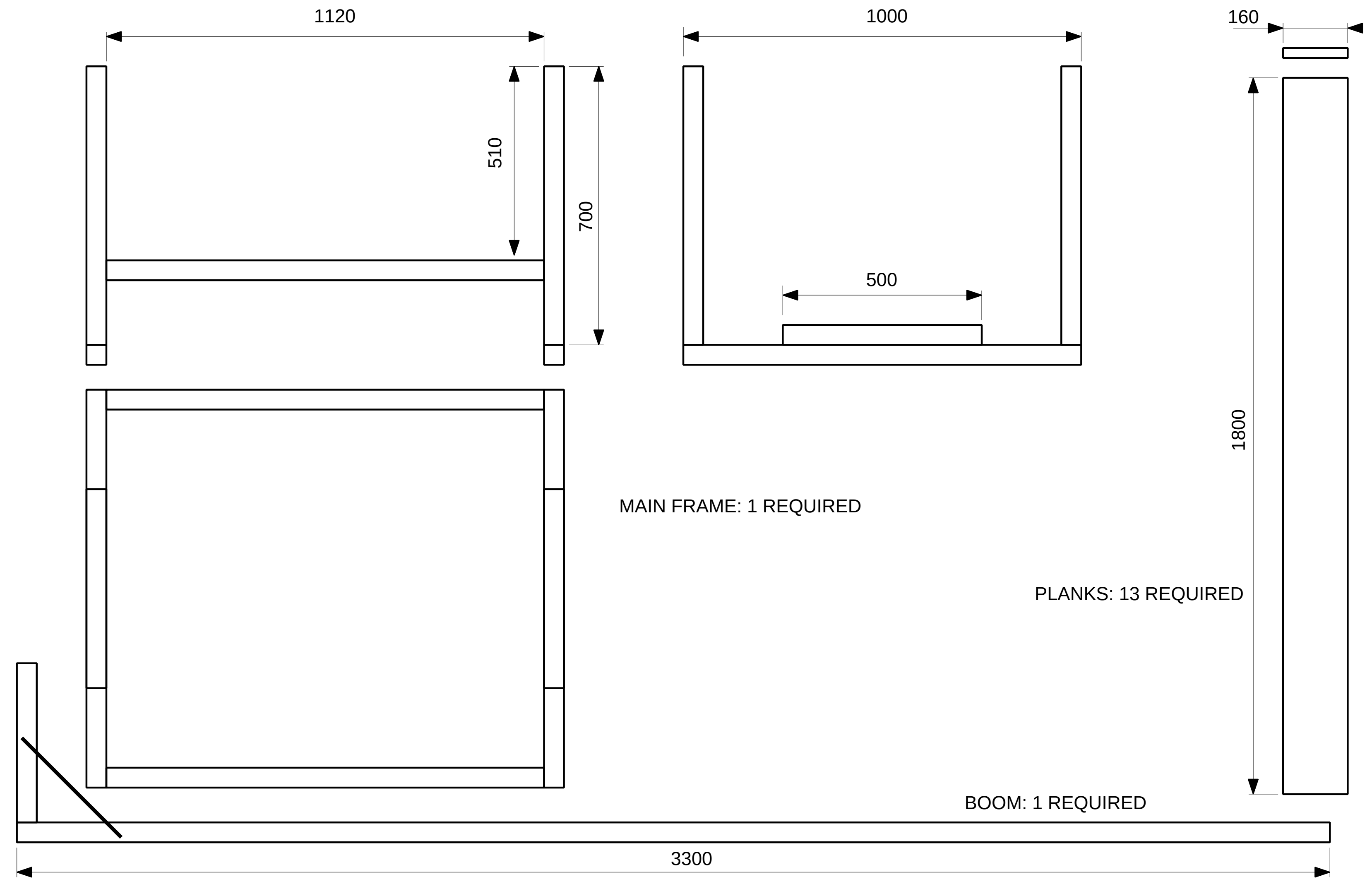
Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



					Scale	80 mm	Title STEEL & WOOD TWO DONKEY-CART	Drawn by	CEO
					Date	9/7/95		Dwg No.	1/2



					Scale	80 mm	Title COMPONENTS: STEEL & WOOD TWO DONKEY CART	Drawn by	CEO
					Date	25/8/97		Dwg No.	2/2

KENDAT

DTU

Animal Cart Programme

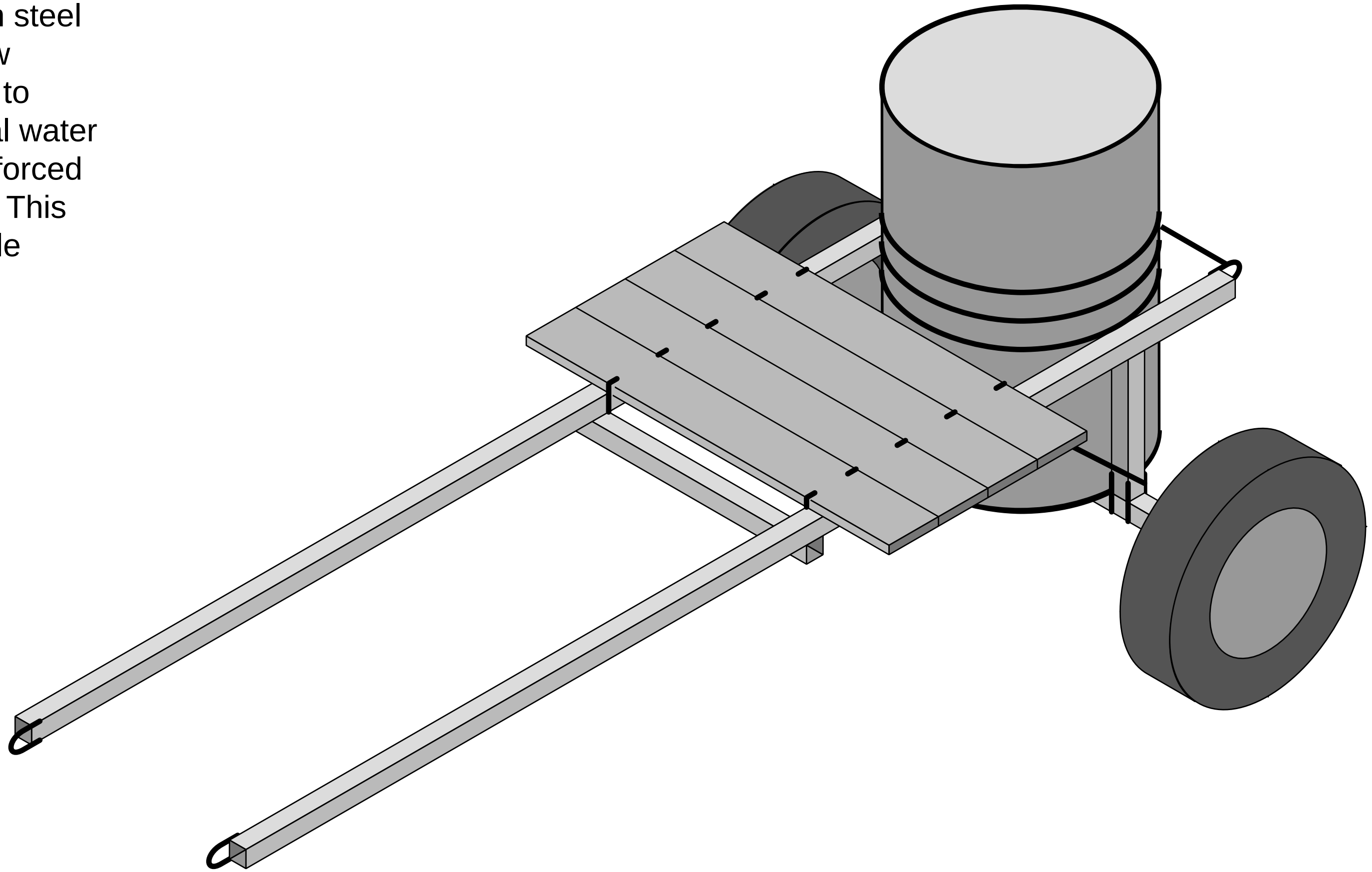
TECHNICAL
30
RELEASE

LIGHT SINGLE DRUM WATER CARRIER

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: donkey cart made from square box tubing.

This cart uses square section steel box tubing (RHS rolled hollow section) and a 200 litre drum to make a quick and economical water carrying cart. The cart is reinforced with concrete reinforcing bar. This cart has two shafts for a single donkey.



Lightweight water carrier cart for one donkey.

Introduction

In many towns and villages in Africa water is delivered commercially. You buy water from people selling it from carts. This cart design was developed in Kenya where water is widely sold like this and the cart is as simple and cheap as we can design it from new materials there.

This booklet tells you how to construct a water carrier cart - you will need to use another booklet Technical Release 28 to tell you how to make the axle. You should be able to make the cart make one cart including the axle per day.

In other booklets in this series we tell you how you can make other simple low-cost carts and axles

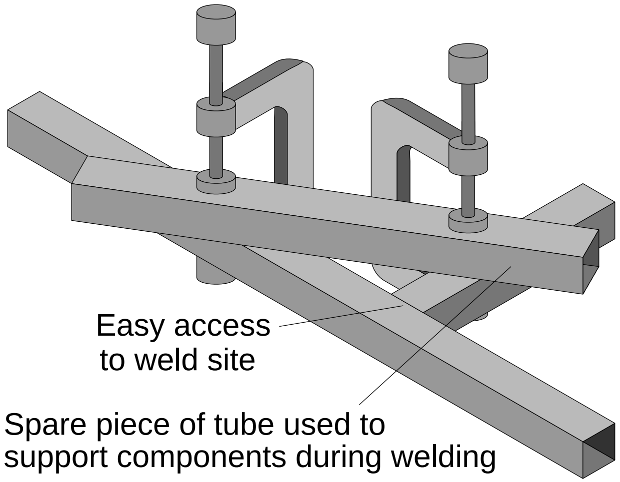


Figure 2: supporting components during welding.

Idea Behind Design

The idea behind the designs in these booklets is to allow construction without lots of special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a welder and a hacksaw. You might also find that a couple of 4" or a 5" G clamps (or something like it) are useful too. A wheelbrace or carpenters brace is also useful - you can make the drill bit yourself.

You will see that there are no mitres and unusual angles to cut in the square tubing so you save time when making the cart. Also the exact lengths of the components are not very fussy. But you will find that the carts look better if you take trouble to

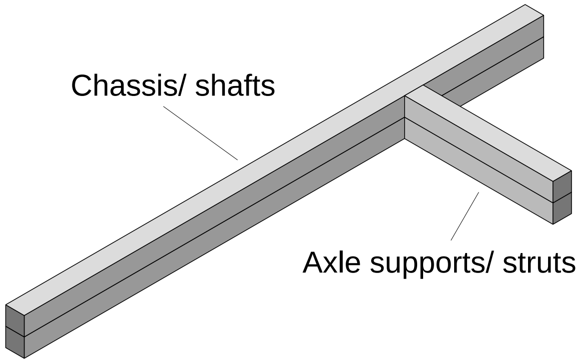


Figure 3: welding animal shaft and axle support strut assemblies.

get things square and straight.

Carts like these have been tested in Kenya. We have not had problems with them but if you find that the square tube breaks too easily you can reinforce it by welding on some round bar or concrete reinforcing bar 8mm, 10mm or 12mm. If you used tubing with a wall thickness of 2.0 mm or more you probably would not need to put these strengtheners on.

Cutting list and costs

Table 1 shows a cutting list for a complete cart. Recent prices of materials in Kenya are shown converted to £ UK. The

Table 1: materials and costs.

TABLE 1: cutting list.				
component	material	# lengths	total	materials
		& length	material	cost in
		reqd	in cart	Kenya
		[#*mm]		[£UK]
animal shafts	50×50 RHS	2×3000	6000	14.37
body cross pieces	50×50 RHS	2×700	1400	3.35
axle struts	50×50 RHS	2×375	750	1.80
shaft strengtheners	8mm to 12mm round bar[1]	4×600	2400	0.66
axle strut braces	8mm to 12mm round bar[1]	2×600	1200	0.33
axle fixing bars	8mm to 12mm round bar[1]	8×70	560	0.15
hitch/ drum tie loops	8mm to 12mm round bar[1]	2×250	500	0.14
TOTAL =				20.67

[1] The round bar can be anything actually - it doesn't even have to be round, so deformed or high-yield re-bar is fine. You could even use flat strip as long as its more than say 8mm thick.

method shown in Figure 2 is probably the easiest way to support the components during welding the first shaft and support. It's quick and if you do not tighten the clamps too tight to start with, you can tap the parts with a hammer until everything is square and straight. Then tighten the clamps before you weld.

Repeat the process using the first shaft and support as a pattern for the second as shown in Figure 3. Make sure you do not weld the two assemblies together!

3) Now you can weld the cross pieces under the animal shafts after you have checked for squareness etc.

4) Next weld the axle on using the method shown in Figure 4. Here small pieces of round bar eg 12 mm are welded across the join. If you do it like this you can easily remove the axle just by cutting the rod. This is much easier than grinding welds away. Alternatively you can bolt the axle on

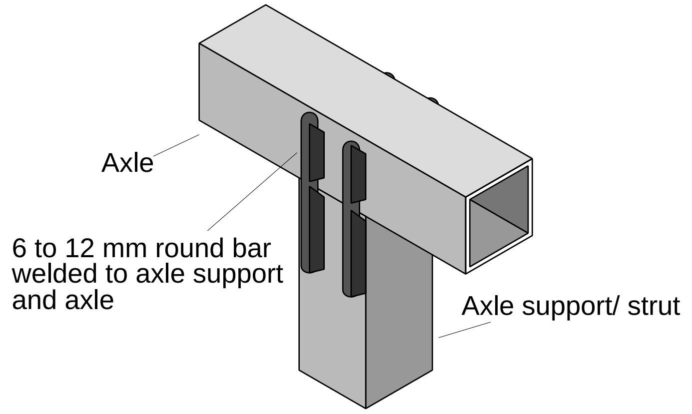


Figure 4: method of fixing axle to axle supports.

as shown in Figure 5.

5) Next you need to weld on the axle support strut braces to the square tube. Figure 5 shows the frame nearly ready.

6) Now fix the planks on using clenched over 8 mm re-bar as shown in Figure 6

7) Paint the cart. You've finished it!

Getting water out

Users usually cut a hole in the top of the drum or cut the whole top off to make it easy to pour water in quickly. To get water out some users in Kenya fix some 3 inch layflat hose onto a piece of pipe fixed to the larger threaded hole in one end of the drum. Tying the end up stops water coming out and letting it down squirts water out very quickly.

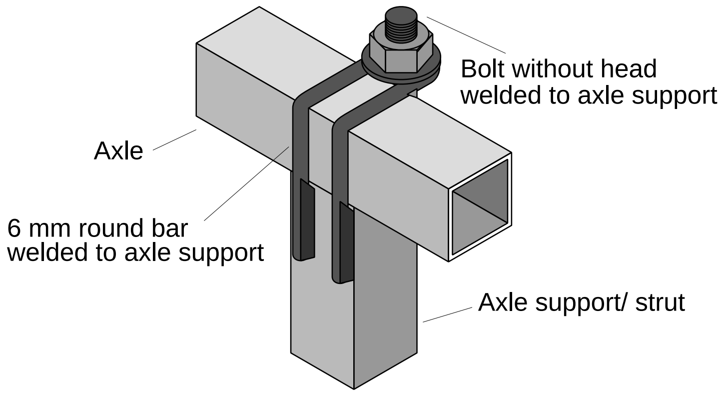


Figure 5: fixing planks to frame with clenched studs.

Modifications

This cart could be used as a flatbed cart by removing the drum and putting more planks on top of the shafts.

Other DTU cart developments

The DTU has been working on a range of cart body types for use with both donkeys and oxen. It has designs for both wooden and steel framed types. The wooden types are cheaper in material terms, but the steel framed ones are easier to make because the joints are more straightforward - but you can make either type of cart in only a day or two.

The DTU has also been working on new designs of wheels, hubs and bearings to bring down their costs and make things

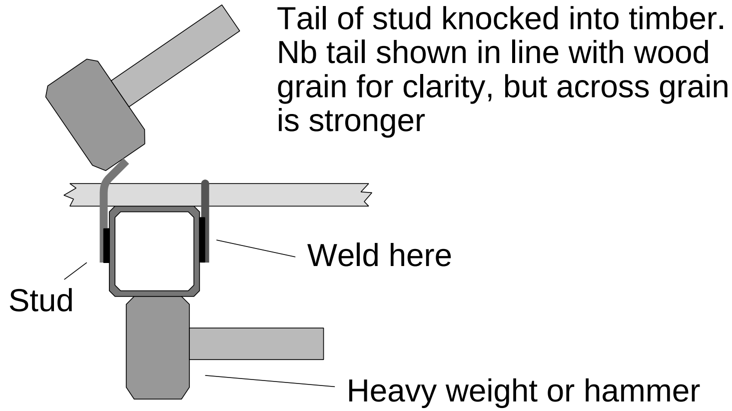


Figure 6: fixing planks to frame with clenched studs.

more locally manufacturable. We have developed easily made wooden bearings, bearings from PVC pipe, axles using old ball races and axles where you make your own roller bearings. Technical releases for all these are available.

Cart Drawings

The drawing for the cart is shown on the following page and the list of materials has been shown on a previous page.

Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

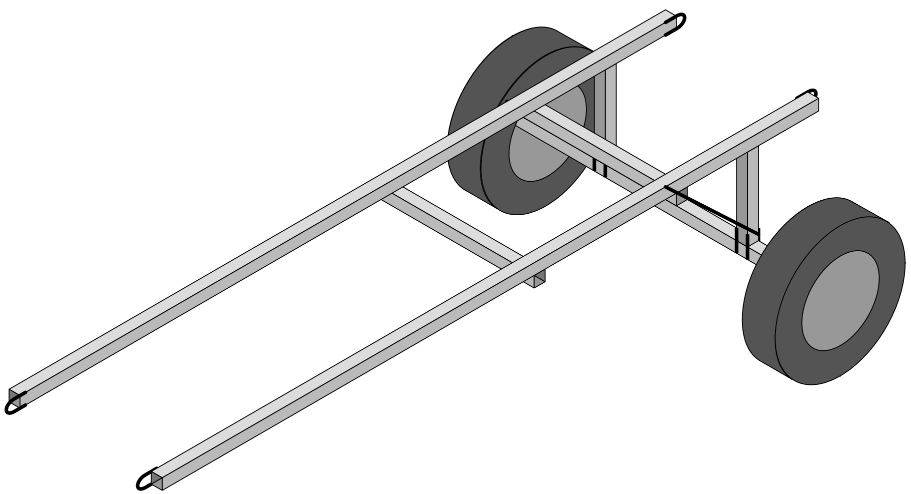
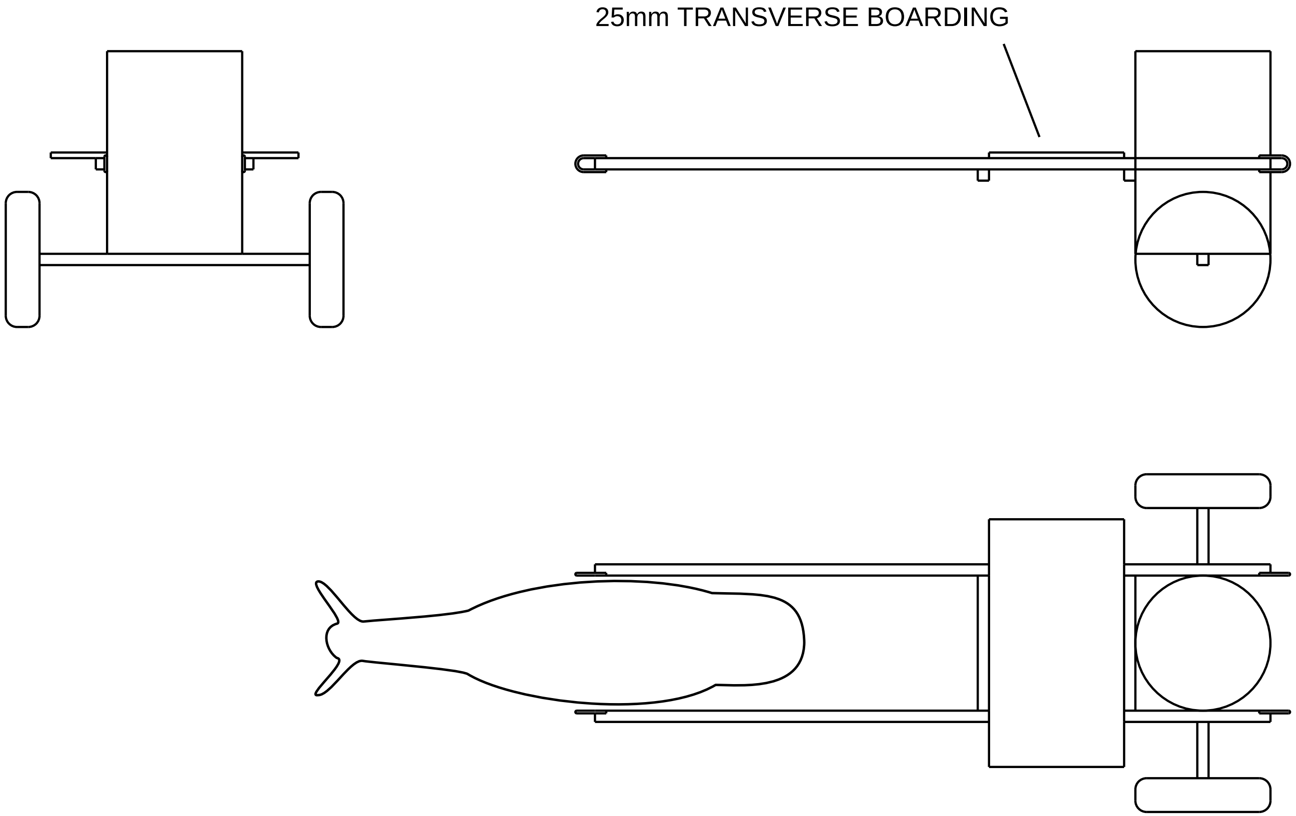
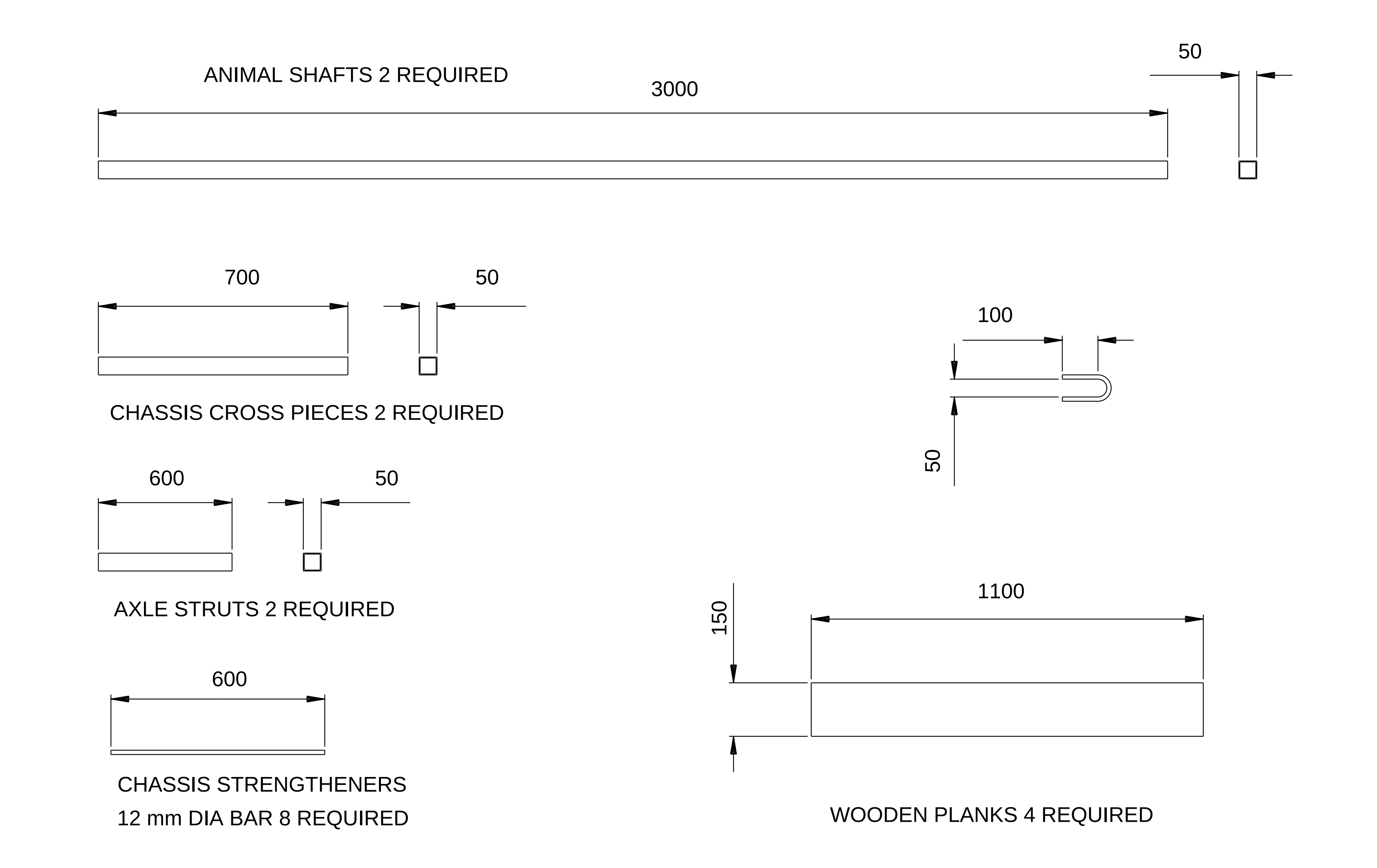


Figure 7: cart frame without drum and planks.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



Scale	200mm	Title WATER CART FOR SINGLE DONKEY	Drawn by	CEO
Date	13-7-93		Dwg No.	1/2



Scale	100mm	Title: WATER CART COMPONENTS	Drawn by	CEO
Date	8-3-99		Dwg No.	2/2

KENDAT

DTU

Low-Cost Animal Cart Programme

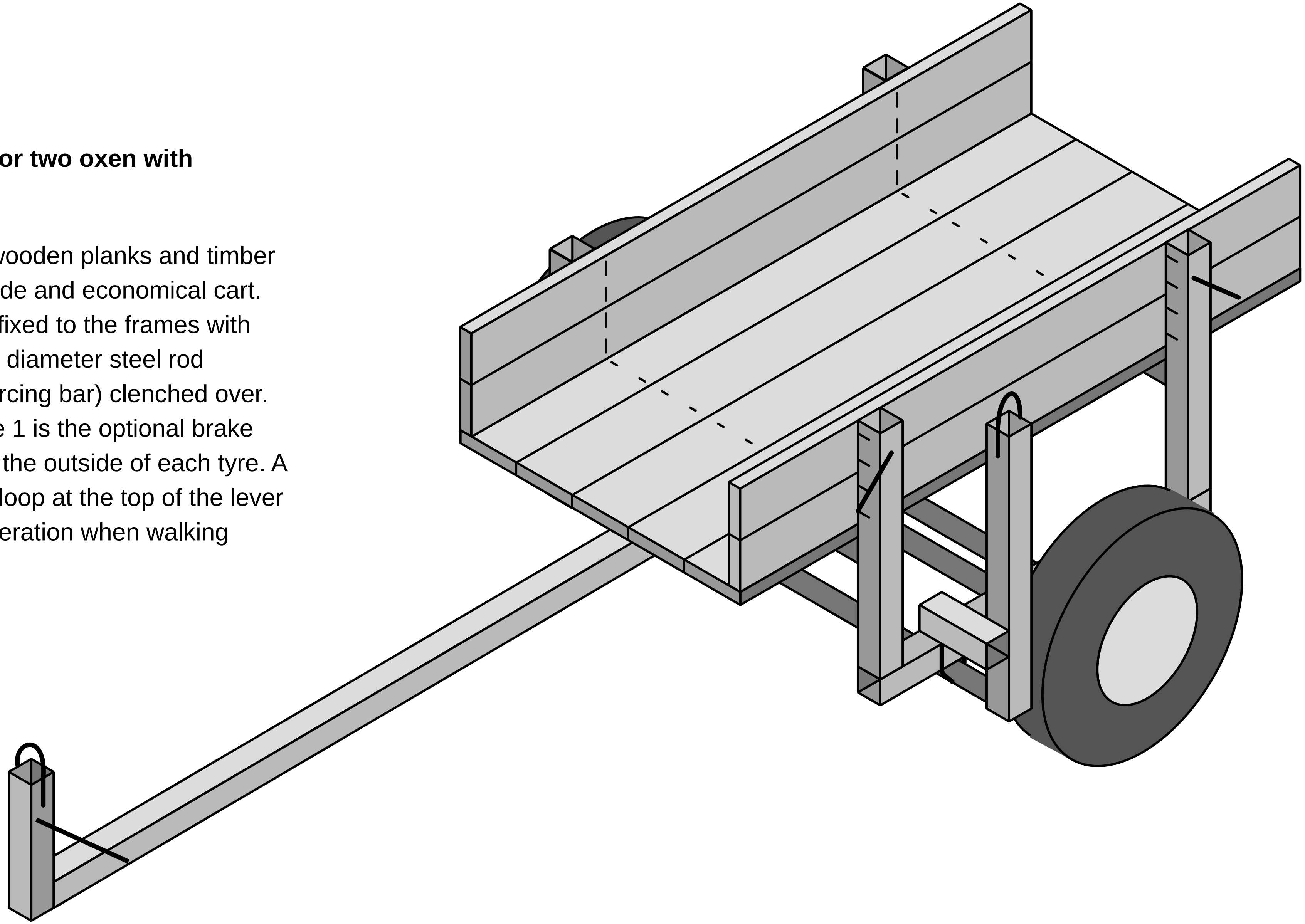
TECHNICAL
33
RELEASE

Low Cost Vestigial Steel Framed Cart for Two Oxen

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: cart for two oxen with brake.

This cart uses wooden planks and timber for a quickly made and economical cart. The planks are fixed to the frames with 8 mm or 10 mm diameter steel rod (concrete reinforcing bar) clenched over. Shown in Figure 1 is the optional brake which works on the outside of each tyre. A rope tied to the loop at the top of the lever allows brake operation when walking behind the cart.



Steel Box Tubing and Timber Cart for two Oxen.

Introduction

In this booklet we tell you how to make a simple cart from square steel tube and timber. This Technical Release does not tell you here how to make the axle - you will have to read another Technical Release to for this. We have designs for stub axles with PVC bearings and with needle roller bearings that you can make yourself and we have designs for twin offset axles using PVC, wood and scrap ball bearings. All axles can be made without machine tools - in fact you do not even need a drill!

This Technical Release describes an ox cart made from 60×60×3 square steel tubing and is suitable for oxen weighing 350 kg to 500 kg. If you have really strong animals then use 75×75×3 tube - its only a little bit more expensive.

You should find that you can make the 60×60×3 cart body for about £ 50 depending on the cost of the materials and labour. An axle plus wheels, tyres and tubes will cost another £ 60 - £ 70. Once you get organised, two men can probably make one cart in a day. We've designed these carts to be easy and quick to make.

Idea Behind Design

These carts are designed to be constructed without lots of special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a welder, a wood saw,

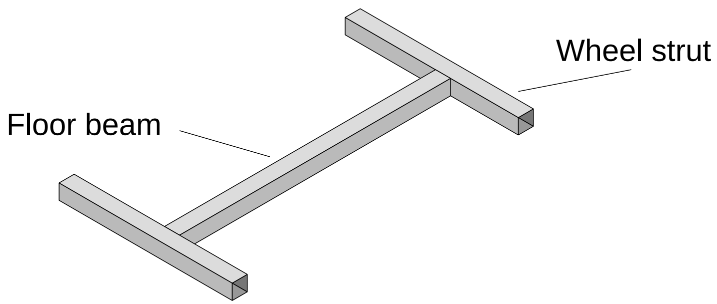


Figure 2: finished H frame.

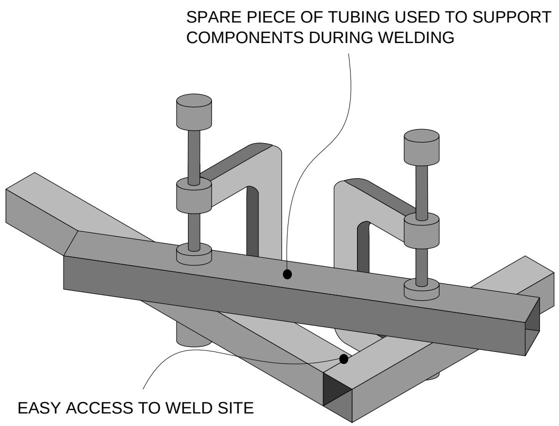


Figure 3: holding frame components during welding.

a hacksaw, and a hammer. You might find that a couple of 5" or a 6" G clamps (or something like it) are useful too.

The cart frames are fixed together by welding and the wooden planks are fixed to the frames with clenched steel bar. You weld 8 mm diameter re-bar (concrete reinforcing bar) to the steel box tubing so it sticks out about 20 mm beyond the surface of the planks, and then knock the ends over with a hammer so they lie on the surface of the wood.

You will see that there are no mitres or complicated angles or

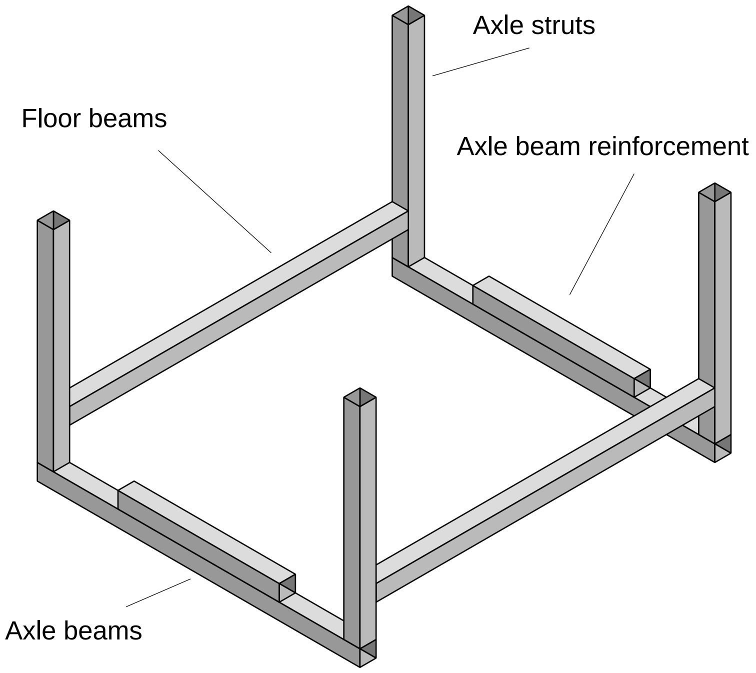


Figure 4: main frame assembly.

joints to cut when making the cart, so you save time. Also the exact lengths of the components are not very critical - again it saves a little time, but you will find that the carts look better if you take a little trouble to get things square and even etc and welding is easier with good square ends. It is much better to use a try square to mark the position of a cut than guess.

These carts have been tested in Kenya and Uganda and we have had only a few serious failures caused by poor welding or incorrect material. We think that they are strong enough, but we cannot be sure - somebody will always break anything.

TABLE 1: 60×60 RHS vestigial frame ox cart.

description	length mm	#	total mm	cost £UK
60×60 RHS:			11520.00	**36.67**
floor beams 7 × 160 mm (floor planks 160 mm)	1120.00	2	2240.00	
wheel struts 3 × 160 + 25 + 60 + 180	745.00	4	2980.00	
axle beams 1100 mm long	1100.00	2	2200.00	
draw pole	2700.00	1	2700.00	
draw pole upright extn	400.00	1	400.00	
draw pole reinforcement	1000.00	1	1000.00	
R8			5850.00	**0.90**
plank fixings each plank (13 off) takes 6	75.00	78	5850.00	
R12			1200.00	**0.33**
yoke loop	400.00	1	400.00	
tie cleats	200.00	4	800.00	
6"×1" timber			23.40	**4.99**
tray planks 13 off	1.80	13	23.40	
TOTAL =				**42.89**

Cutting list and costs

Table 1 shows a cutting list for a complete cart - Recent prices of materials in Kenya are shown converted into £$_{UK}$.

Construction step by step

1) First get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete. Start by cutting the 60 × 60 box section steel into the right lengths, as in the cutting list, then cut the bottom and side planks. Lastly cut the 8 mm dia or whatever re-bar for the fixings ie the studs.

2) Next make up the two H-shaped frames shown in Figure 2. If you have a couple of G clamps you can use them to hold two pieces of the frame together during welding as shown in Figure 3. It's quick and you can tap the parts with a hammer until everything is square and straight and then weld.

3) Then stand the two H frames on the axle beams as shown in Figure 4 and weld up the main frame.

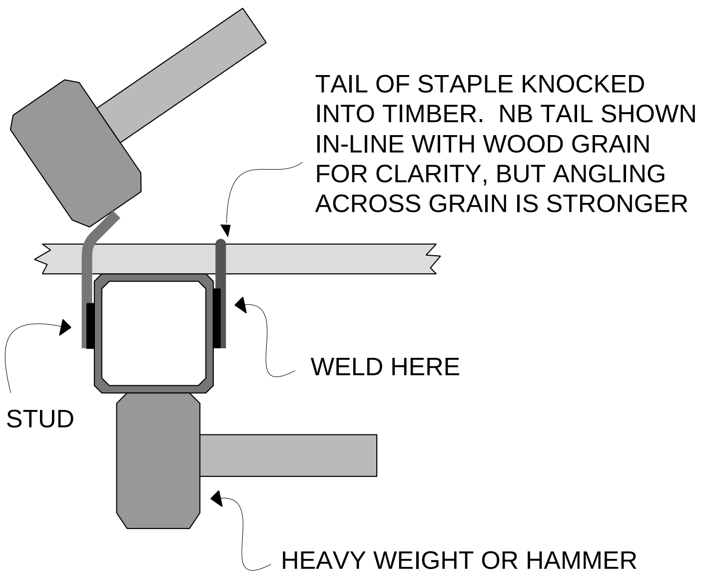


Figure 5: tightening welded stud.

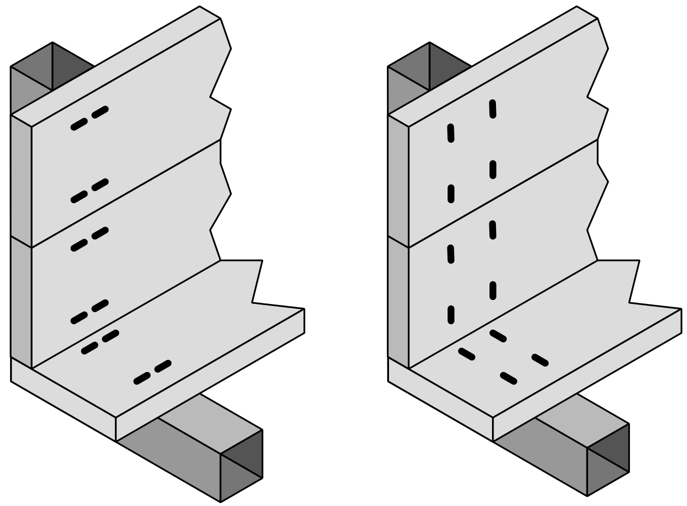


Figure 6: studs or staples bent in line with grain (left) or across it (right).

4) Next you can fit the side and the bottom planks to the frames with studs. Studs are just short lengths of round bar welded to the sides of the box section as shown in Figure 5 (which also shows how these studs can be tightened with a hammer and a weight or another hammer).

Studs can either be put through holes in the planks or they can just be welded at the edge of each plank and then simply bent over the edge.

When you bend the end of the stud over you can either bend it in line with the grain of the wood or across the grain, as shown in Figure 6. Bending it in line as shown on the left lets it go into the wood nicely and looks neat, but bending it over across the grain gives a stronger joint.

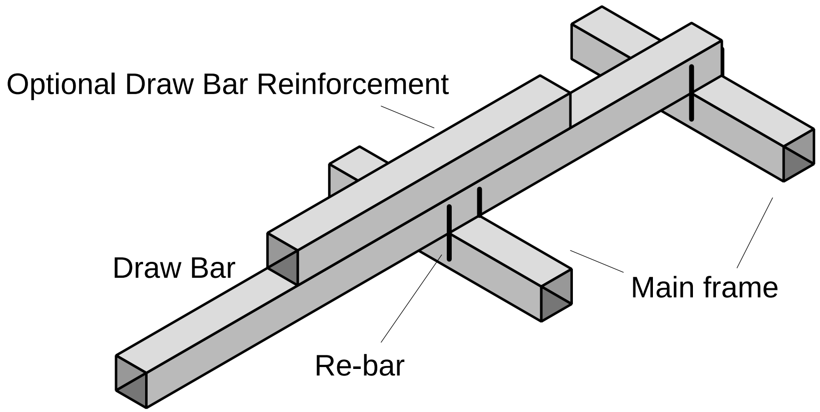


Figure 7: method of fixing draw bar to body. (View of cart upside down.)

5) Nearly there! Now you need to fix the draw pole. It is best to fix the draw pole to the body so it can be taken off and replaced if it gets damaged. A good way to do this is with short lengths of round bar welded on as shown in Figure 7. It is easy to cut through the re-bar hoops if you need to change the draw bar. You will need to use new re-bar of course when you put the new draw bar on. Also shown in Figure 7 is an easy way to reinforce the boom with an extra piece of steel tube welded to the main tube.

6) If you want to make it so that the ends of the load tray can be removed easily you can do so in the way we have shown in Figure 8. This is a good way because it is cheap

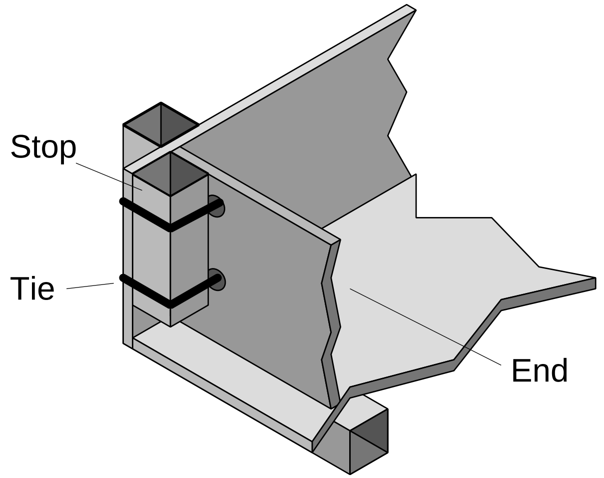


Figure 8: method of fixing tray ends with rubber or rope

and very easily repairable.

7) Paint or creosote the cart. You've finished it!

Modifications

There are many different versions of this cart. You can try longer or shorter carts and you can make them wider or narrower. When you do this, check the length and width of the planks of wood that you will use - you do not want to find that you are two inches short of being able to get two runs of plank out of one piece of timber, or that its just too narrow and you have to fiddle about and fit in a narrow strip.

Other DTU cart developments

The DTU has been working on a range of carts for use with both donkeys and oxen. It has designs for wooden and steel framed bodies and for a range of wheels and axles. All steel framed carts can be fitted with a simple brake. The DTU also has designs for single and double donkey harness.

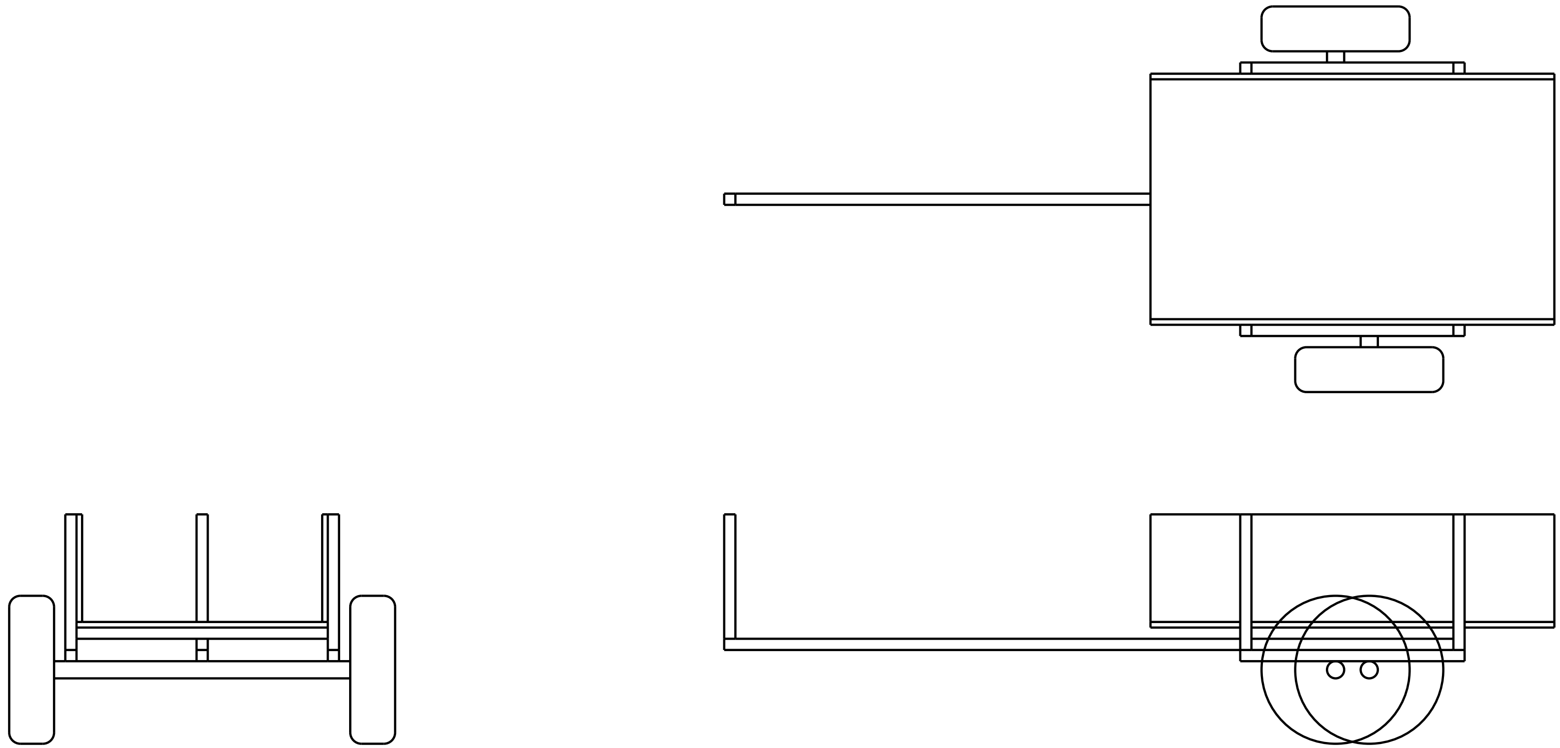
Cart Drawings

You will find two drawings on the next pages, the first one gives a general view of the cart, and the second gives a view of the main components. As we have said you can vary the size of the cart quite a bit and even make it much longer if you add extra frames. You could even make a four wheeled cart like this!

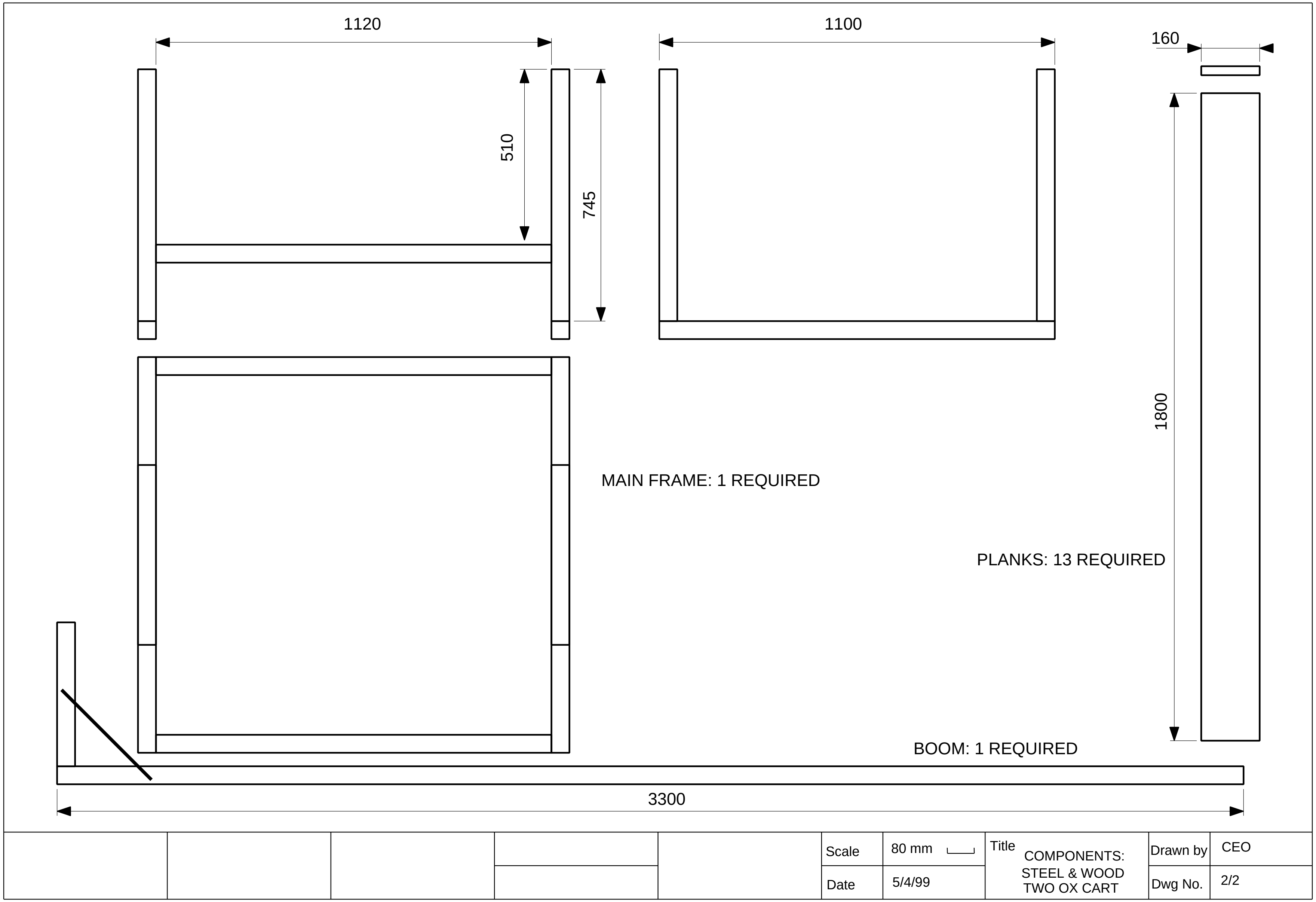
Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



Scale	80 mm	Title STEEL & WOOD TWO DONKEY-CART	Drawn by	CEO
Date	9/7/95		Dwg No.	1/2


1120
1100
160
510
745
1800
MAIN FRAME: 1 REQUIRED
PLANKS: 13 REQUIRED
BOOM: 1 REQUIRED
3300
Scale
80 mm
Date
5/4/99
Title
COMPONENTS:
STEEL & WOOD
TWO OX CART
Drawn by
CEO
Dwg No.
2/2

KENDAT

DTU

Animal Cart Programme

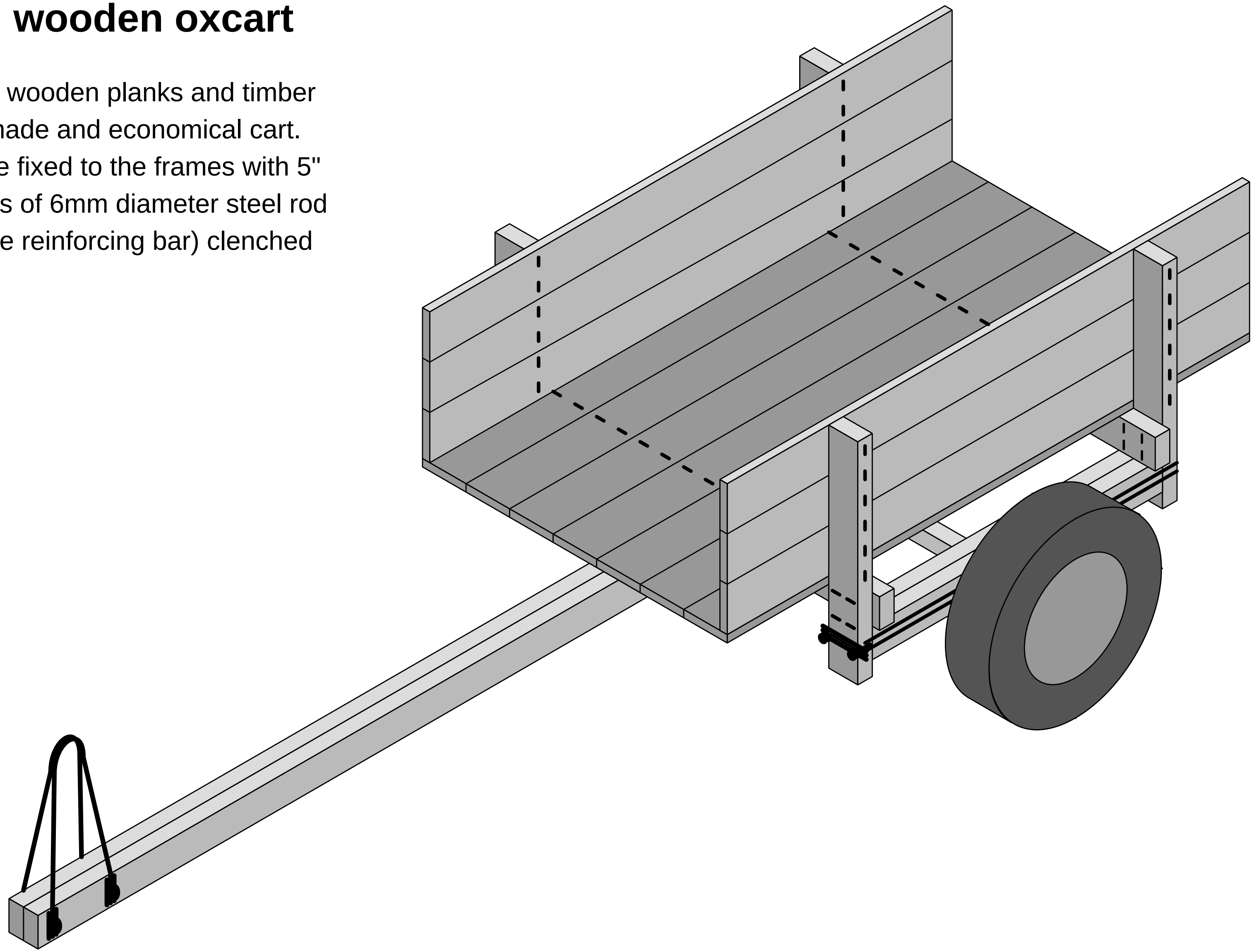
TECHNICAL
34
RELEASE

VESTIGIAL WOODEN FRAME 2-OX CART

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@uk.ac.warwick.eng
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: wooden oxcart

This cart uses wooden planks and timber for a quickly made and economical cart. The planks are fixed to the frames with 5" nails or lengths of 6mm diameter steel rod (small concrete reinforcing bar) clenched over.



Ox Cart Body Made From Timber

Introduction

This Technical Release tells you how to make a low-cost wooden cart to be pulled by two oxen. It does not cover the construction of the axle. You can use either a scrap axle from a pick-up truck or car or you can make one. We can supply a number of Technical Releases which cover different axle designs.

We have designs for fixed axles with PVC plastic bearings or roller bearings you can make yourself if you are a good welder/ fabricator. And we have double axle systems using wooden bearings, PVC bearings and ball bearings. These ball bearings can be new or scrap and no machining is needed.

You should find that you can make the cart body for about £ 30 depending on the cost of the materials and labour. Once you get organised, two men can make a cart body in one day. This is quite a lot faster than most carts can be made and it follows from the simplifications which we have made to the design. We've designed it to be easy to make.

Idea Behind Design

We have designed this cart to be easy to make without lots of special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a woodsaw, a hacksaw or cold chisel and a hammer and for some of the components

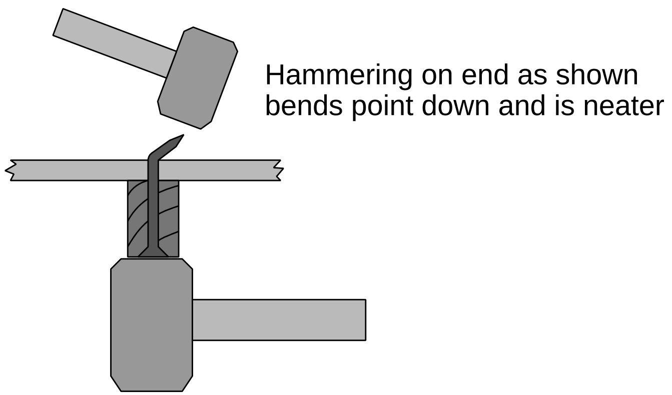


Figure 2: using two hammers to clench nail.

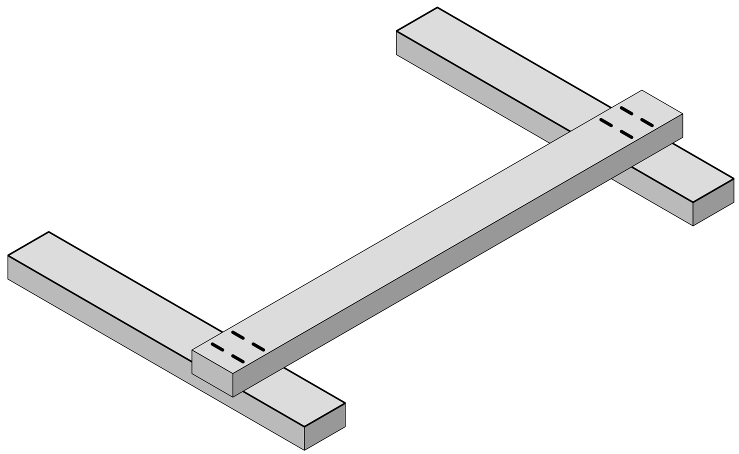


Figure 3: finished U-frame.

shown below you must be able to **weld** - there are alternative ways of making these but they are not as strong. In any case if you make an axle you will have to weld. You might find that a couple of 4" or a 5" G clamps (or something like it) are useful too. For drilling wood we have used flatbits sometimes (Figure 11). We have a Technical Release which describes how to make them.

Most of this cart is joined together using clench nailing which is a bit like riveting. You use nails which are about an inch (25mm) longer than the total thickness of the wood. You probably need to put a piece of scrap timber under the timber

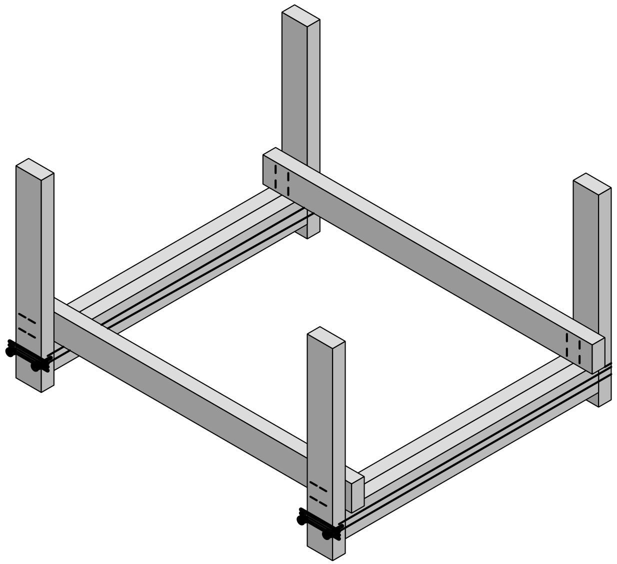


Figure 4: finished framework before planking.

you are working on to stop the nails going into the floor. When you have put all the nails in you put a hammer against the head of each nail in turn and then knock the pointed end over as shown in figure 2. Knock the end of the nail into the timber to make it neat.

If you do not have any nails you can use 6 mm round bar. We tell you how to make joints with 6mm bar at the end of this booklet.

You will see that there are no mitres and complicated angles to

TABLE 1: cutting list for wooden oxcart.				
component	material	# lengths	total	materials
		& length	material	cost in
		reqd	in cart	Kenya
		[#*mm]	[mm]	[£]
animal shaft/ boom	100×50 r/s timber or bush pole	2×3400	6800	2.23
body frame bottoms	100×50 roughsawn timber	2×1100	2200	0.72
body frame sides	100×50 roughsawn timber	4×725	2900	0.95
axle beams	100×50 roughsawn timber	4×1000	4000	1.31
axle chocks	100×50 roughsawn timber	4×500	2000	0.66
tray planks	25×150 or similar timber	13×1800	23400	5.76
tray ends planks	25×150 or similar timber	6×1000	6000	1.48
plank fixing nails/ rivets	6" nails/ 8mm dia re-bar	13×4	6.50	3.25
body frame nails/ rivets	6" nails/ 8mm dia re-bar	4×4	2.00	1.00
axle beam ties	8-12mm dia re-bar or similar	4×2400	9600	1.76
axle fixing loops	12mm dia re-bar or similar	2×400	800	0.23
draught pole bolts	M12 bolts	4×50	4	1.04
draught pole bolt extns	12mm dia re-bar or similar	2×520	1040	0.30
draught pole vertical extn	12mm dia re-bar or similar	2×900	1800	0.53
dr. pole vertical extn bolts	M12 bolts	2×75	2	0.52
d. pole vert extn bolt extns	12mm dia re-bar or similar	2×120	240	0.07
TOTAL->				21.81

cut in the timber so you save time when making the cart. Also the exact lengths of the components are not very critical - again it saves a little time, but you will find that the carts look better if you take a little trouble to get things square and even etc.

We have tested these carts in Kenya and Uganda and we think that they are strong enough, but we cannot be sure. So treat the cart carefully at first!

Construction step by step

Table 1 shows a cutting list for a complete cart - Recent prices of materials in Kenya are shown converted into £UK.

1) The first job, is to get all the materials together and clear a space to work. Ideally you will be able to work on a flat

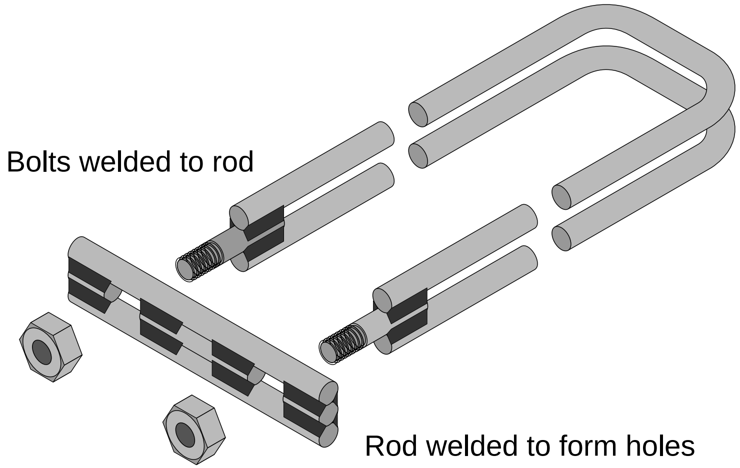


Figure 5: U-bolts for fixing axle beams to U frames.

area of concrete. Start by cutting the 100×50 timber into the right lengths, as in the cutting list, and then you can cut the bottom and side planks. Because timber comes in such variable sizes its best not to cut the steel components until you are sure how big they must be.

2) Next make up the two U-shaped front and back frames like those shown in figure 3. If you have a G clamp you can use it to hold two pieces of the frame together during nailing and clenching. It's quick and you can tap the bits with a hammer until everything is square and straight.

3) Next fit the axle beams in between the two U frames to make the frame shown in figure 4. The best way to join the U frames and the axle beams together strongly is to make up some big U bolts like those shown in Figure 5. These are made from 8 to 12mm round bar and some bolts. You will see from the drawing that they are made by welding rod together, not drilling holes.

If you cannot get these U-bolts made, use some 6 mm or

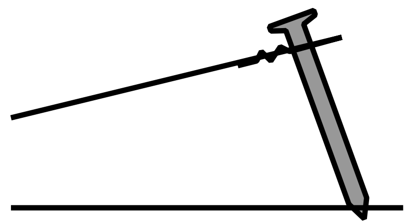


Figure 6: tightening wire by angled nailing.

8 mm round steel bar to tie the frames and beams together. Be very careful if you twist the ends together because some wire breaks quite easily. You can tighten the wire by nailing at an angle as shown in Figure 6 it. You can also tighten the wire slightly after nailing by kinking it.

4) Next you can fit the side and the bottom planks with more clenched nails.

5) Now you need to fix the draught pole. The best way is to use a big U-bolt to fix it to each U-frame. Do not drill holes in the U-frames or the draught pole for these bolts but arrange them as shown in Figure 7. If you cannot get U-bolts made up you will need to fix the draught pole with wood clenched nails and wire.

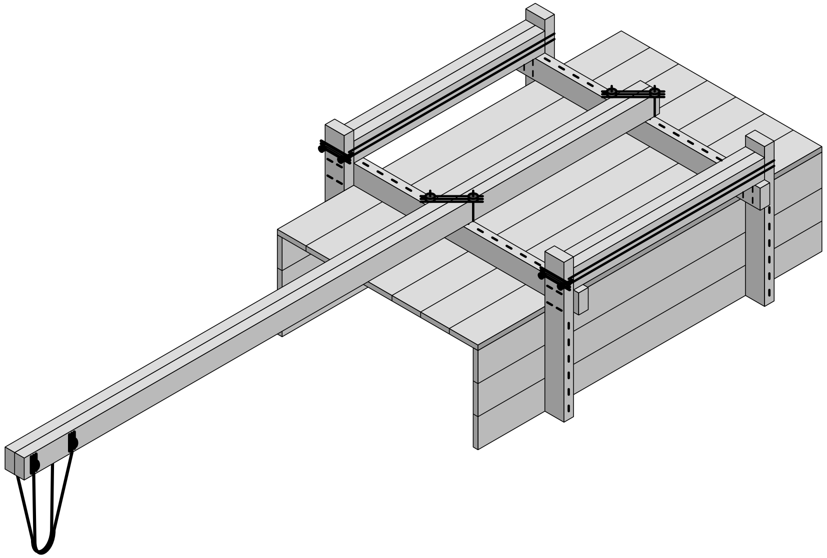


Figure 7: method of fixing draught pole using U-bolts.

6) The draught pole vertical extension is made from 12mm round bar. To fix it you will need to make long bolts as in Figure 8 from round bar and M12 bolts.

7) Nearly there! Fix the axle with more U-bolts around the axle and the axle beams as shown in Figure 9. If you are using a scrap vehicle axle you may need to weld extra pieces of steel to it to make it fit properly to the cart axle beams. Whichever axle you use you will find it stronger to nail pieces of wood to the bottom of the axle beams to stop the axle sliding along it.

8) Removable ends for the load tray can easily be provided in the way we have shown in Figure 10. The end is tied against the stop with rope or rubber. (Notice that there is a gap between the stop and the load tray so that the tray can be cleaned easily.

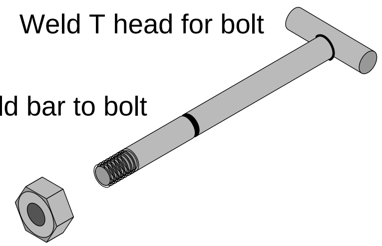


Figure 8: method of fixing draught pole using U-bolts.

9) Paint or creosote the cart. You've finished it!

Modifications

There are many different versions of this cart. You can try longer or shorter carts and you can make them wider or narrower. When you change things check the length and width of the planks of wood that you will use - you do not want to find that you are two inches short of being able to get two runs of plank out of one piece of timber, or that its just too narrow and you have to fiddle about and fit in a narrow strip.

Another modification is to make the cart higher see Figure 11. This lifts the cart body and raises the draught pole reducing the required size of the vertical extension.

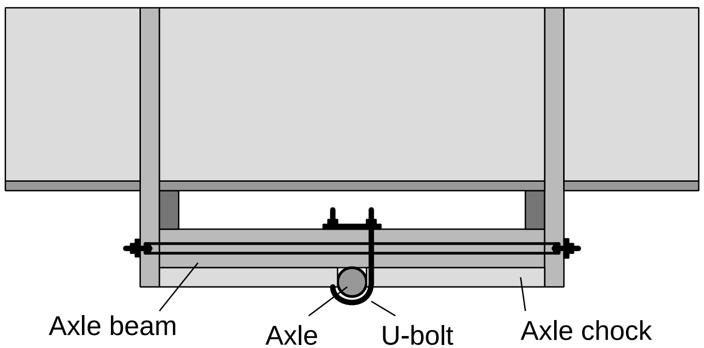


Figure 9: chocking axle to axle beam.

Using 6mm bar instead of nails

You can use 6 mm round bar instead of nails. You will need 16 pieces 150 mm long for the frames and 52 pieces 175 mm long to fix the planks on. You will need to drill a 6mm hole in the wood to use this method. Then put a straight piece of 6mm diameter re-bar (concrete reinforcing bar) right through so it sticks out about 25mm both sides. Then you just knock one end over with a hammer so it lies on the surface of the wood. Next you bend the other end over. Tighten the joint using a second hammer as described above.

You can make the drill yourself if you have to - see our Technical Release on flat bits.

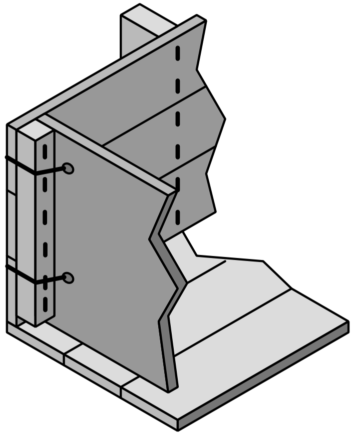


Figure 10: method of fixing tray ends with rubber or rope

Other DTU cart developments

The DTU has been working on a range of cart body types for use with both donkeys and oxen. It has several designs for steel framed types. The wooden types are cheaper in material terms, but the steel framed ones are easier to make because the joints are more straightforward - nevertheless you can make either type of cart in only a day or so if you are reasonably set up with tools and materials.

The DTU has also been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. It has designs for wooden and PVC plain bearings and for using scrap or new ball bearings - without needing machining, It also has a system of needle roller bearings which you can make without machining.

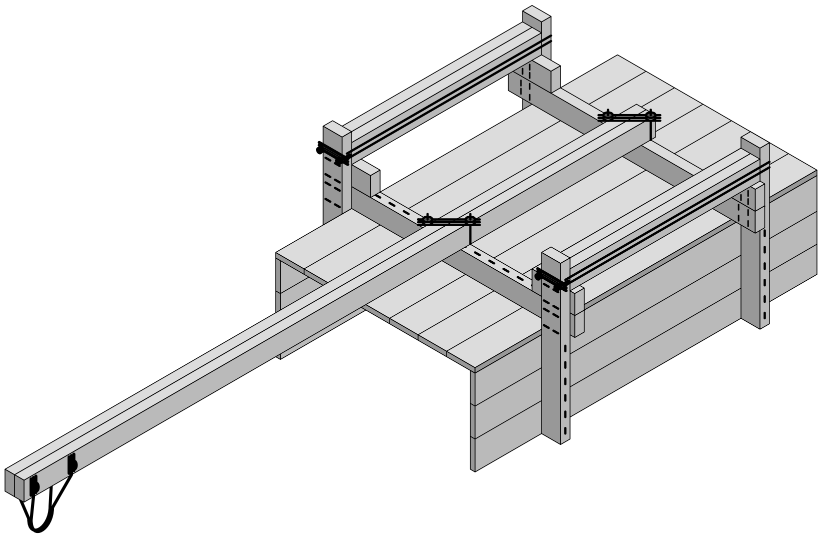


Figure 11: raised cart

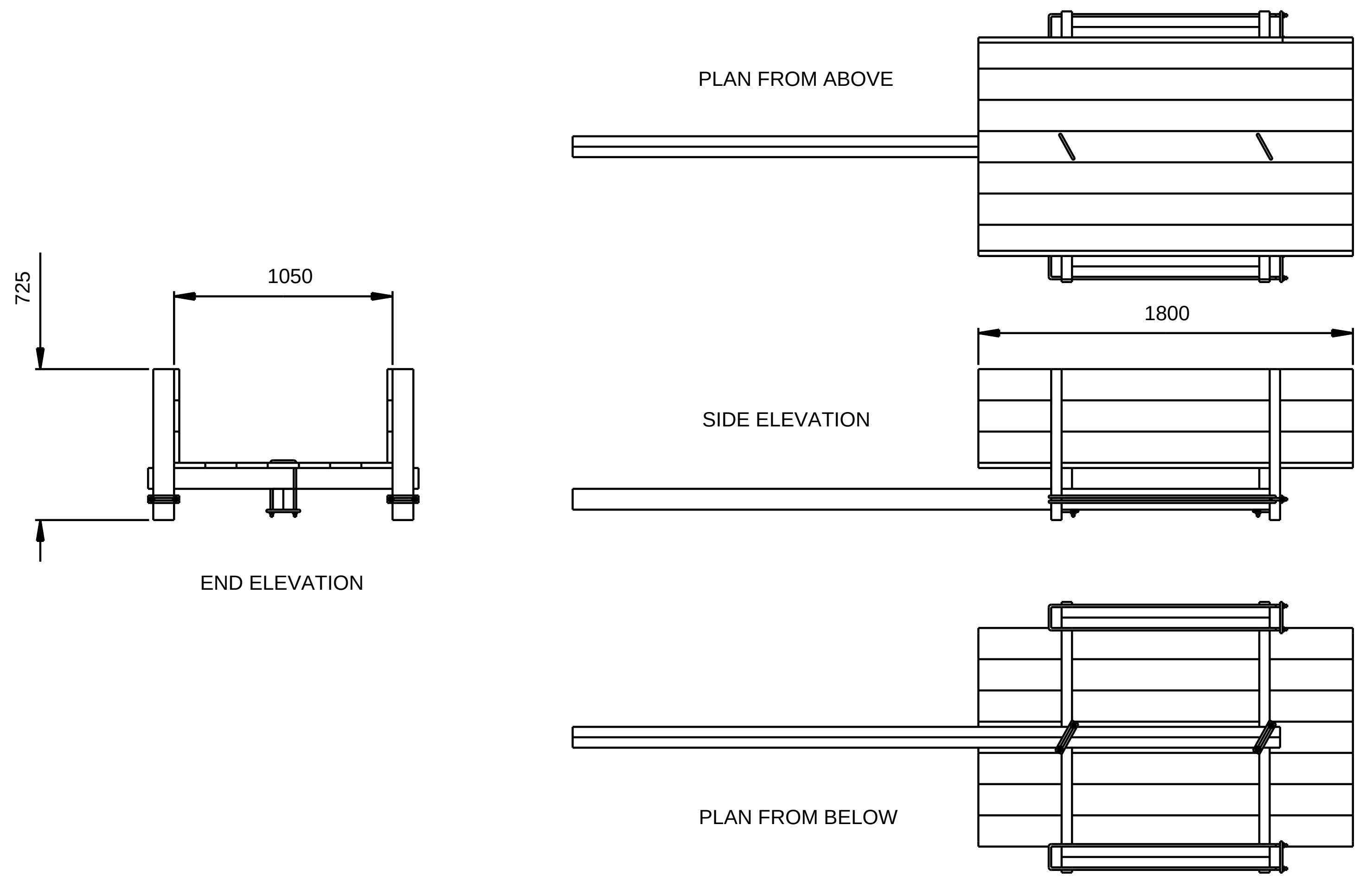
Cart Drawings

You will find two drawings below, the first one gives a general view of the cart and the second, a view of the main components. As we have said you can vary the size of the cart quite a bit and even make it much longer if you add extra frames. You could even make a four wheeled cart like this!

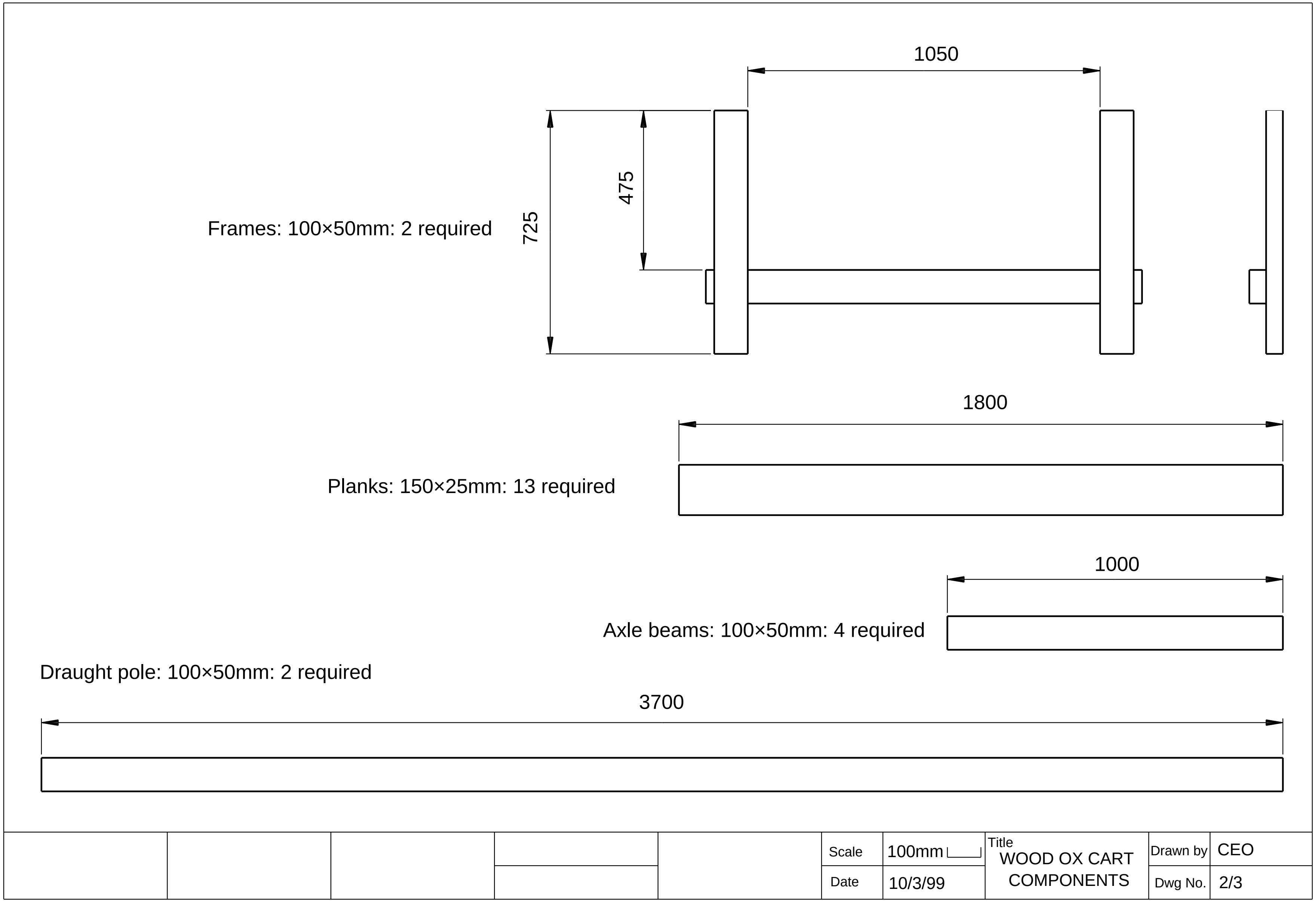
Acknowledgements

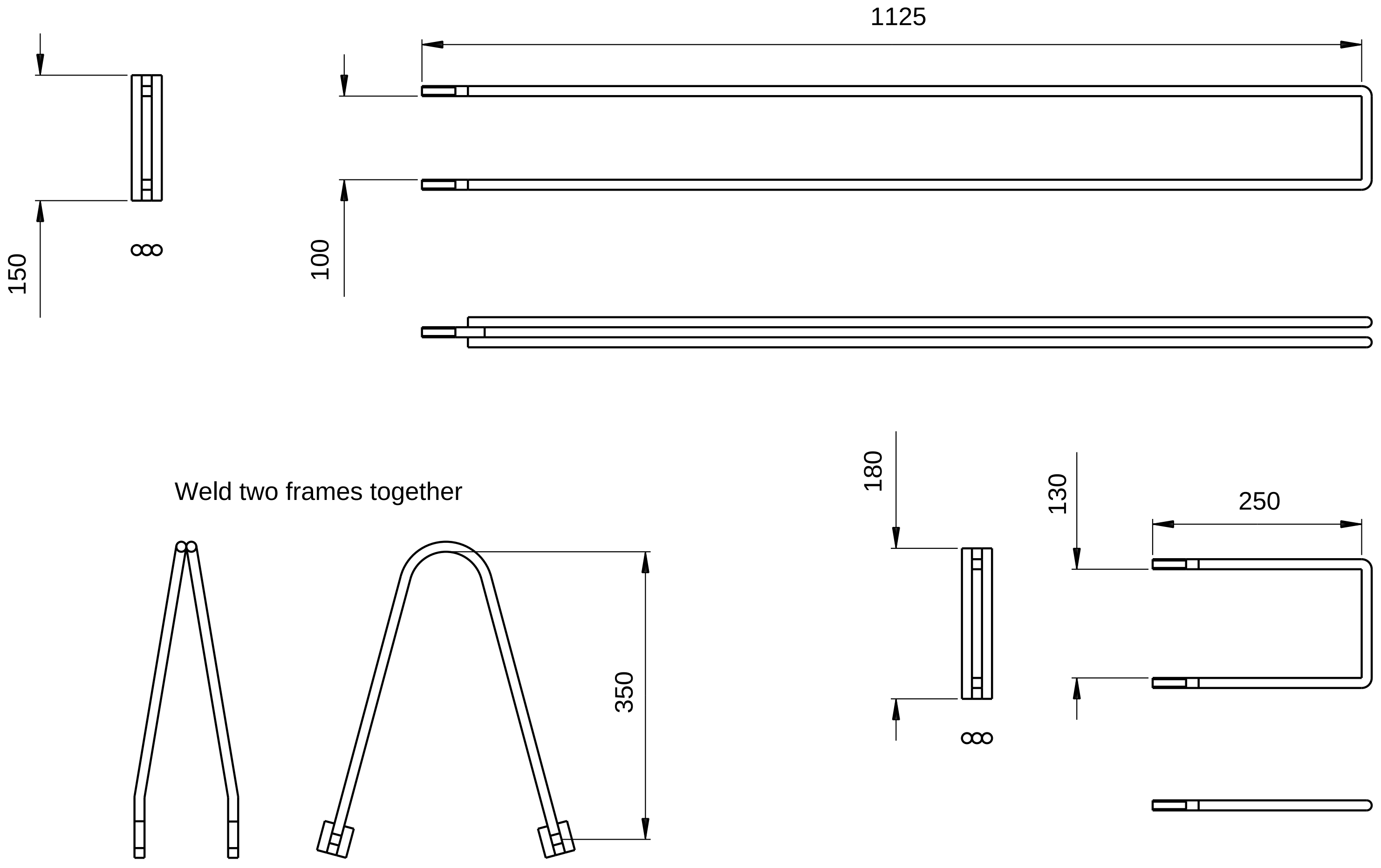
The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



					Scale	200mm	Title VESTIGIAL FRAME WOOD OX CART	Drawn by	CEO
					Date	10/3/99		Dwg No.	1/3


1050
475
725
Frames: 100×50mm: 2 required
1800
Planks: 150×25mm: 13 required
1000
Axle beams: 100×50mm: 4 required
Draught pole: 100×50mm: 2 required
3700
Scale
100mm
Date
10/3/99
Title
WOOD OX CART
COMPONENTS
Drawn by
CEO
Dwg No.
2/3



					Scale	100mm	Title STEEL OX CART COMPONENTS	Drawn by	CEO
					Date	10/3/99		Dwg No.	3/3

KENDAT

DTU

Animal Cart Programme

TECHNICAL
35
RELEASE

Making a flatbit for drilling holes in wood

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@uk.ac.warwick.eng
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Making a Flatbit.

This type of drill bit can be made in a few minutes and will drill holes well in wood and soft metal such as copper and aluminium. If it is hardened it will work in steel as well.

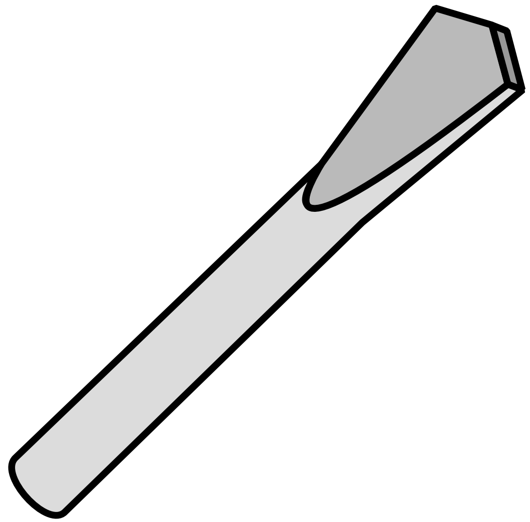


Figure 1: flat bit for drilling holes in wood.

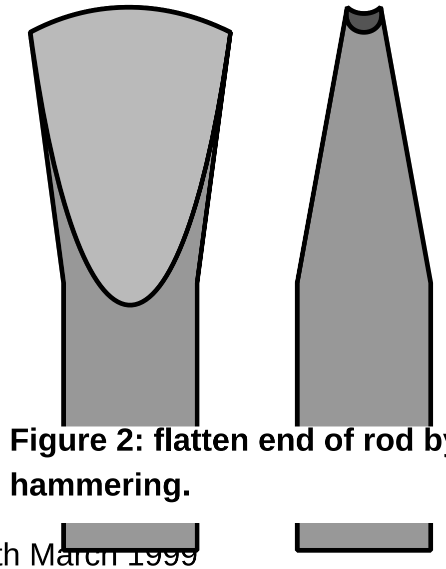


Figure 2: flatten end of rod by hammering.

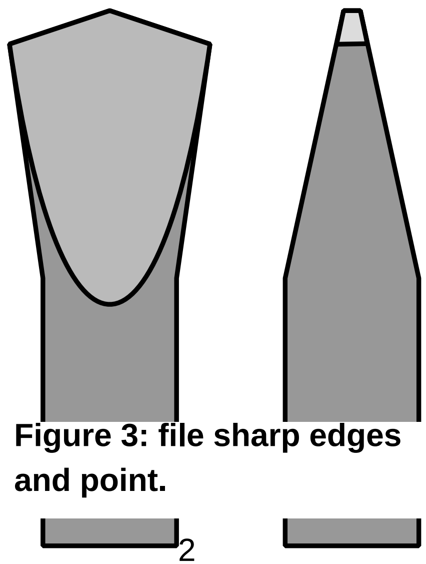


Figure 3: file sharp edges and point.

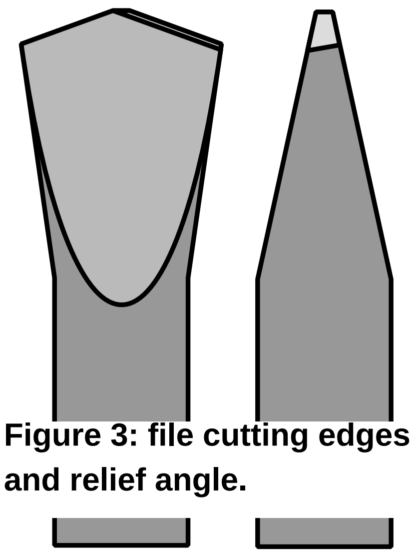


Figure 3: file cutting edges and relief angle.

Making a flatbit

Flatbits for drilling wood are easy to make and quite useful because you can make them very long and drill holes in quite awkward places. Of course you do need something to turn the bit - like a wheelbrace or a 'drill'. You can use a bent piece of steel like that shown in Figure 2 and weld the flatbit to the end of the rod. To make it comfortable to push you could make a handle from a piece of pipe with an end on it or by cutting it and bending a bit of the pipe over. Another way is by winding a spring of say 6mm round bar around the rod and then bending the end of the wire over as we have shown.

If you can harden the cutting edges of a flatbit you can use it to drill holes in metal too as long as you do not want to drill deep holes - it tends to wander if the depth is more than the diameter.

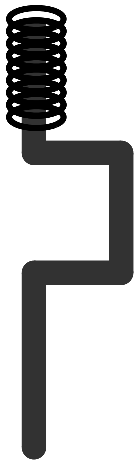


Figure 2: using bent steel rod to rotate drill.

To get hard cutting edges you will need to use 'silver steel' or spring steel, which you will have to harden and temper if you can. To get it very hard, heat until bright red/ orange hot and plunge into water. It will then be very hard and brittle so if you can you need to reheat slowly at a point two inches from the tip. You will see the steel take on a range of colours and just when the cutting edges look yellow or straw coloured, re-plunge into water. This is a fairly skilled operation but it does give a good result if you get it right.

Another way to get a harder cutting edge is to use a hardfacing welding electrode. This will not be as good as the heat treated steel above but it will be better than mild steel.

Lastly some concrete reinforcing bar called 'high yield' may be a bit harder.

1) To make the bit, get some steel bar of the same size as the hole you want to make, or a little bit smaller. Then hammer the end to flatten it a little (a bit like a screwdriver). The drawing shows what we mean.

2) Then file the end pointed and finally sharpen the cutting edges.

3) That's it!

Acknowledgements

This Technical Release was written as part of a three year animal cart development project in Kenya and Uganda.

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.

KENDAT

DTU

Low-Cost Animal Cart Programme

TECHNICAL
36
RELEASE

Twin Plastic Plain Bearing Axle for Donkey Carts

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk

KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

These bearings are designed to fit carts like that shown in Figure 2 but any cart could be modified to take the bearings described in this Technical Release.

These bearings are made of PVC water pipe. Polythene pipe would be much better but is not so easily available.

The system shown here is for two offset live axles (ie the axles go round). Using two axles allows any wheels to be used and provides plenty of room for the bearings.

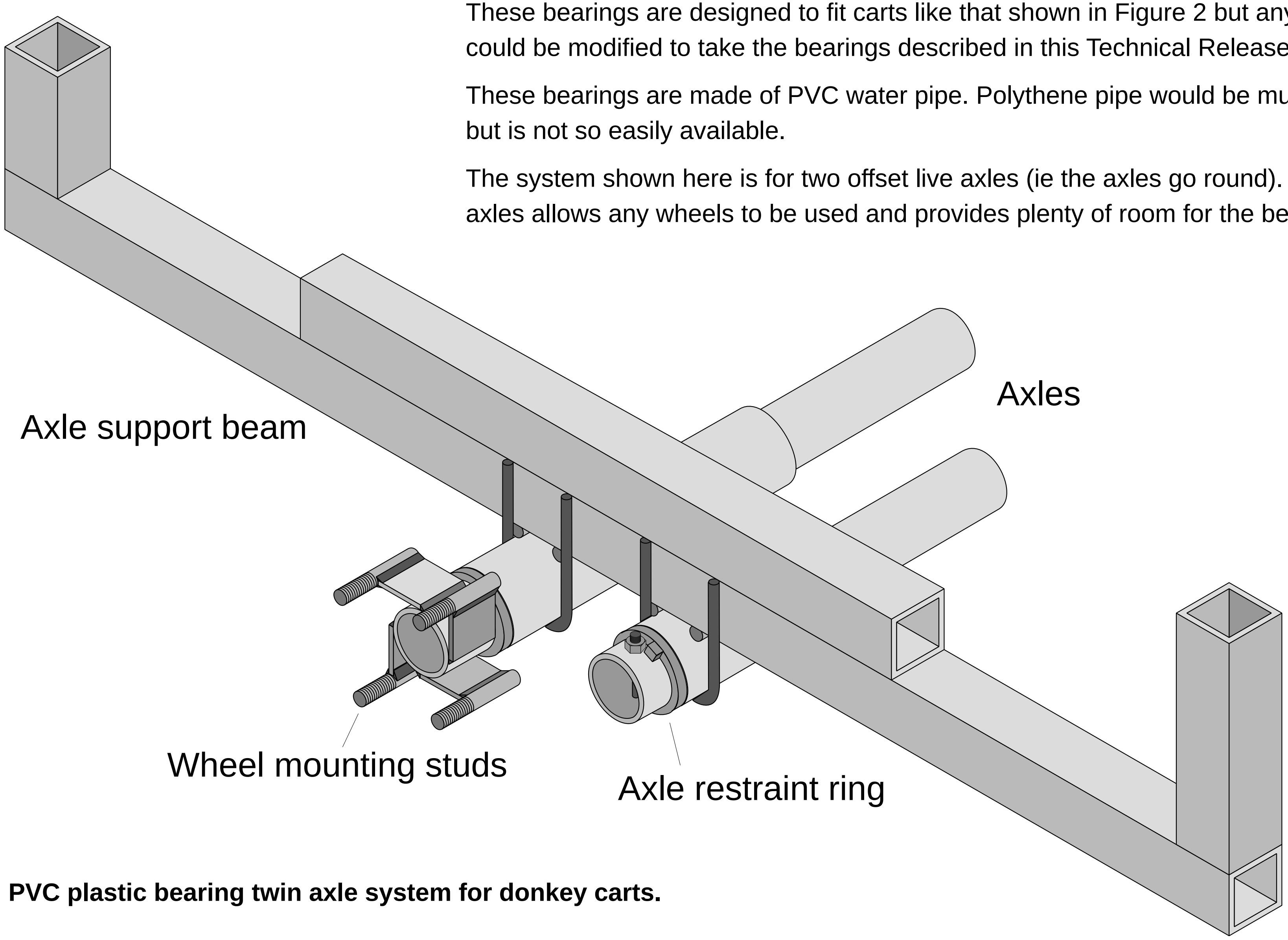


Figure 1: PVC plastic bearing twin axle system for donkey carts.

PVC plastic sleeve plain bearing axle system for a donkey cart.

Introduction

In this booklet we tell you how to make an axle system for a simple donkey cart from round steel tube and PVC plastic pipe. The instructions do not cover how to make the cart itself - you will need to read other Technical Releases from us to find out how to make the carts.

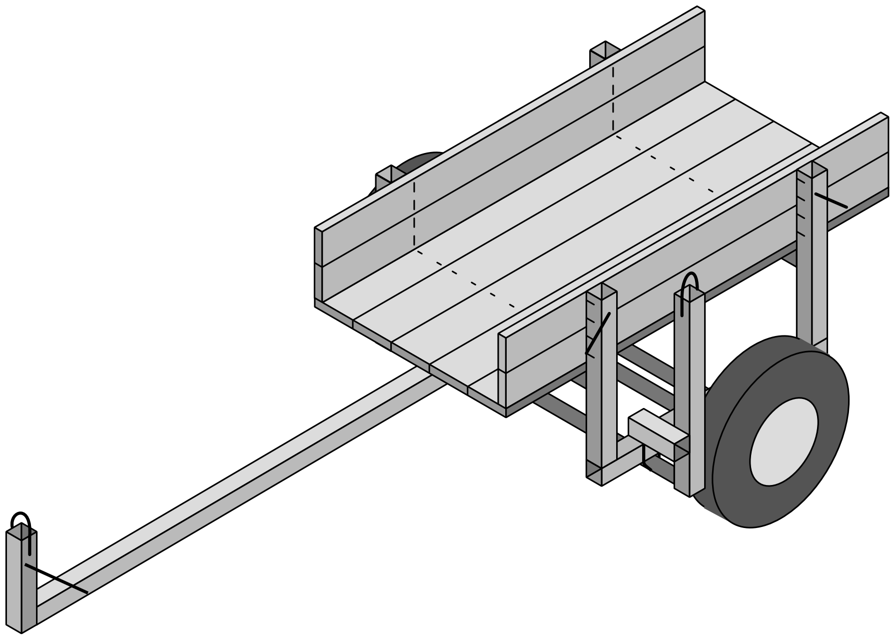


Figure 2: DTU donkey cart fitted with twin axles and PVC bearings.

You should find that you can make the axle system for about £50 including the wheels, tubes and tyres. This cost will depend on the cost of the materials and labour. Once you get organised, two men can probably make and fit one cart with axles in half a day. This is quite a lot faster than it takes to find and a scrap car axle and it will be much cheaper.

In other booklets in this series you can find out how to make other low-cost axle systems and carts.

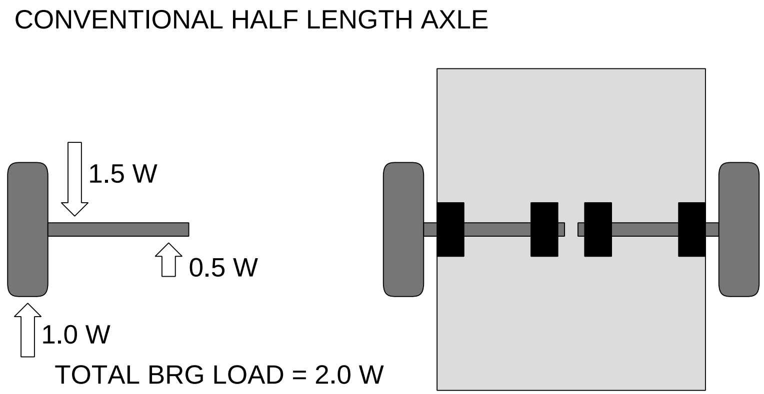


Figure 3: bearing loads in conventional half shaft axle.

Why have twin axles?

There are two types of axle: fixed or stub axle - the wheel hub rotates on the stationary axle; live axle - the axle revolves in stationary bearings.

With the stub axle types the bearings must be inside the wheel. This is easy with expensive ball bearings but more difficult with cheap wooden bearings. You need to make them quite long to stop wheel wobble and so they stick out of the wheel. It is also quite difficult to make without jigs and special tools. If you really want that type we have quite a good system using PVC tube for the bearing. We can send you a Technical Release on how to do it.

Twin axles allow much bigger bearings and do not require great accuracy in manufacture. Figure 3 shows the bearing loads of the usual way of doing it and Figure 4 shows the DTU method. You will see that bearing loads are 30% lower. Surprisingly there is no extra steel required either because there would have to be some steel to support the middle bearings anyway.

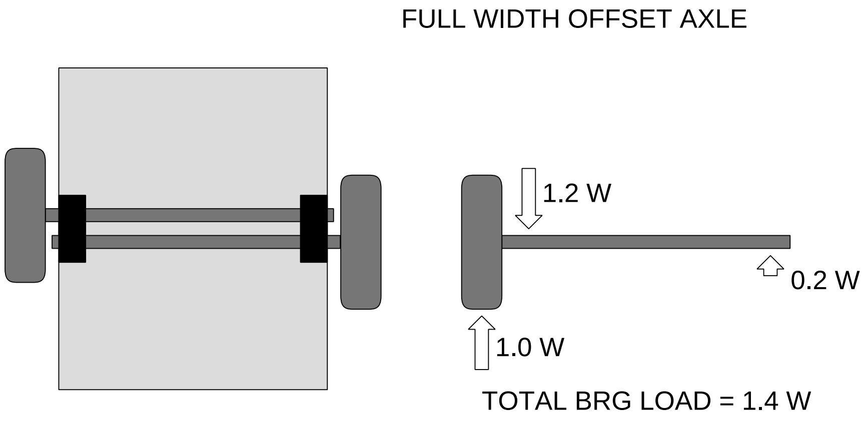


Figure 4: bearing loads in twin offset axles.

Easy to make design

These axles are designed to be constructed without any special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a simple welder, a hacksaw, and a hammer. You might find that a couple of 4" or a 5" G clamps (or something like it) are useful too. We have deliberately designed the cart so that drilling is not required.

You will see that there are no mitres or complicated angles or joints to cut when making the axles either, so you save time. Also the exact lengths of the components are not very critical -

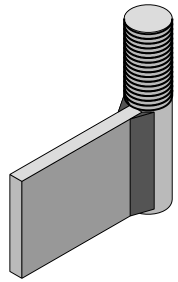


Figure 5: a welded wheel stud and strut fabrication.

again it saves a little time, but you will find that the axles look better if you take a little trouble to get things square and even etc and welding is easier with good square ends. It is much better to use a try square to mark the position of a cut than guess. In the instructions we have described how to mark pipe so you can cut it accurately.

We have tested many of these axles in Kenya and Uganda and we have had only a few failures caused by poor welding or incorrect material ie too thin. We think that they are strong enough, but you can always find someone to break anything. Really to get a reasonable cost you need to experiment a bit to

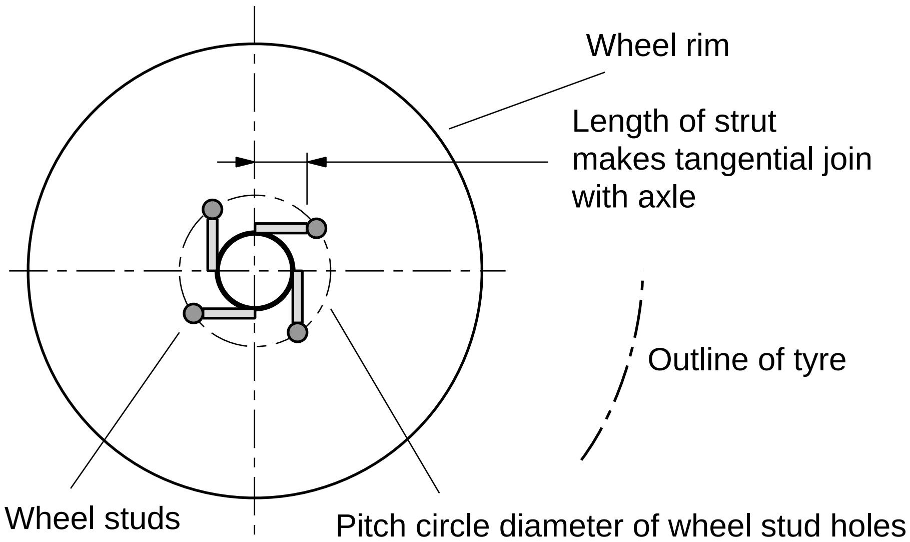


Figure 6: length of wheel strut.

see how the farmers treat their carts and what they expect them to stand.

It is also important to check the plastic sleeve is not worn through and to grease it every few weeks.

Cutting list and costs

Table 1 shows a cutting list for a complete axle - Recent prices of materials in Kenya are shown converted into £UK.

Construction step by step

1) The first job, is to get all the material together and clear a

TABLE 1: 50×50 RHS vestigial donkey cart.

component	material	# lengths reqd [#×mm]	total material for one axle [mm]	cost [UK£]
wheel studs	50×M12 nuts and bolts	10	10	2.60
wheel stud struts	6 × 40 flat bar	10×65	650.00	0.49
axial thrust rings	10 × 10 square bar	4 × 200	800.00	0.25
pipe clips	50 mm pipe clips	4	4	2.40
axle cross bolts	75×M12 nuts and bolts	4	4	1.04
axles	1-½"" BSP malleable iron pipe	2 × 1500	3000.00	6.00
hub outer tube	2 BSP malleable iron pipe	2×200	400.00	0.95
plastic bearing sleeves	1½"" or 2"" PVC plastic pipe	2×300+2×100	800.00	0.53
wheel rims, tyre + tubes				25.00
			TOTAL =	**39.25**

space to work. Ideally you will be able to work on a flat area of concrete.

2) Start by making the wheel stud struts shown in Figure 5. You need to make one of these struts for every stud hole in the wheels you are going to use. Figure 6 shows how to measure the length of the struts. The struts are made from 6×40 flat bar and M12 bolts 50mm long. The flat bar should be long enough so that it meets the axle tube tangentially as shown in Figure 6.

3) Once you have made these struts, screw a nut onto each one until it touches the 40×6 metal. Then put the thread through the hole in the rim and screw another nut onto the thread. Tighten this nut lightly with a spanner. Repeat for all the struts so that they all point the same way round the axle, as in Figure 6, and leave a gap for the axle.

4) Now centre the axle in the rim and get it square using a tape measure, a trysquare and a plank or piece of steel resting on the tyre.

   To do this put the wheel rim on the floor and put the axle in place in the middle. You should put something under the end of the pipe to rest it on to get it in the right position as shown in Figure 7. Get an assistant to hold the top end of the pipe and tell him to keep very still! Use your tape to measure from the outside of the pipe to the inside of the rim as Figure 7 shows. Measure in one place and then measure the gap opposite. Move the axle pipe over until it is central. Repeat this for the other direction at right angles.

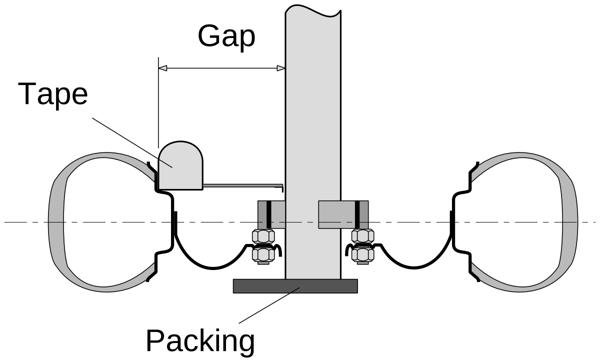


Figure 7: using tape measure to centre axle in wheel.

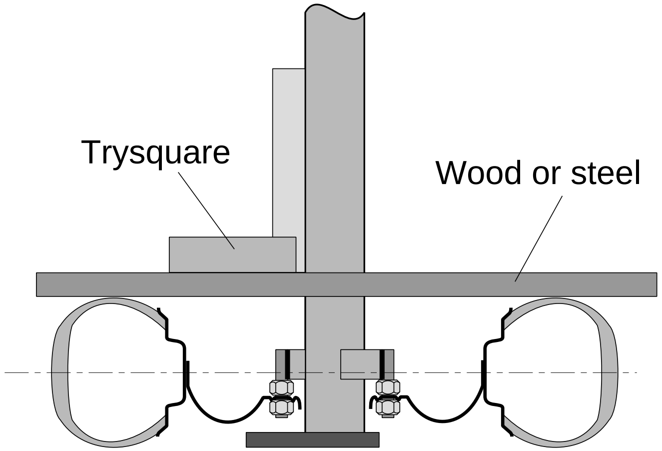


Figure 8: using trysquare to get axle square to wheel.

Now use the trysquare and a piece of wood to get the axle square to the rim as shown in Figure 8. You put the wood on the tyre or rim so that it is flat and you put the trysquare on the wood. You have to move the axle until it is straight with the trysquare and your assistant must hold it without moving. Check it several times - its hard to correct!

5) Once you have it in position, tack weld the ends of the struts to the axle tube as shown in Figure 9, then weld the struts on properly. Do as much welding as you can without taking the axle out of the wheel because the metal changes size as it heats and cools and it may move out of place.

6) Next make the bearing tubes from 2" round pipe to the lengths in the cutting list. To mark a line around the pipe to cut it square, wrap a strong piece of paper or thin card around the pipe, get the edge in line and use the edge to guide the felt tip pen or scriber as you mark the line.

7) Cut eight 50mm pieces of 8 to 12mm round bar for the bearing tube chocks. Mark the centre of the axle support beam of the cart and put marks on the beam 60mm either side of this. You need to weld the chocks on to the beams so that the bearing tubes just sit nicely in the space between the chocks as shown in Figure 10.

8) Fix the axle bearing tubes with loops of 8 to 12mm round bar as shown in Figure 1 and Figure 11. Do not weld the

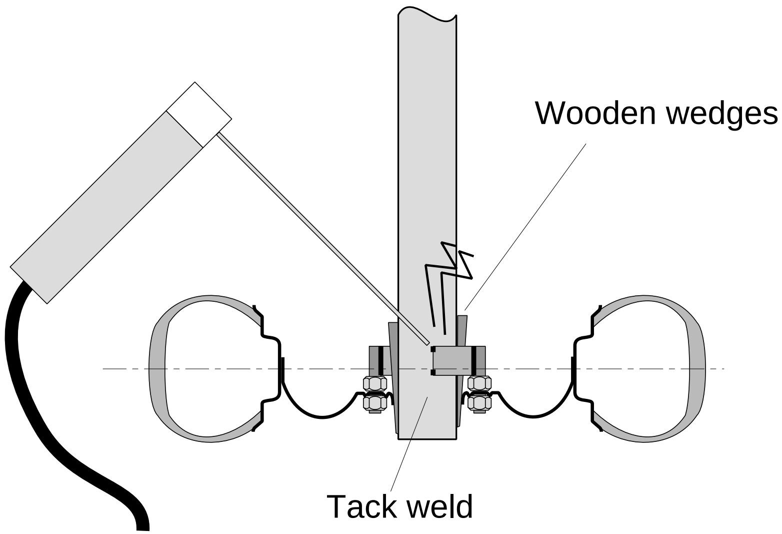


Figure 9: tyre, wheel and axle tube during tack welding stud support struts

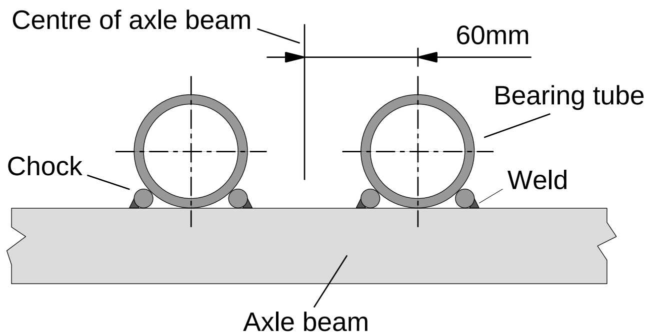


Figure 10: position of bearing tubes and chocks.

bearing tubes directly to the axle support beam. The bearing tubes must be slightly flexible so that they can align themselves with the other bearing on the same axle.

9) Make four axial rings of 10 to 12mm square or round bar which just go over the axles. Square bar is better but round will do. You must remember to weld on a tag made of a 15mm length of bar to each ring as shown in the drawings. This makes the rings go round with the axle and stops wear in the wrong places. Put a ring on each axle so the tag goes between two stud support struts.

10) Insert an axle into its bearing tubes and push right through until the axial ring touches the bearing tube and then put the second axial ring on.

11) Now mark the position of the cross bolt hole. Remember that the nut will have to be turned so do not make the hole too close to the ring - centre about 15mm away is fine. Use the welder to blow a hole through for the bolt. Of course a hole will have to be blown through on the other side of the axle too.

12) Cut pieces of PVC tube to lengths shown in the cutting list and either cut along their length or cut around like a spring or screw as shown in Figure 12. Remove axles from bearing tubes, apply lots of grease, fit the PVC bearing pipes and the pie clips and refit axles. Use the pipe clips to stop the PVC pipe coming out of the bearing tubes. Tighten the cross bolts.

13) You've finished it!

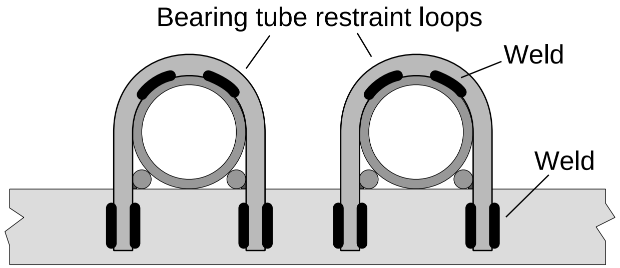


Figure 11: position of bearing tubes and chocks.

Other DTU cart developments

The DTU has been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. It has designs for twin axles with wooden bearings and twin axles with scrap or new ball bearings which do not need any machining. And it has two systems of fixed axle: one with PVC bearings and another

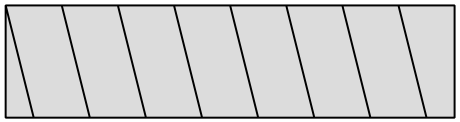


Figure 12: cut PVC bearing tubes like spring to fit axle.

using needle roller bearings which you make yourself. Again for these needle roller bearings no machining is necessary.

Other hub designs using, for example aluminium castings, have been in production in Nigeria and we are trying to reduce or eliminate the machining in these. Also wheel designs in steel sheet, cast aluminium and timber are under development.

The DTU has also been working on a range of cart body types for use with both donkeys and oxen. It has designs for wooden and steel framed types. The wooden types are cheaper in material terms, but the steel framed ones are easier to make because the joints are more straightforward - nevertheless you can make either type of cart in only a few hours, if you are reasonably set up with tools and materials.

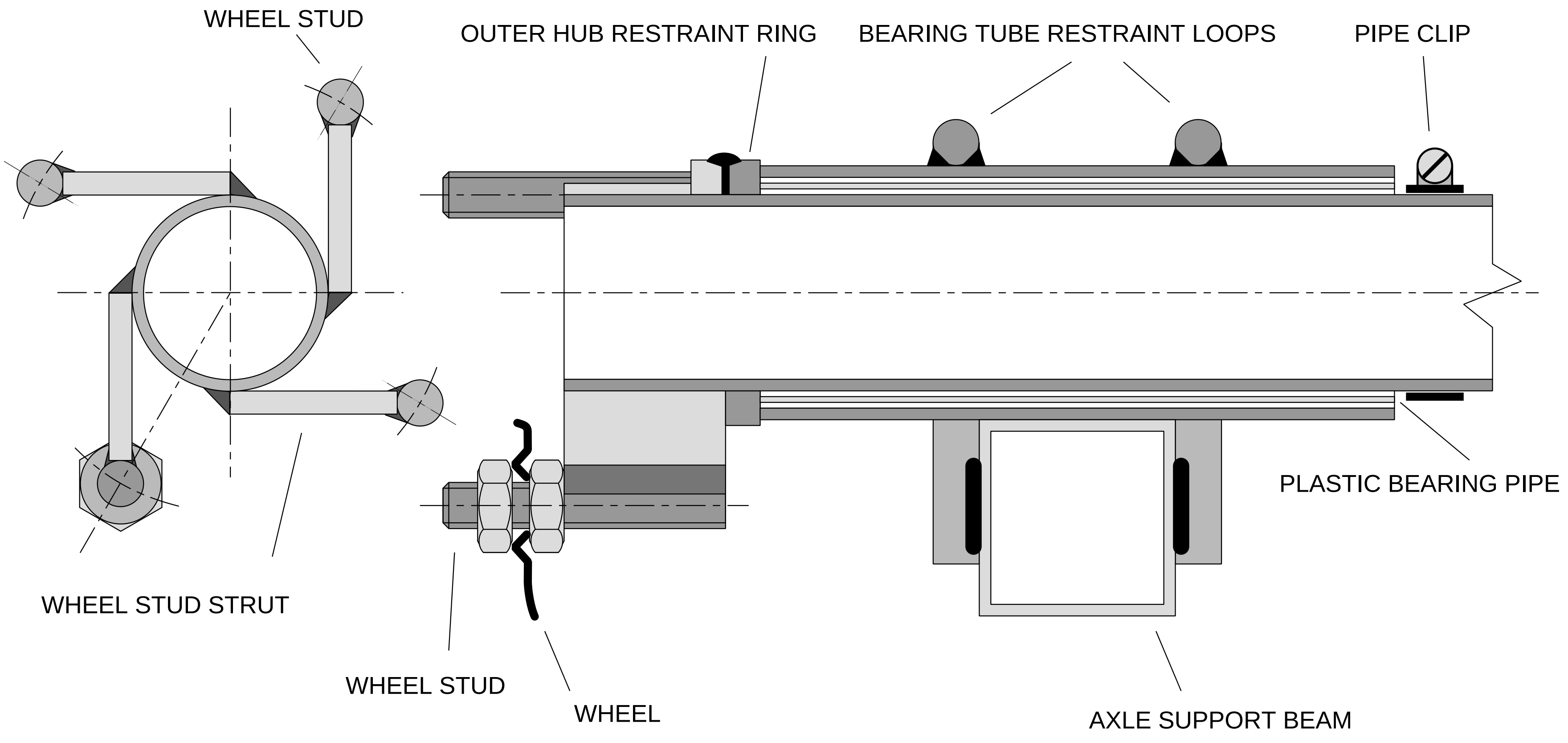
Drawings

You will find four drawings on the next pages, the first two give a general section view of the axle. The third gives a view of the components of the axle itself and the fourth a view of the bearing tube and PVC bearing sleeve.

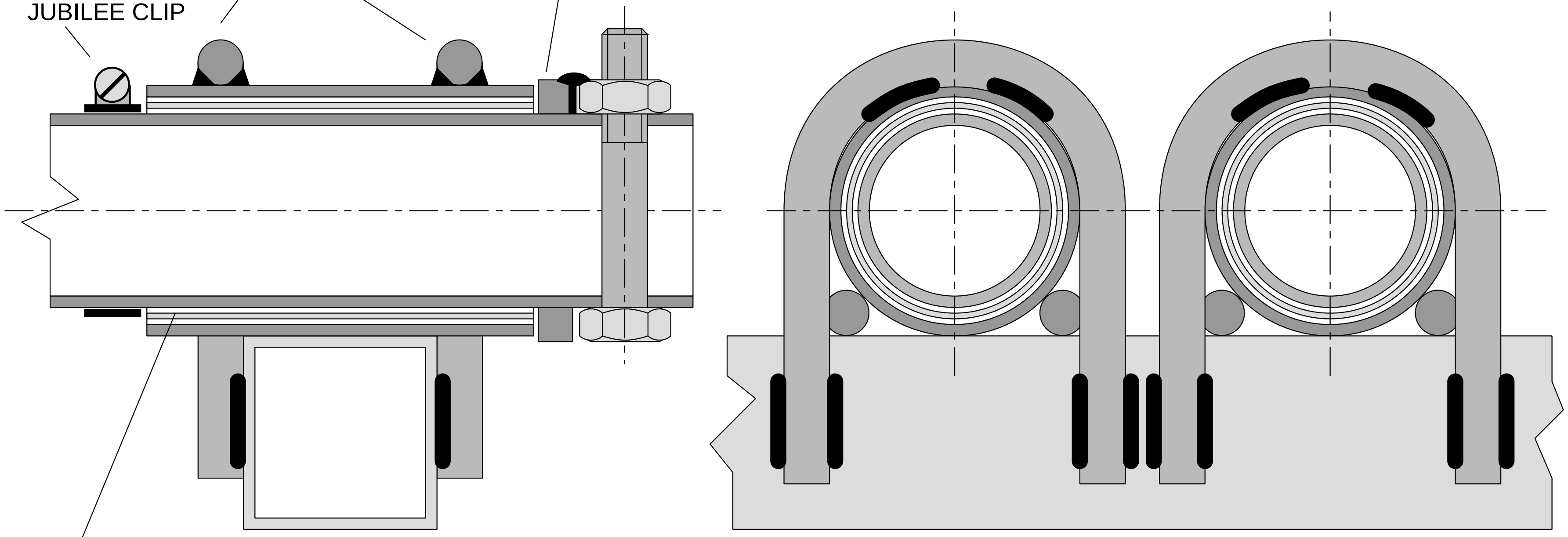
Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

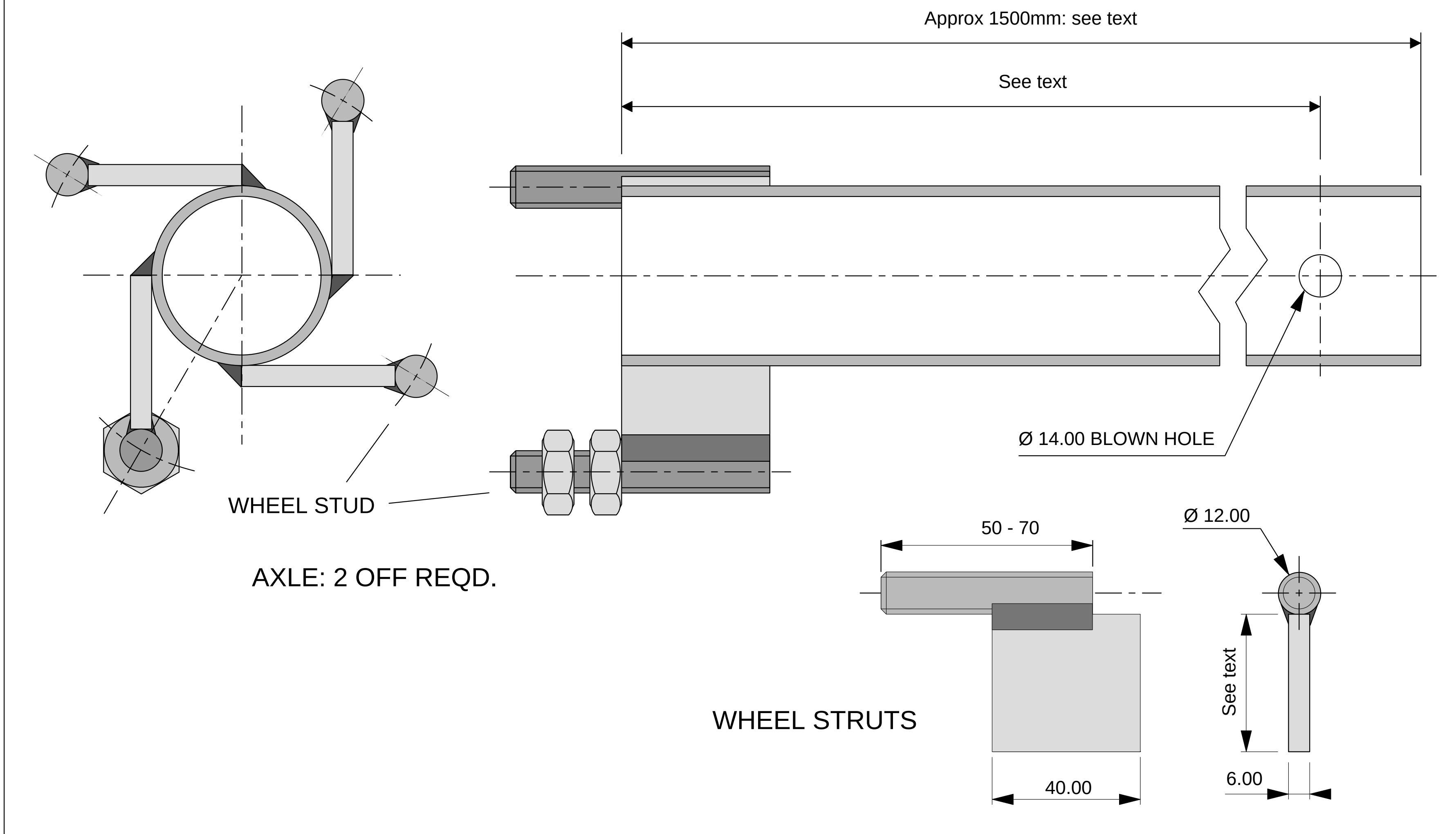
The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



	GENERAL ASSEMBLY DRAWING 1	Scale	10mm	Title PVC BEARING TWIN DONKEY CART AXLE		Drawn by	CEO
		Date	16-3-99			Dwg No.	1/4

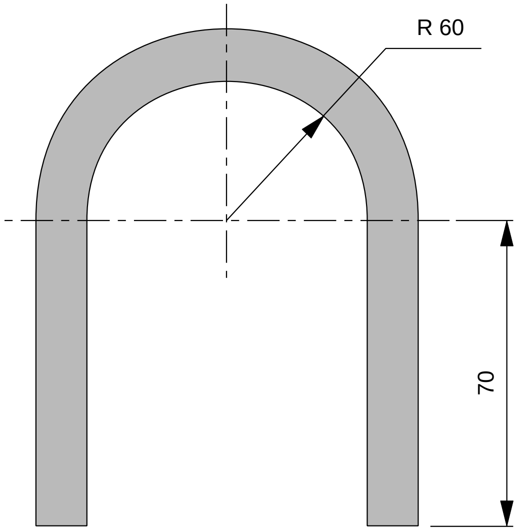


GENERAL ASSEMBLY DRAWING 2	Scale	10mm	Title PVC BEARING TWIN DONKEY CART AXLE	Drawn by	CEO
	Date	16-3-99		Dwg No.	2/4

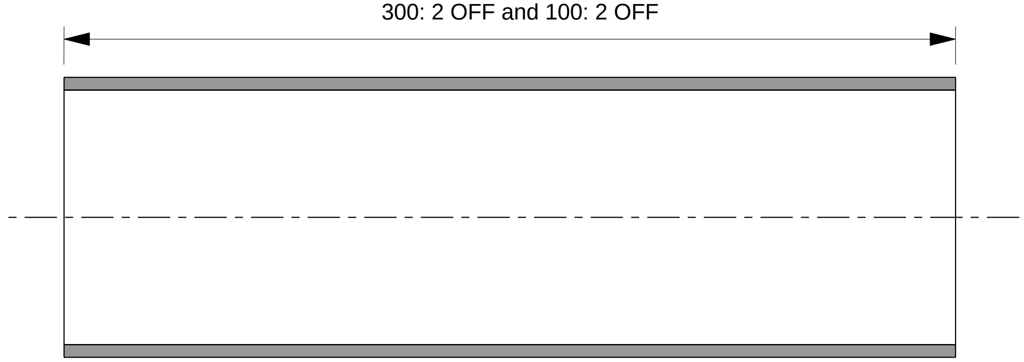


	AXLE COMPONENTS	Scale	10mm	Title PVC PLAIN BEARING DONKEY CART AXLE	Drawn by	CEO
		Date	29-12-96		Dwg No.	3/4

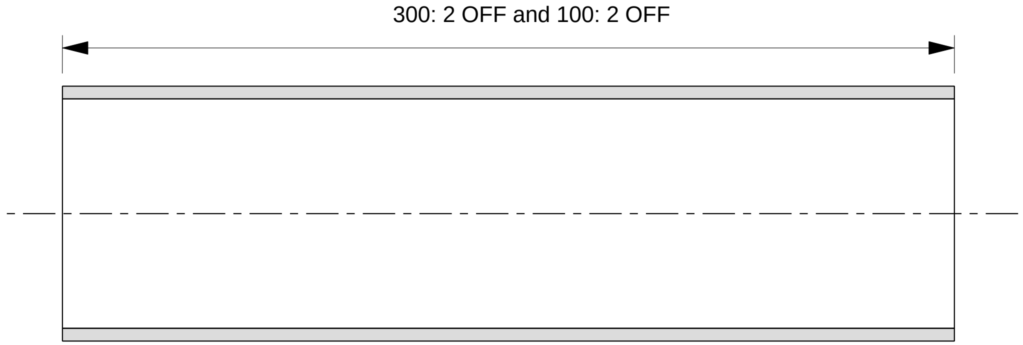
BEARING TUBE SUPPORT LOOPS

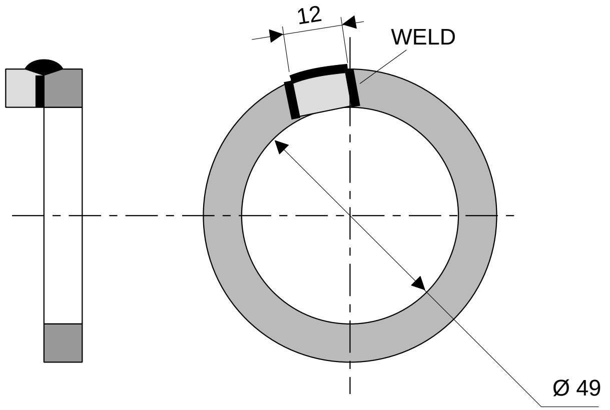


HUB TUBE 2" BSP



PLASTIC BEARING PIPE (SEE TEXT)





AXIAL RESTRAINT RING

BEARING COMPONENTS	Scale	10mm	Title PVC PLAIN BEARING DONKEY CART AXLE	Drawn by	CEO
	Date	16-3-99		Dwg No.	4/4

KENDAT

DTU

Low-Cost Animal Cart Programme

TECHNICAL
37
RELEASE

Twin Wood Plain Bearing Axle for Donkey Carts

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk

KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

These bearings are designed to fit carts like that shown in Figure 2 but any cart could be modified to take the bearings described in this Technical Release.

The system shown here is for two offset live axles (ie the axles go round). Using two axles allows any wheels to be used and provides plenty of room for the bearings.

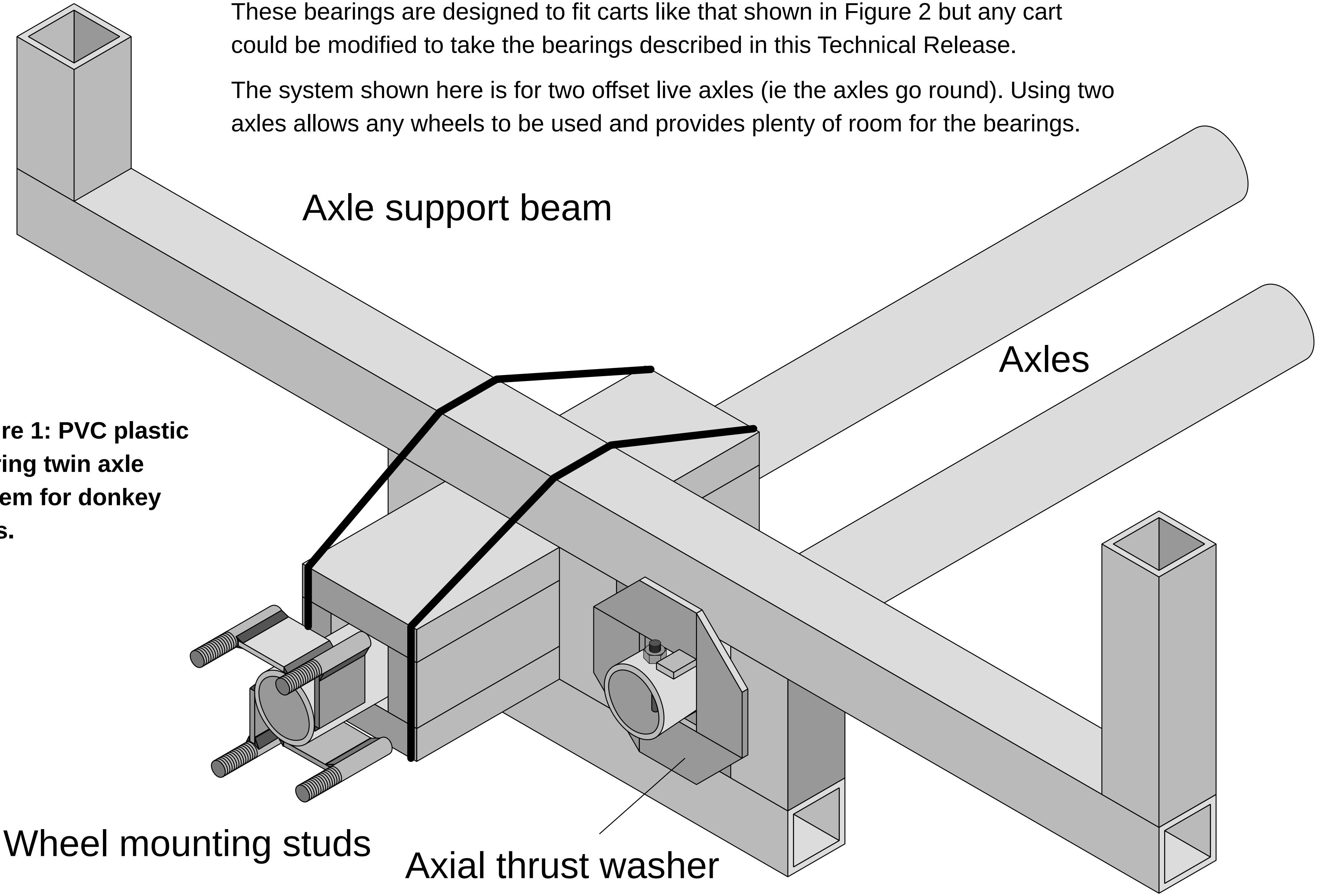


Figure 1: PVC plastic bearing twin axle system for donkey carts.

Wood plain bearing twin axle system for donkey carts.

Introduction

In this booklet we tell you how to make an axle system for a simple donkey cart from round steel tube and wooden planks. The instructions do not cover how to make the cart itself - you will need to read other Technical Releases from us to find out how to make the carts.

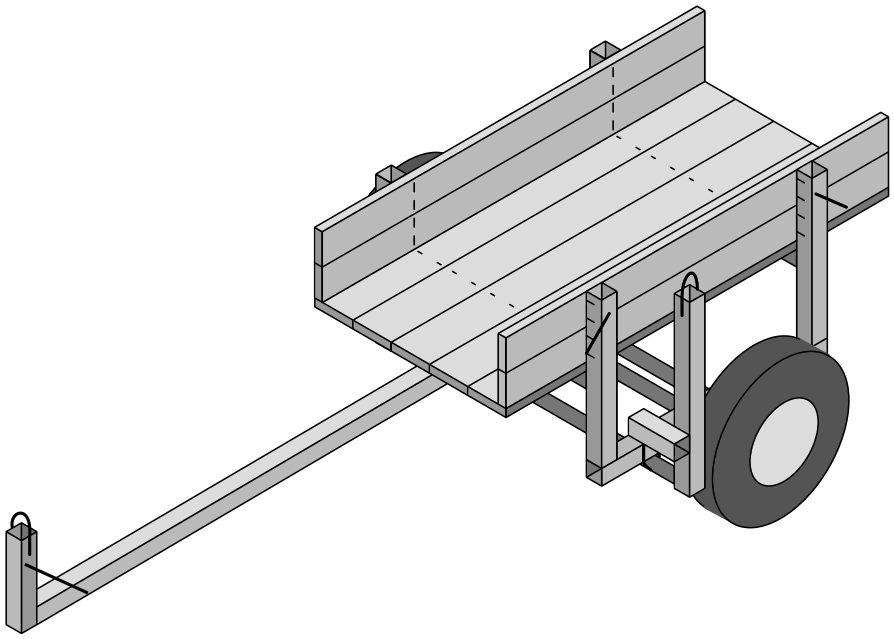


Figure 2: DTU donkey cart fitted with twin axles and PVC bearings.

You should find that you can make the axle system for about £40 including the wheels, tubes and tyres. This cost will depend on the cost of the materials and labour. Once you get organised, two men can probably make and fit one cart with axles in half a day. This is quite a lot faster than it takes to find and a scrap car axle and it will be much cheaper.

In other booklets in this series you can find out how to make other low-cost axle systems and carts.

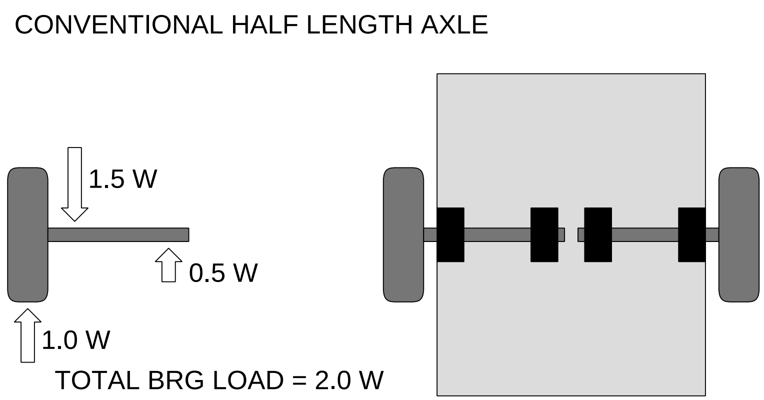


Figure 3: bearing loads in conventional half shaft axle.

Why have twin axles?

There are two types of axle: fixed or stub axle - the wheel hub rotates on the stationary axle; live axle - the axle revolves in stationary bearings.

With the stub axle types the bearings must be inside the wheel. This is easy with expensive ball bearings but more difficult with cheap wooden bearings. You need to make them quite long to stop wheel wobble and so they stick out of the wheel. It is also quite difficult to make without jigs and special tools. If you really want that type we have quite a good system using PVC tube for the bearing. We can send you a Technical Release on how to do it.

Twin axles allow much bigger bearings and do not require such great accuracy in manufacture. Figure 3 shows the bearing loads of the usual way of doing it and Figure 4 shows the DTU method. You will see that bearing loads are 30% lower. Surprisingly there is no extra steel required either because there would have to be some steel to support the middle bearings anyway.

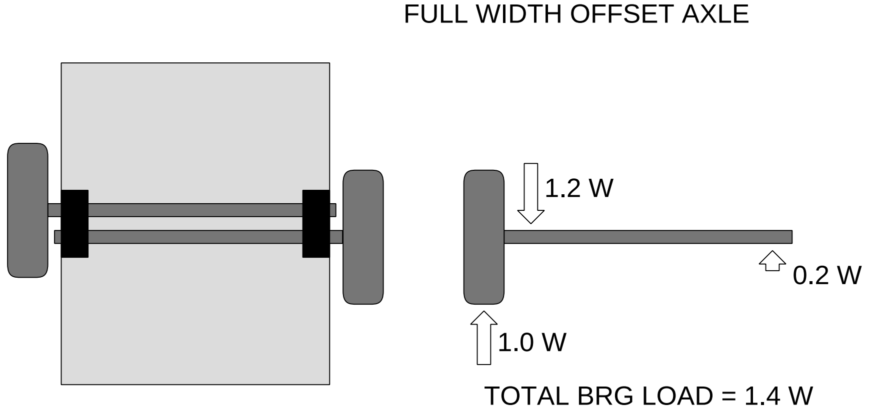


Figure 4: bearing loads in twin offset axles.

Easy to make design.

These axles are designed to be constructed without any special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a simple welder, a hacksaw, and a hammer. You might find that a couple of 4" or a 5" G clamps (or something like it) are useful too. We have deliberately designed the axle so that drilling is not required.

We have tested many of these axles in Kenya and Uganda and we have had only a few failures caused by poor welding or

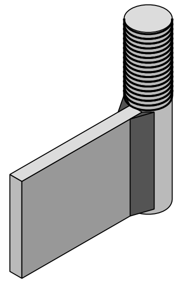


Figure 5: a welded wheel stud and strut fabrication.

incorrect material. We think that they are strong enough, but you can always find someone to break anything. To get a reasonable cost you need to experiment a bit to see how the farmers treat their carts and what they expect them to stand.

It is important to grease the bearings every few weeks.

Cutting list and costs

Table 1 shows a cutting list for a complete axle - Recent prices of materials in Kenya are shown converted into £UK.

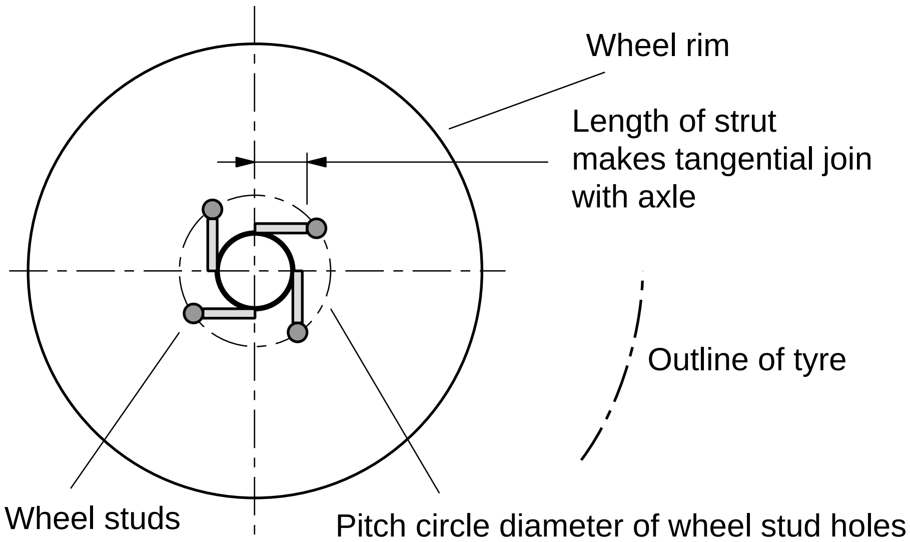


Figure 6: length of wheel strut.

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete.

2) Start by making the wheel stud struts shown in Figure 5. You need to make one of these struts for every stud hole in the wheels you are going to use. Figure 6 shows how to measure the length of the struts. The struts are made from 6×40 flat bar or similar and M12 bolts 50mm or 60mm long. The flat bar should be long enough so that it meets the axle tube tangentially as shown in Figure 6.

TABLE 1: materials for wooden bearing twin axle.

component	material	# lengths reqd [#×mm]	total material for one axle [mm]	cost [UK£]
wheel studs	50×M12 nuts and bolts	10	10	2.60
wheel stud struts	6 × 40 flat bar	10×65	650.00	0.49
axial thrust washers	6 × 40 flat bar	8×90	720.00	0.54
axle cross bolts	75×M12 nuts and bolts	4	4	1.04
axles	1-½" BSP malleable iron pipe	2 × 1500	3000.00	6.00
bearing block restraint loops	R 8 re-bar	8×430	3440.00	0.56
main bearing top + bottom	100×25mm timber	4×300	1200.00	0.20
main bearing sides	50×25mm timber	4×300	1200.00	0.20
small bearing top + bottom	100×25mm timber	4×50	200.00	0.03
small bearing sides	50×25mm timber	4×50	200.00	0.03
wheel rims, tyre + tubes				25.00
			TOTAL	**36.69**

3) Once you have made these struts, screw a nut onto each one until it touches the 40×6 metal. Then put the thread through the hole in the rim and screw another nut onto the thread. Tighten this nut lightly with a spanner. Repeat for all the struts so that they all point the same way round the axle, as in Figure 6, and leave a gap for the axle.

4) Now centre the axle in the rim and get it square using a tape measure, a trysquare and a plank or piece of steel resting on the tyre.

   To do this put the wheel rim on the floor and put the axle in place in the middle. You should put something under the end of the pipe to get it in the right position as shown in Figure 7. Get an assistant to hold the top end of the pipe and tell him to keep very still! Use your tape to measure from the outside of the pipe to the inside of the rim as Figure 7 shows. Measure in one place and then measure the gap opposite. Move the axle pipe over until it is central. Repeat this for the other direction at right angles.

   Now use the trysquare and a piece of wood to get the axle square to the rim as shown in Figure 8. You put the wood on the tyre or rim so that it is flat and you put the trysquare on the wood. You have to move the axle until it is straight with the trysquare and your assistant must hold it without moving. Check it several times - its hard to correct!

5) Once you have it in position, tack weld the ends of the struts to the axle tube as shown in Figure 9. (As Figure 9

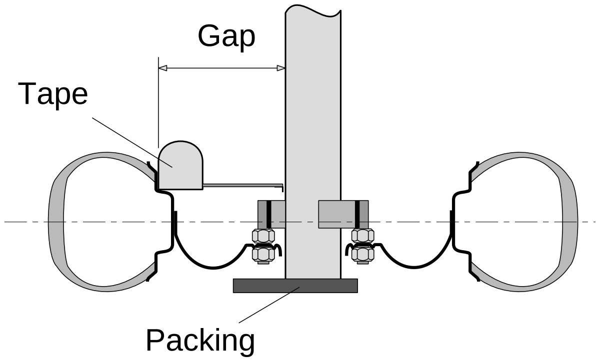


Figure 7: using tape measure to centre axle in wheel.

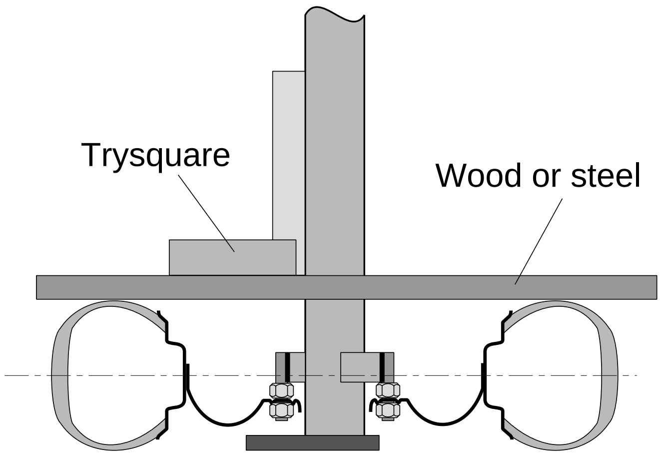


Figure 8: using trysquare to get axle square to wheel.

shows, you can use wooden wedges to hold the axle in place in the rim.) Then weld the struts on properly. Do as much welding as you can without taking the axle out of the wheel because the metal changes size as it heats and cools and it may move out of place.

6) Next make the bearing blocks from 100×25mm timber and 50×25mm timber to the lengths in the cutting list.

7) Cut six 105mm pieces of square tubing for the bearing boxes. Mark the centre of the axle support beam of the cart and put marks on the beam 25mm and 125mm either side of this. You need to weld the 105mm pieces on to the beams as shown in Figure 10.

8) Fit the bearing blocks and fix them in place with loops of 8 to 12mm round bar as shown in Figure 1 and Figure 11.

9) Make four axial thrust washers from 40×6 or 40×3 or similar flat bar like those shown in Figure 12. You must remember to weld on a tag made of a 20mm length of bar to each ring as shown in the drawings. This makes the washers go round with the axle and stops wear in the wrong places.

10) Insert an axle into its large bearing but not its small bearing

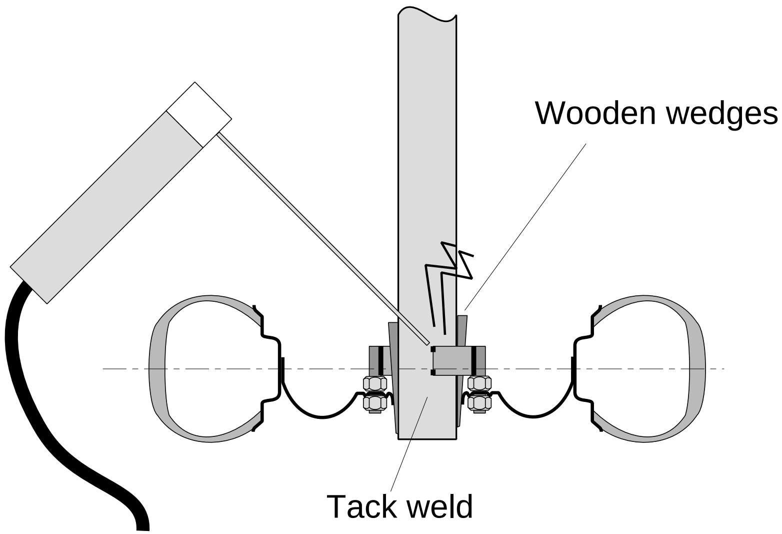


Figure 9: tyre, wheel and axle tube during tack welding stud support struts

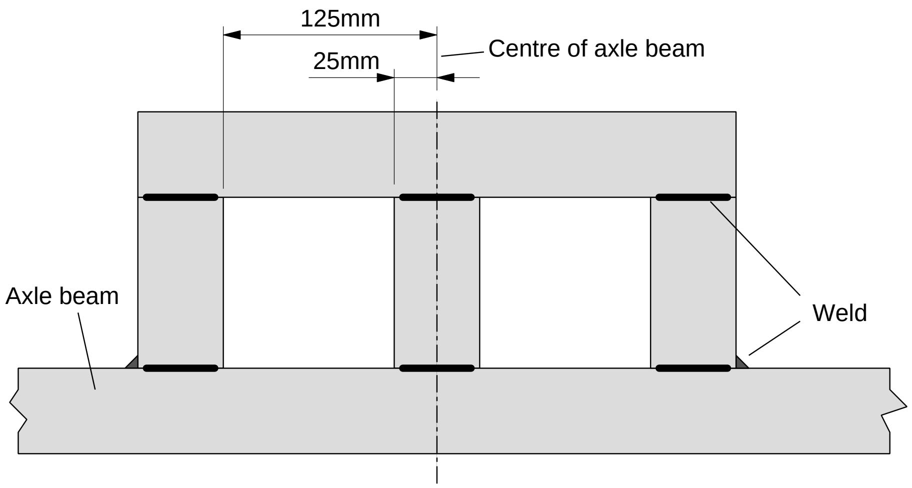


Figure 10: position of bearing support blocks.

and tn put on one of the axial bearing washers with the tag away from the small bearing. Then push the axle all the way through until there is a 20 mm gap between the wheel stud struts and the bearing blocks. Put the second axial ring on to the axle. You should now have an axial washer either side of the small bearing as shown in Figure 13.

11) Now mark the position of the cross bolt holes. Remember that the nuts will have to be turned so do not make the holes too close to the washers - centre about 15mm away is fine. Use the welder to blow the holes.

12) Remove axles from the bearing, apply lots of grease and refit the axles, washers and cross bolts.

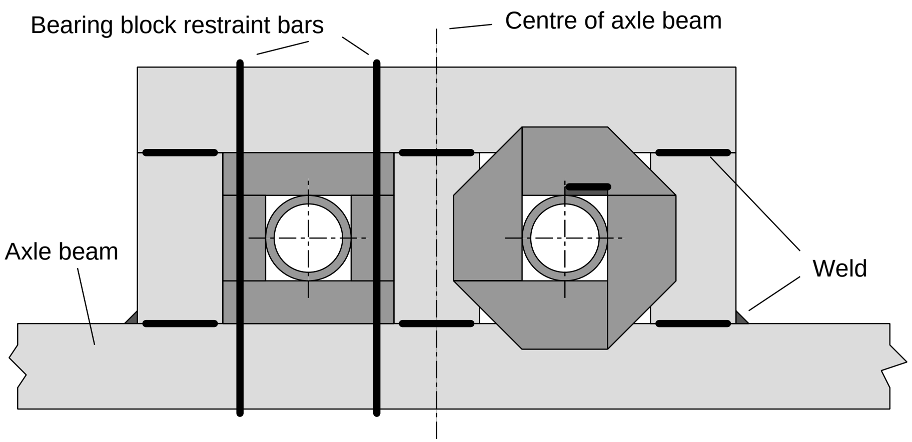


Figure 11: axial bearing washer.

13) Cut the excess axle off about 40mm from the washer. To mark a line around the pipe to cut it square, wrap a strong piece of paper or thin card around the pipe, get the edge in line and use the edge to guide the felt tip pen or scriber as you mark the line.

14) You've finished it!

Other DTU cart developments

The DTU has been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. It has designs for twin axles with wooden bearings and twin axles with scrap or new ball bearings which do not need any machining. And it has two systems of fixed axle: one with PVC bearings and another

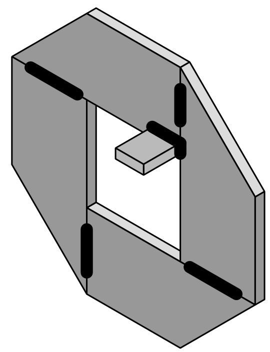


Figure 12: axial bearing washer.

using needle roller bearings which you make yourself. Again for these needle roller bearings no machining is necessary.

Other hub designs using, for example aluminium castings, have been in production in Nigeria and we are trying to reduce or eliminate the machining in these. Also wheel designs in steel sheet, cast aluminium and timber are under development.

The DTU has also been working on a range of cart body types for use with both donkeys and oxen. It has designs for wooden

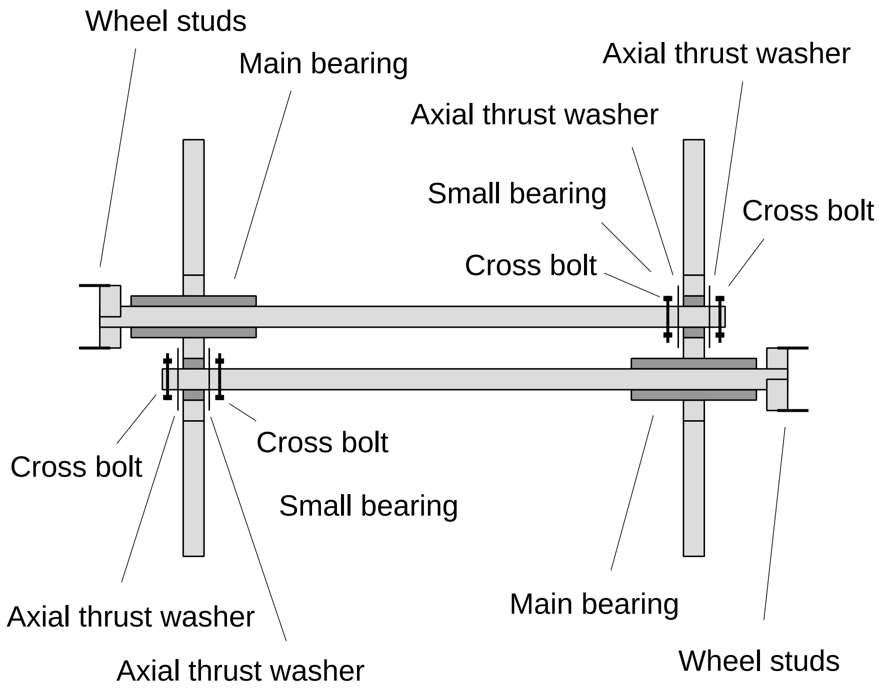


Figure 13: axle and bearing arrangement.

and steel framed types. The wooden types are cheaper in material terms, but the steel framed ones are easier to make because the joints are more straightforward - nevertheless you can make either type of cart in only a few hours, if you are reasonably set up with tools and materials.

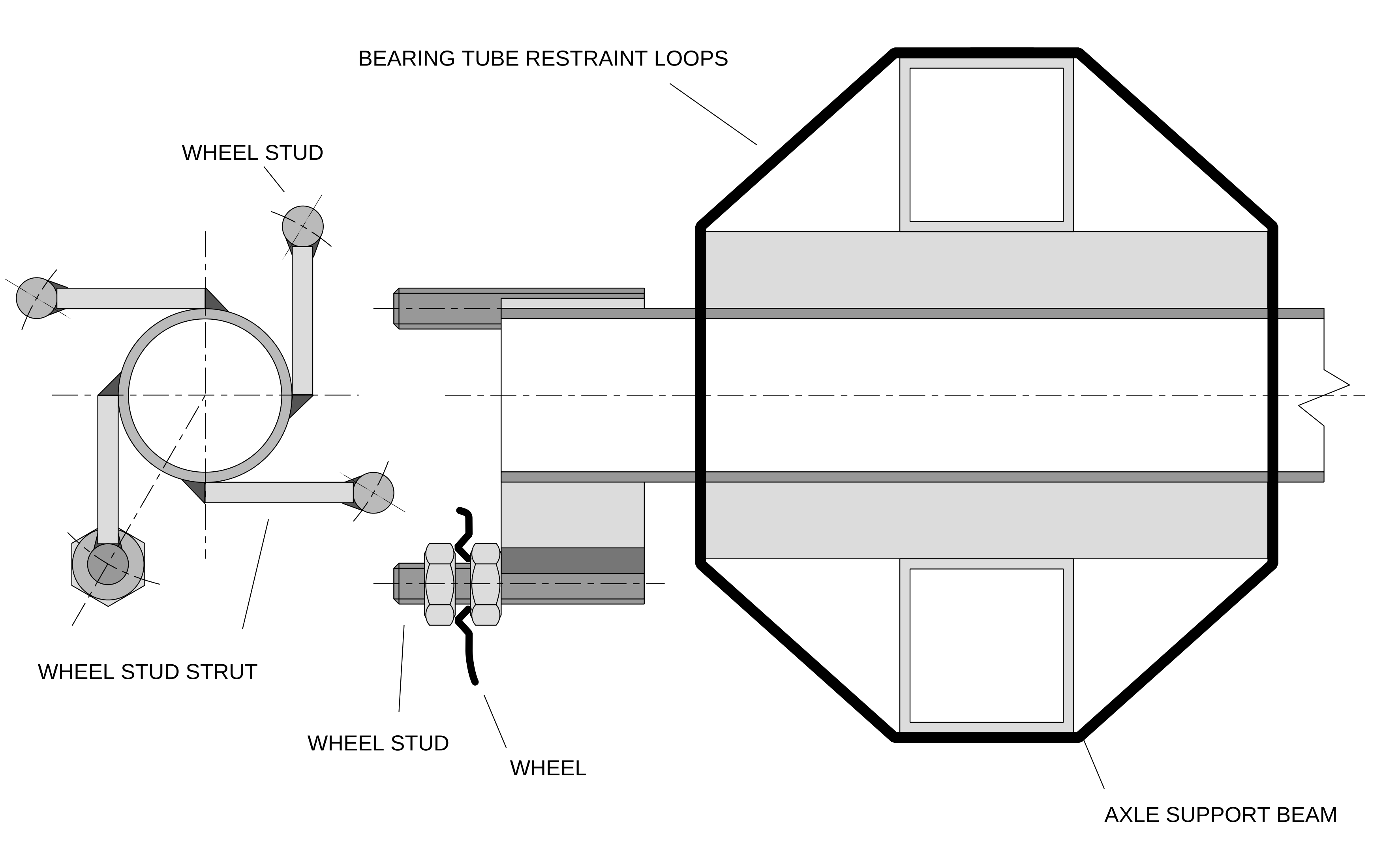
Drawings

You will find four drawings on the next pages, the first two give a general section view of the axle. The third gives a view of the components of the axle itself and the fourth a drawing of the thrust washer.

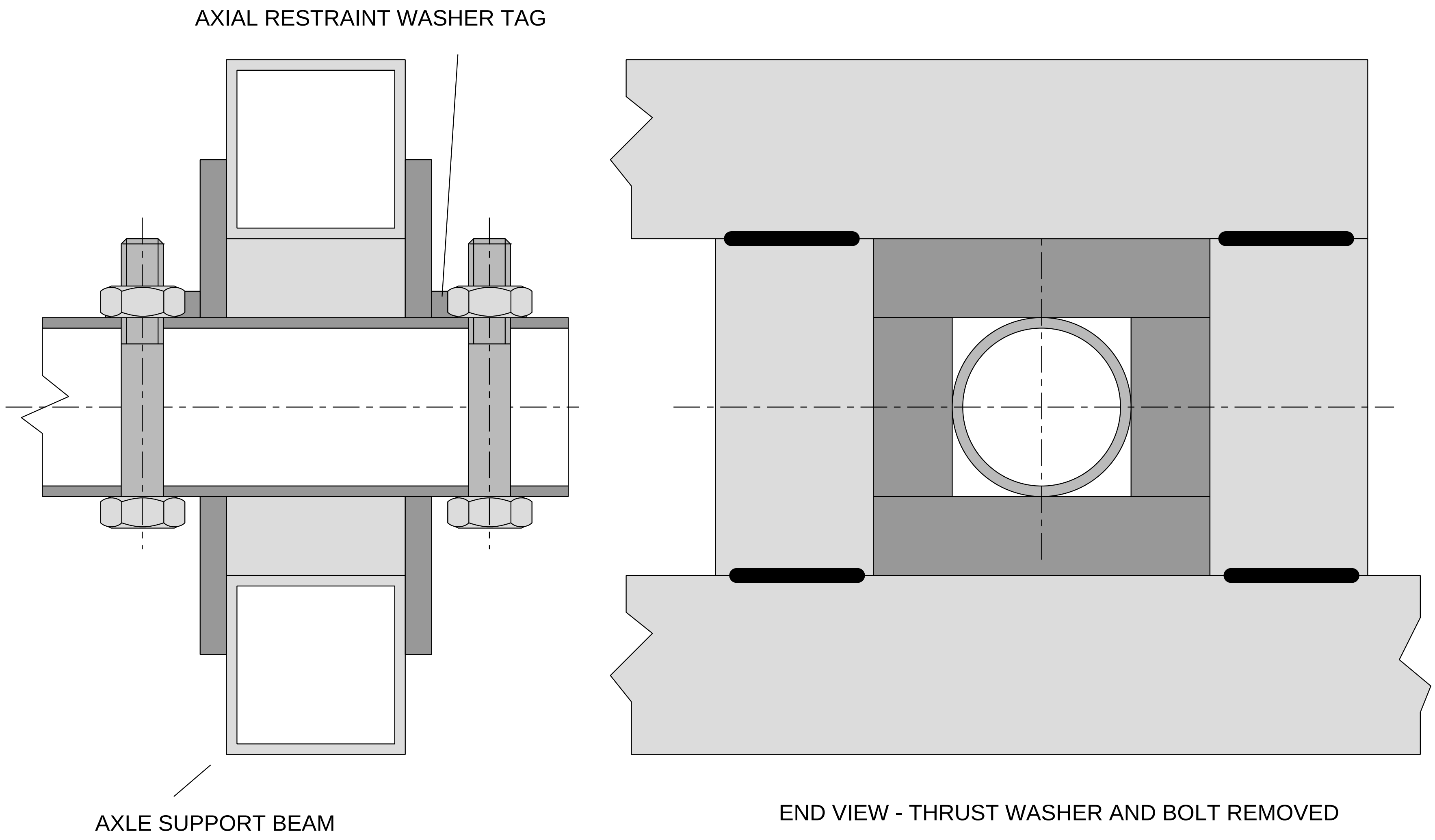
Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

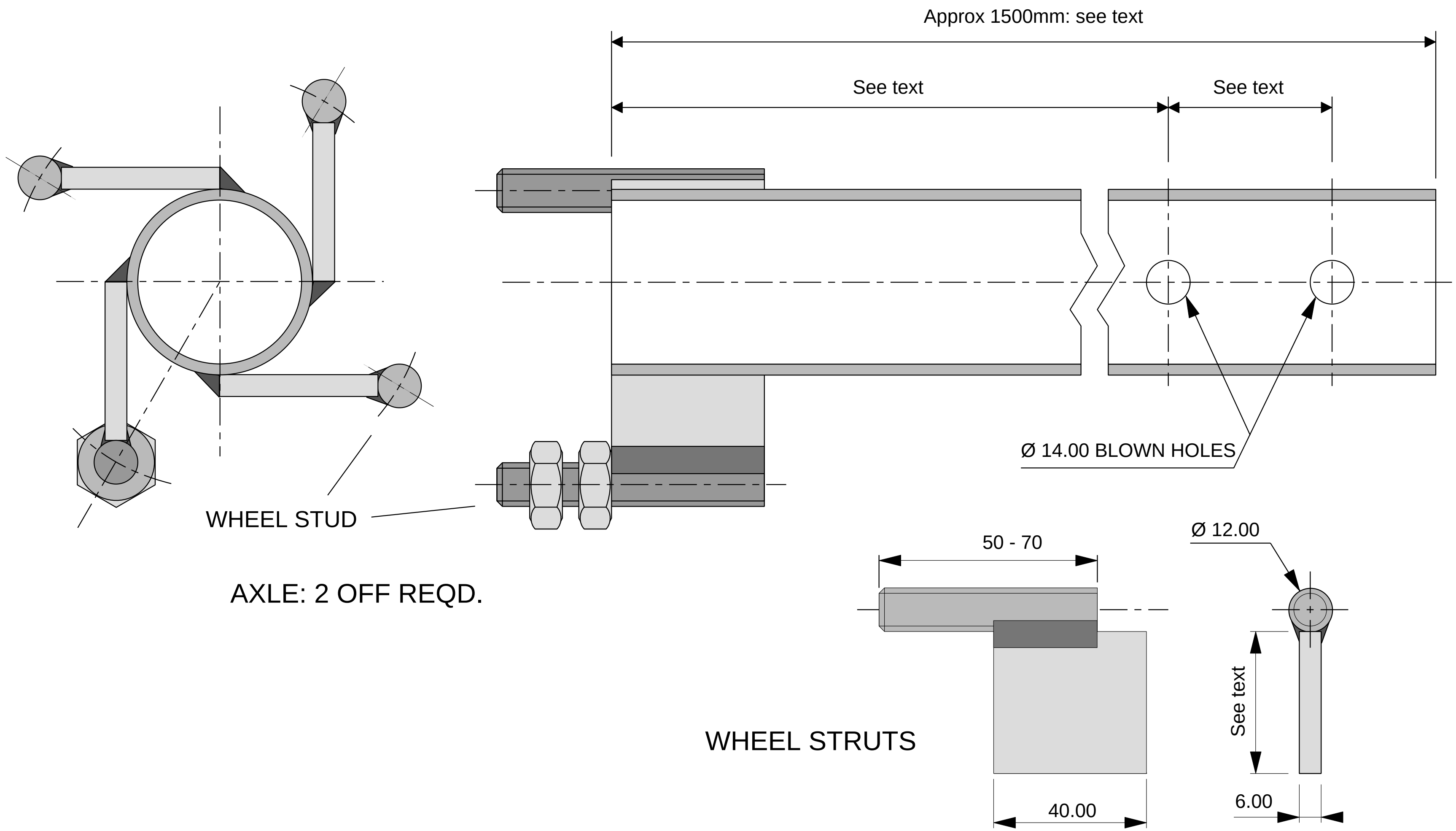
The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



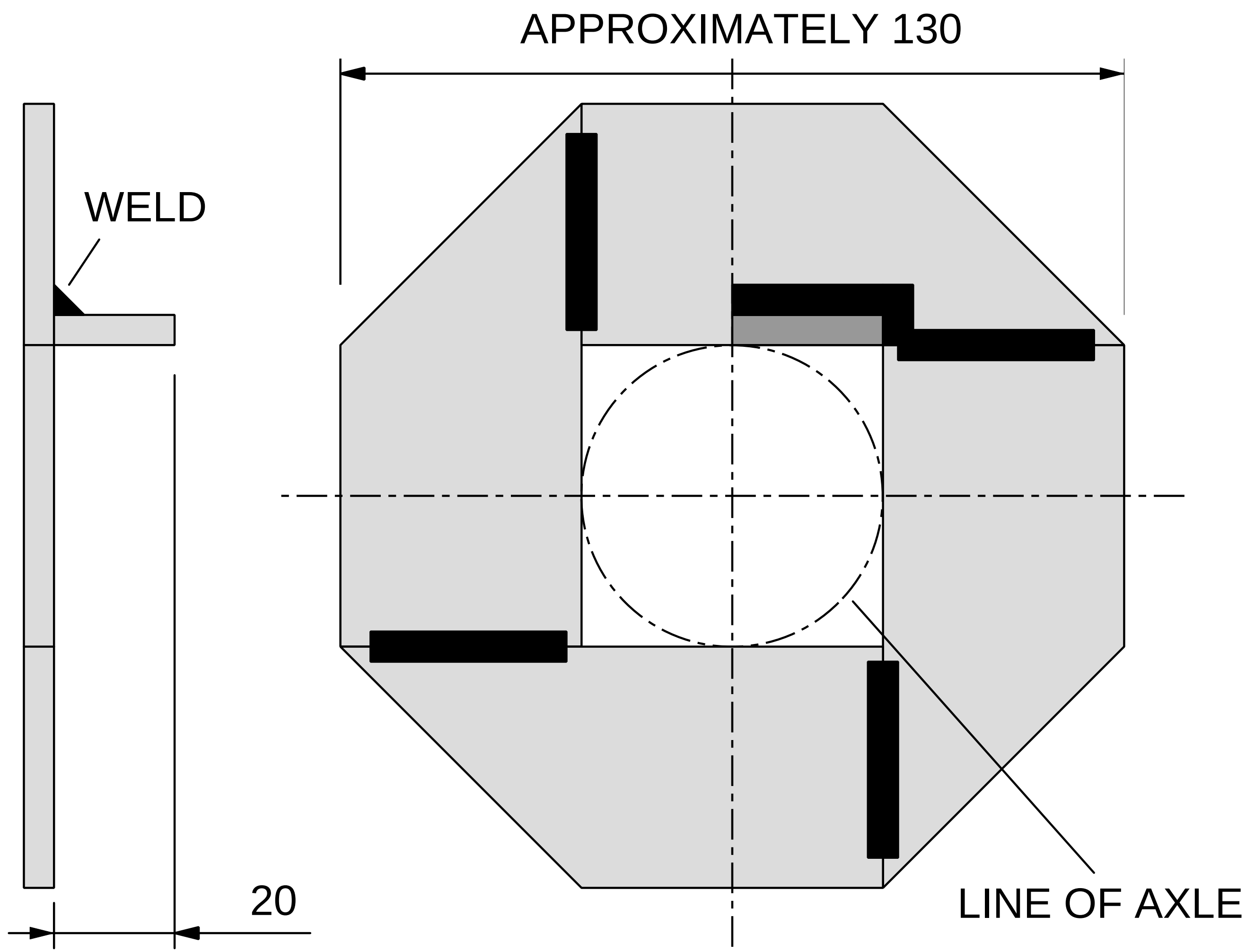
	GENERAL ASSEMBLY DRAWING 1	Scale	10mm	Title WOOD BEARING TWIN DONKEY CART AXLE	Drawn by	CEO
		Date	16-3-99		Dwg No.	1/4



GENERAL ASSEMBLY DRAWING 2	Scale	10mm	Title: WOOD BEARING TWIN DONKEY CART AXLE	Drawn by	CEO
	Date	16-3-99		Dwg No.	2/4



AXLE COMPONENTS	Scale	10mm	Title: WOOD PLAIN BEARING DONKEY CART AXLE	Drawn by	CEO
	Date	19-3-99		Dwg No.	3/4



AXIAL THRUST WASHER: 4 REQD

Scale	10mm	Title WOOD PLAIN BEARING DONKEY CART AXLE		Drawn by	CEO
Date	16-3-99			Dwg No.	4/4

KENDAT

DTU

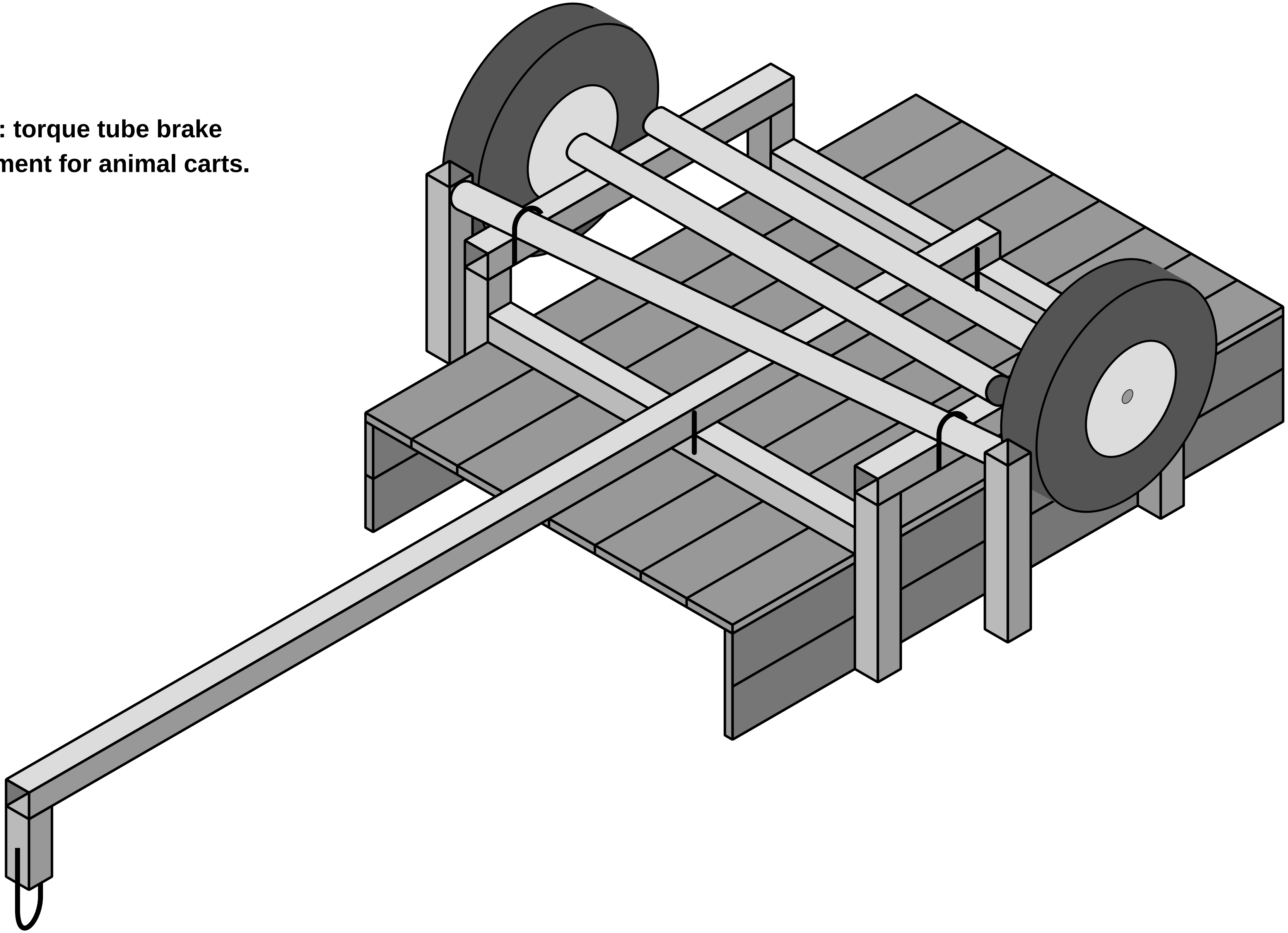
Animal Cart Programme

TECHNICAL
38
RELEASE

Simple Low-cost Brake for Donkey Carts

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: torque tube brake arrangement for animal carts.



Simple low cost torque tube brake for donkey carts.

Introduction

In this booklet we tell you how to make a brake for a steel framed DTU donkey cart from round and square steel tube. The instructions do not cover how to make the cart or the axle - you will need to read other Technical Releases from us to find out how to make these.

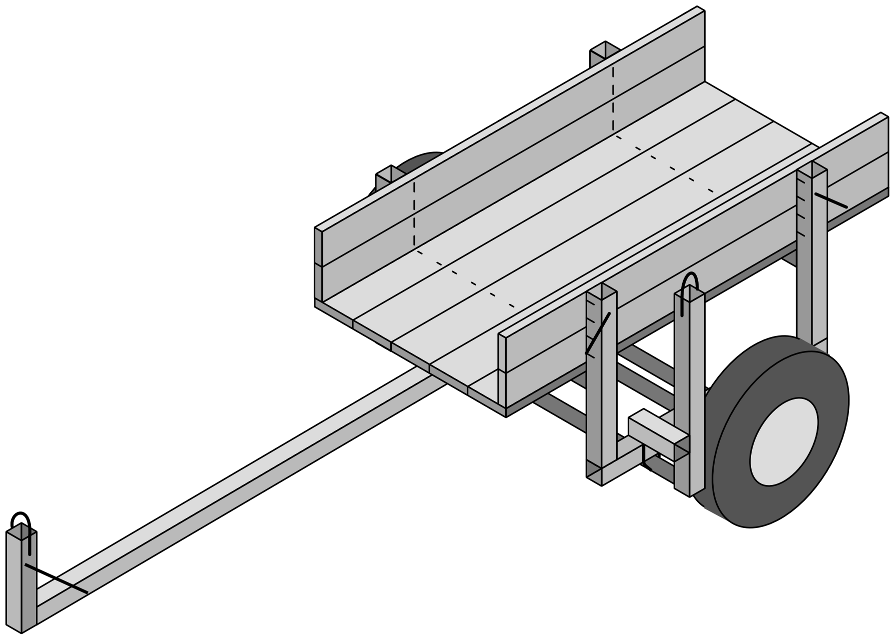


Figure 2: DTU donkey cart fitted with twin axles and simple low-cost brake.

You should find that you can make the brake for about £6. This cost will depend on the cost of the materials and labour. Once you get organised, two men can probably make and fit a brake in an hour.

Easy to make design.

This brake is designed to be constructed without any special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a simple welder, a hacksaw, and a hammer. We have deliberately designed the brake so that drilling is not required.

We have tested these brakes in Kenya and Uganda and we have had only a few criticisms - if the mud is very sticky it jams

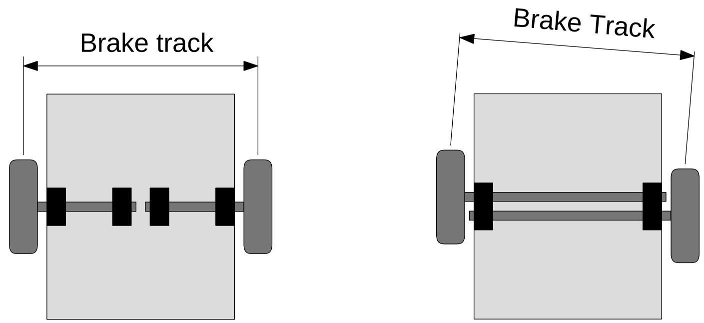


Figure 3: brake track measurement for a conventional half shaft axle and for a twin offset axle.

the brake and makes the cart hard to pull. One modification is to make the brake so it can be removed in bad conditions.

Cutting list and costs

Table 1 shows a cutting list for a brake - Recent prices of materials in Kenya are shown converted into £UK.

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete.

2) Start by measuring the required brake track as shown in Figures 3 and Figure 4.

3) Cut a piece of 1-½" black pipe to this brake track length **minus 50 mm** so for example if the brake track is 1500 mm make the pipe 1450 mm long.

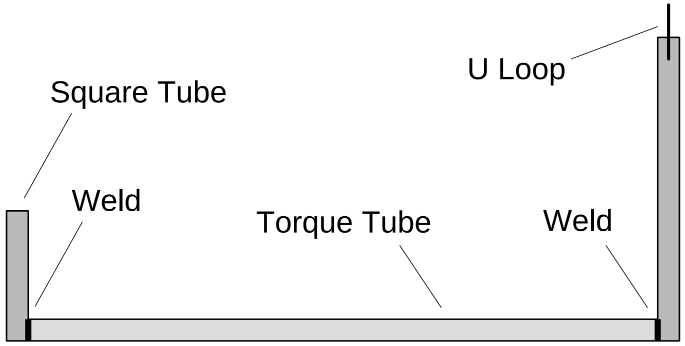


Figure 4: welded brake fabrication.

TABLE 1: materials for torsion tube brake.

component	material	# lengths reqd [#*mm]	total material for one brake [mm]	cost [UK£]
torque tube	1-½"" BSP malleable iron pipe	1 × 1500	1500	3.00
brake pad	50×50 mm square steel tube	1 × 300	300	0.69
brake lever/ pad	50×50 mm square steel tube	1 × 700	700	1.62
brake torque tube loops	R 12 = 12 mm re-bar	4 × 250	1000	0.31
			TOTAL =	5.62

4) Cut a piece of 50 mm × 50 mm × 3 mm wall thickness square tube 300 mm long and another about 700 mm long.

5) Weld the pieces of square tube onto the ends of the round pipe as shown in Figure 5. A trysquare will help you get it square.

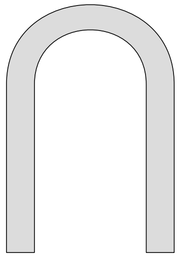


Figure 6: U loop.

6) Make five U shapes from R12 (that is 12 mm round steel bar) each piece 250 mm long as shown in Figure 6.

7) Weld the first U loop onto the end of the 700 mm square tube as shown in Figure 5. A rope tied to this loop can be used to work the brake remotely fo safety.

8) Now turn the cart upside down, put the brake in position and place the remaining four U-loops over the brake torque tube as shown in Figure 7 and Figure 8.

9) Once you have the brake beam in position, tack weld the ends of the loops to the axle support beam, check that the brake can be released clear of the tyres and then weld the loops on properly.

10) Next cut two pieces of R 12 about 25 mm long and weld them to the brake torque tube either side of a support loop so that the brake beam cannot move endwise.

11) You've finished it!

Other DTU cart developments

The DTU has been working on new designs of carts and all their components to bring down their costs and make things more locally manufacturable. It has designs for bodies, wheels, hubs, bearings and animal harness all available from DTU as Technical Releases.

Drawings

You will find two drawings on the next pages, the first gives a general view of the brake and the second gives a view of the components of the brake itself.

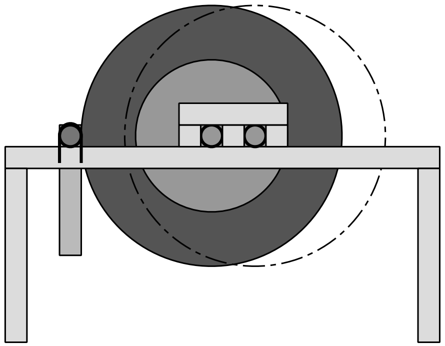


Figure 7: brake beam position.

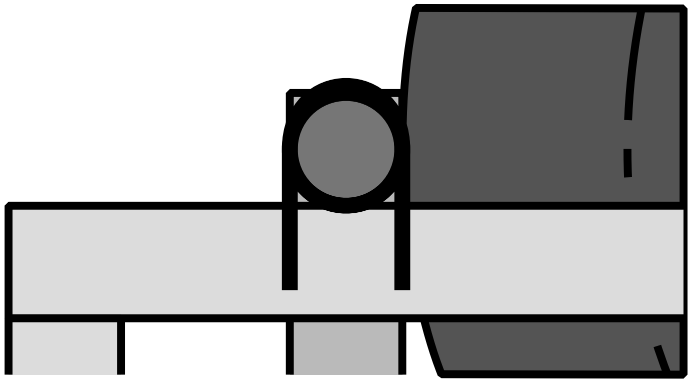
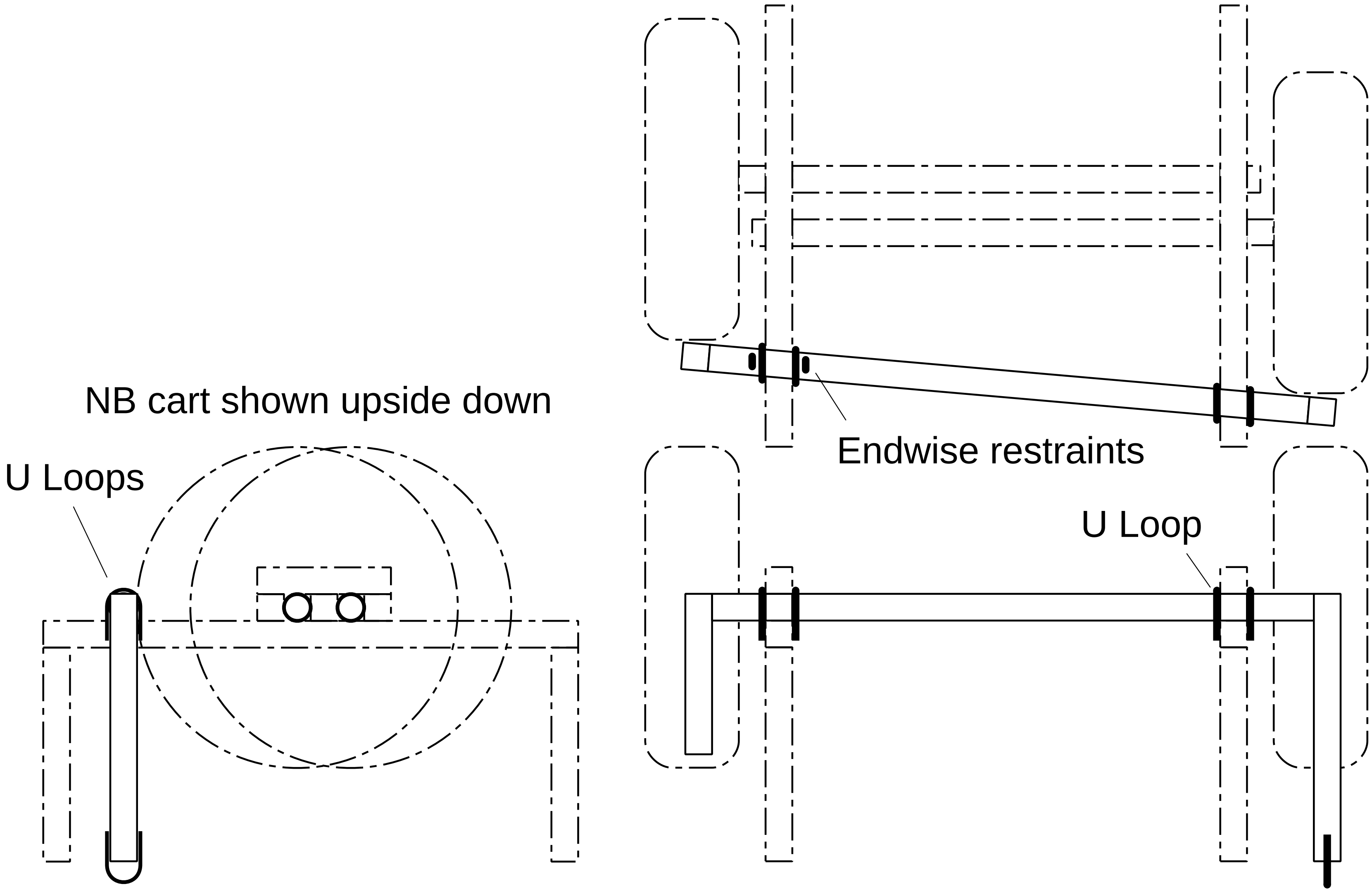


Figure 8: brake beam position - detail.

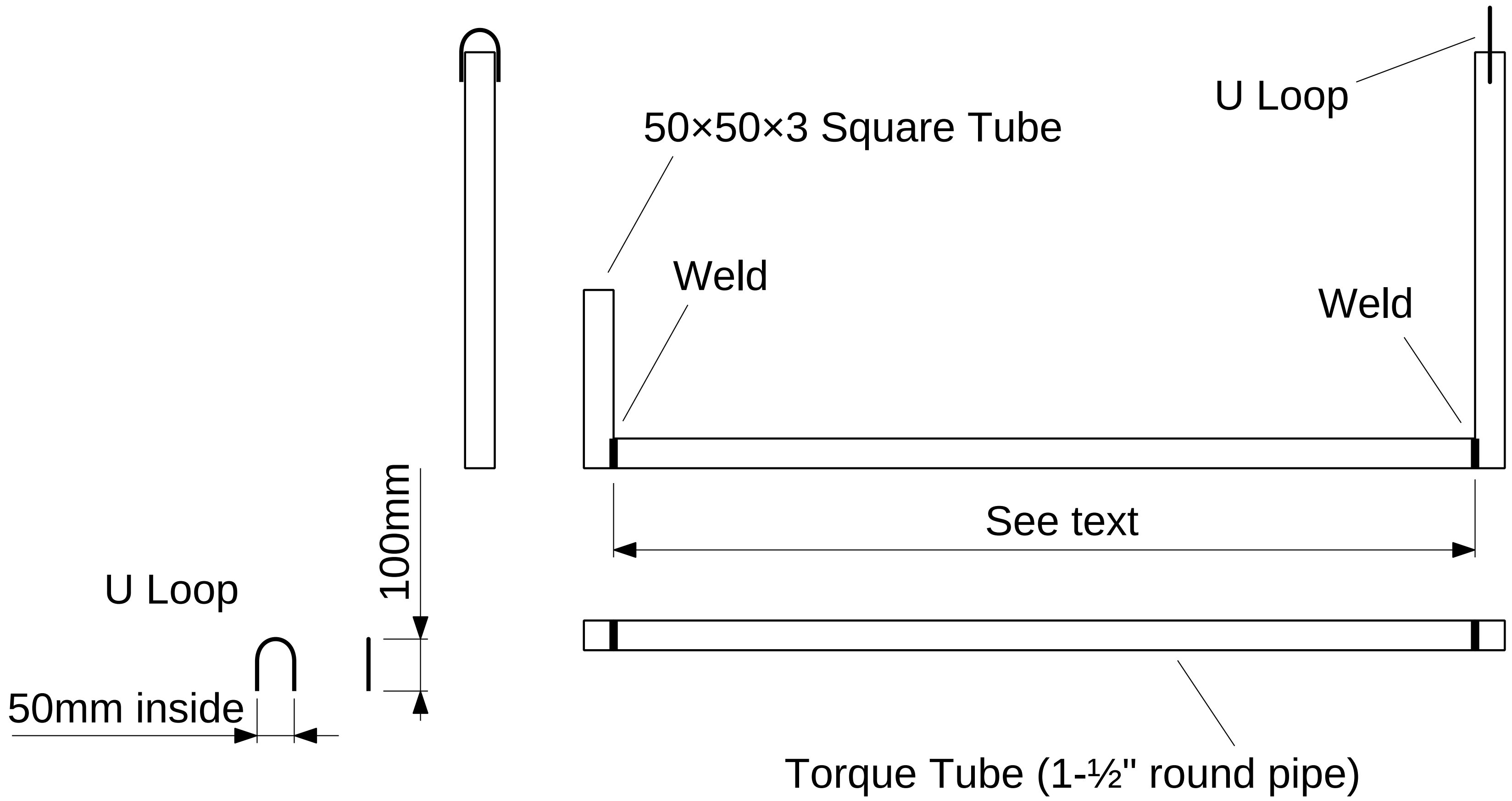
Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



Torque Tube Brake	Scale	10mm	Title TORQUE TUBE BRAKE FOR DONKEY CARTS	Drawn by	CEO
	Date	2-4-99		Dwg No.	1/2



Brake Components	Scale	10mm	Title TORQUE TUBE BRAKE FOR DONKEY CARTS	Drawn by	CEO
	Date	2-4-99		Dwg No.	2/2

KENDAT

DTU

Animal Cart Programme

TECHNICAL
39
RELEASE

Steel Wire Rim Wheel for Donkey Carts

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk

KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: wire rim wheel for animal carts.

Steel Wire Rim Wheel for Donkey Carts.

Introduction

In this booklet we tell you how to make steel wire or bar rim wheels for donkey carts. The instructions do not cover how to make the cart or the axle - you will need to read other Technical Releases from us to find out how to make these.

The advantages of this type of wheel are that it cannot be punctured and that it is quite easy to make.

You should find that you can make a wheel for about £12. This cost will depend on the cost of the materials and labour. Once you get organised, two men can probably make a pair of wheels in half a day.

Easy to make design.

This wheel is designed to be constructed without any special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a simple welder, a hacksaw, and a hammer.

Unfortunately we have only tested one of these wheels in Kenya and Uganda but we had no problems. A good modification is to cut the tread from an old tyre and bolt it to the outside of the wheel. Then it is quiet on tarmac roads.

Cutting list and costs

Table 1 shows a cutting list for a wheel - recent prices of materials in Kenya are shown converted into £UK.

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete.

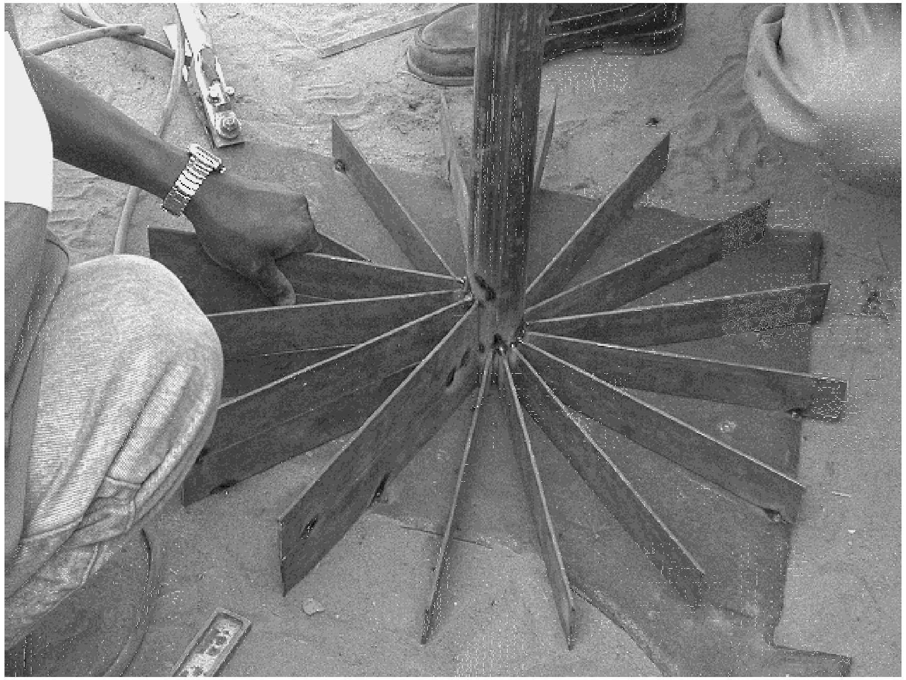


Figure 2: welding spokes to axle.

2) Start by cutting the 16 spokes each of which needs two pieces of flat bar. If you want to make the wheel an exact size subtract the axle radius and the wire diameter from the required radius for example if you want the diameter to be 600 mm ie a redius of 300 mm and the axle is 50 mm diameter and the wire is 10 mm diameter then the spokes should be 300 – 25 – 10 = 265 mm.

3) Next cut the axles from 1-½" black pipe. These will probably need to be about 1500 mm long.

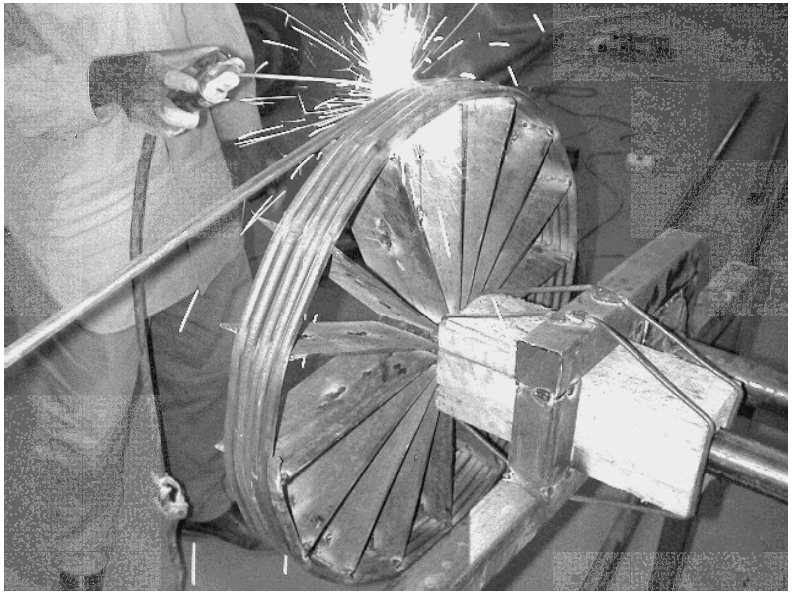


Figure 3: welding steel wire rim wheel for donkey carts. Also shown is wooden bearing.

4) Using a trysquare or a piece of wood or metal to check that the spokes are square to the axle weld the spokes evenly around one end of the axle as shown in Figure 5. Weld one on, then the opposite one, then one at rigth angles to the others, then the one opposite it and so on.

5) Then mount the axle horizontally as shown in Figure 3 so that it can rotate. Straighten the round bar out and then bend about 500 mm of one end so that it fits around the wheel spokes neatly. Weld the end of the wire to the end of a spoke so that it is flush with the outside of the spokes. Weld the wire to the next spoke keeping it flush with the outside of the wheel.

Continue welding the wire to each spoke in turn rotating the wheel as you go. Soon you will start bending the straight part of the wire, but you should find this ok. It is easiest if one person pushes the wire as another welds.

Continue pushing and winding and welding until the wire is the full width of the spokes. You will probably need to start a second length of wire to finish the wheel. Just weld it onto

TABLE 1: materials for torsion tube brake.

description	length m	#	total m	cost £UK
100 wide strip 600 dia single thickness	18.85	1	18.85	3.93
50×3 strip spokes 270 long	0.27	32	8.64	6.62
			TOTAL	10.55

the end of the first piece of wire and continue.

6) When you reach the other side of thewheel and you cannot get any more wire on you've finished it!

Other DTU cart developments

The DTU has been working on new designs of carts and all their components to bring down their costs and make things more locally manufacturable. It has designs for bodies, wheels, hubs, bearings and animal harness all available from DTU as Technical Releases.

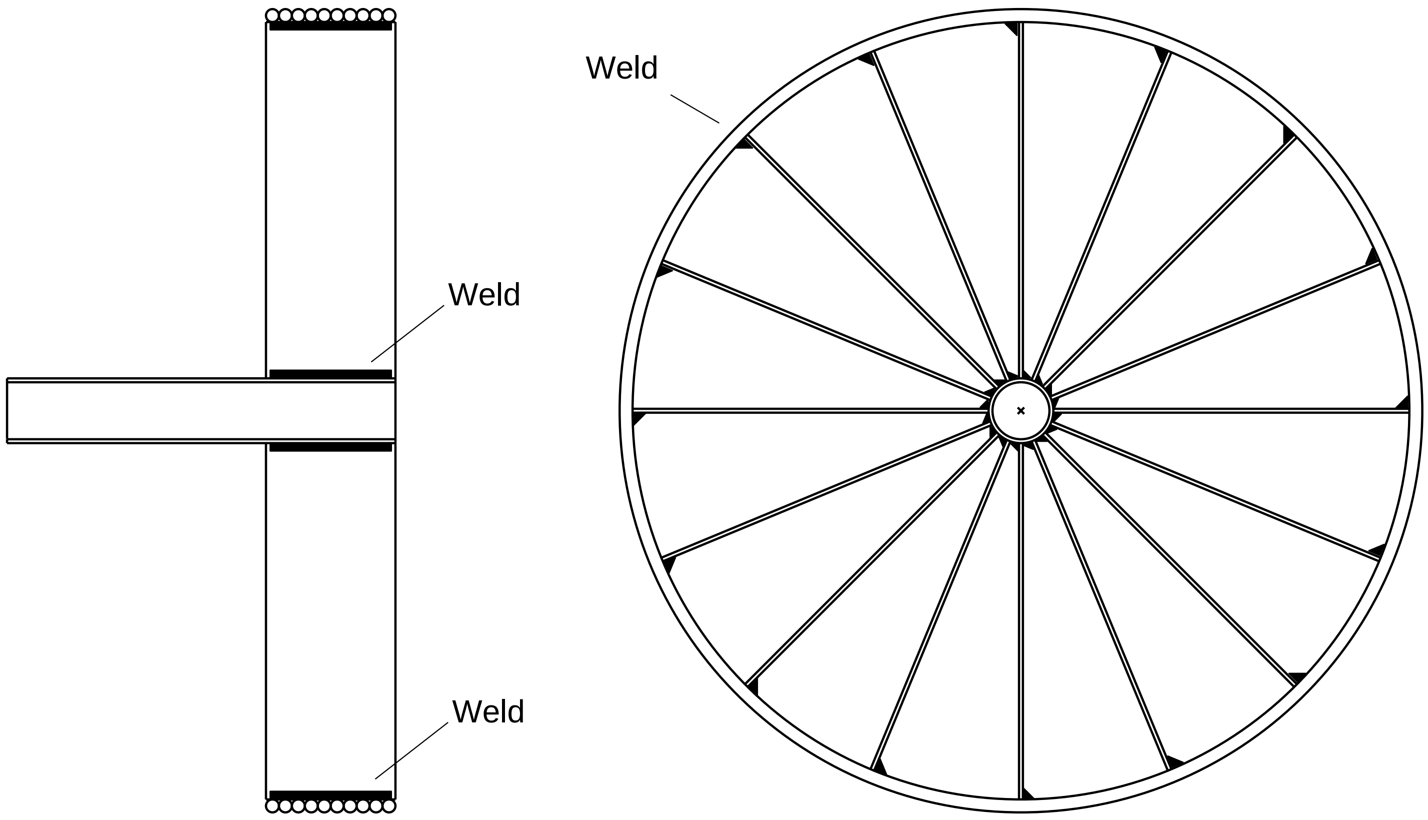
Drawing

You will find a drawing of the wheel on the next page.

Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



	General Arrangement	Scale	10mm	Title: WIRE RIM WHEEL FOR DONKEY CARTS	Drawn by	CEO
		Date	4-4-99		Dwg No.	1/2

KENDAT

DTU

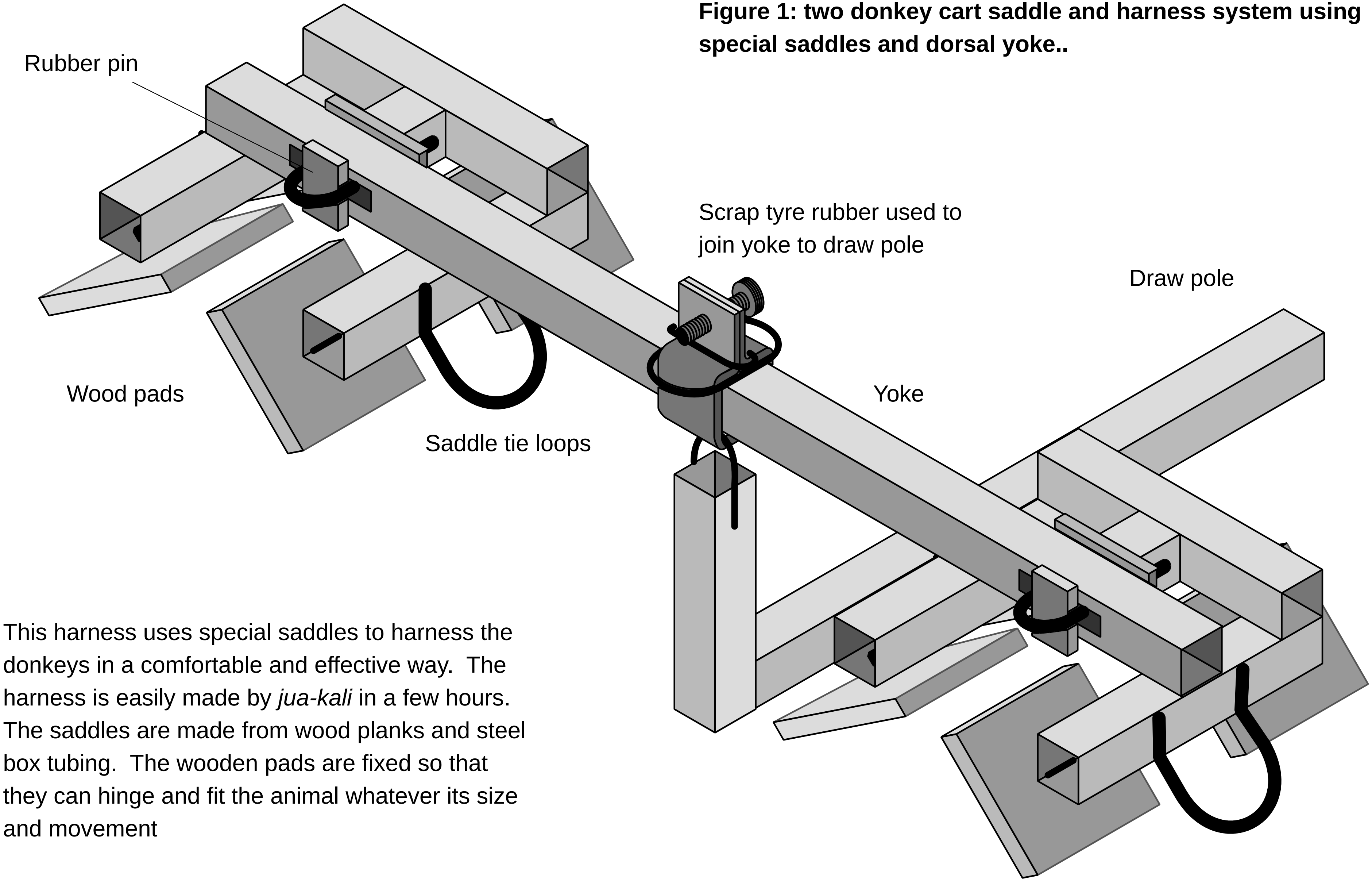
Animal Cart Programme

TECHNICAL
40
RELEASE

Double Donkey Harness for Cart Pulling

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)1203 523523 extn 2339, fax: +44 (0)1203 418922, email: dtu@eng.eng.warwick.ac.uk
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: two donkey cart saddle and harness system using special saddles and dorsal yoke..



This harness uses special saddles to harness the donkeys in a comfortable and effective way. The harness is easily made by *jua-kali* in a few hours. The saddles are made from wood planks and steel box tubing. The wooden pads are fixed so that they can hinge and fit the animal whatever its size and movement

Donkey Harness for Carts Made From Steel Box Tubing, Timber and Canvas/ Sacking

Introduction

This Technical release tells you how to make a saddle and harness system for two donkeys to pull a cart with a single draw pole. Another Technical Release tells you how to make a saddle for single animal use.

You should find that you can make the whole harness for two donkeys for less than £$_{UK}$15, depending on the cost of the materials and labour. Once you get organised, two men can probably make a complete set of harness in four hours - we have designed this harness to be easy to make.

Other booklets in this series tell you how to make simple low-cost axles and carts: we have designs for steel framed and wooden framed carts and for many different kinds of axle. All carts and axles can be made without special tools - even drilling metal is not required.

Idea Behind Design

Saddles are used in many countries to hitch animals to carts. Our saddles provide strong points on the animals' backs and a yoke can easily be fixed to them to carry the end of the cart

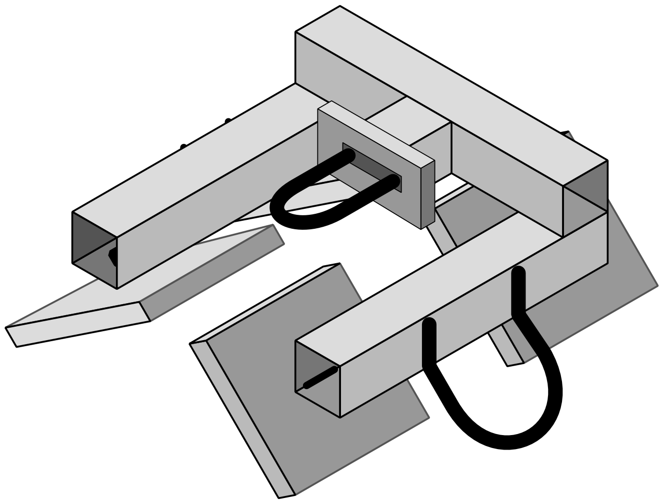


Figure 2: single donkey saddle.

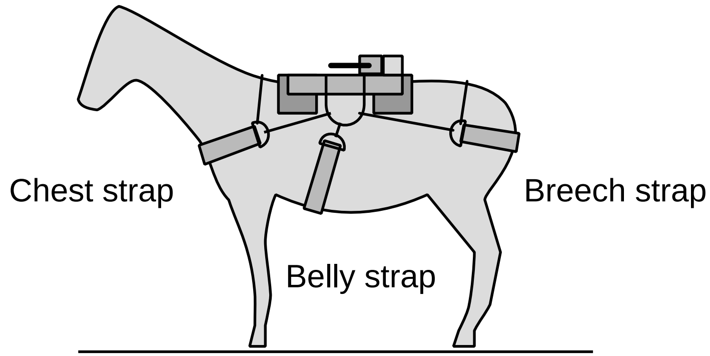


Figure 3: saddle secured to donkey with straps.

draw pole. Using this harnessing method carts can be pulled, steered and braked, and stabilised if the load is too far back on the cart body. This harness allows animals of different sizes to be used together and does not need them to walk exactly side by side. We have even had donkeys jump over a hedge pulling a cart with this harness!

Special tools and jigs and hard-to-get materials are not required. The only tools which you must have are a simple welder, a woodsaw, a hacksaw and a hammer.

The saddle frame is welded and the wooden pads are fixed to the frames with nails which are put through holes in the steel frame and welded so that they are loose and allow the pads to follow the shape of the animal.

These saddles have been tested in Kenya and work well but we would like to test them more. Really we need to test them for a year or two to see how the animals react.

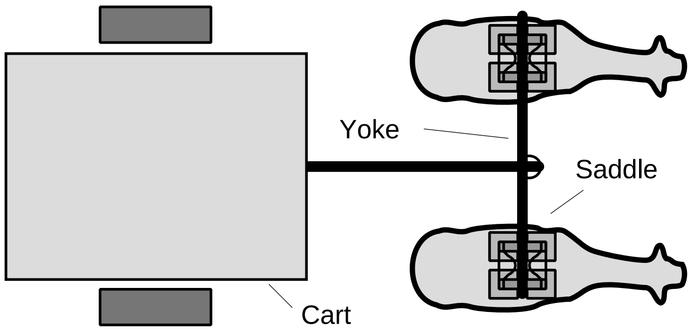


Figure 4: two donkeys harnessed to cart.

Cutting list and costs

Table 1 shows a cutting list for a complete harness - recent prices of materials in Kenya are shown converted into $£_{UK}$.

TABLE 1: harness materials cutting list.

component	material	# components	total mat [mm]	mat cost [$£_{UK}$]
main frames	50×50 box tubing	2×3×300	1800.00	2.11
yoke attachment stub	50×50 box tubing	2×1×60	120.00	0.13
yoke	50×50 box tubing	1×1×1400	1400.00	1.52
yoke fixing loops	12 mm re-bar	2×2×350	1400.00	0.45
strap loops	12 mm re-bar	2×2×300	1200.00	0.39
load pad pivots	12 mm re-bar	2×4×20	160.00	0.05
load pads	25×150 timber	2×4×150	1200.00	0.28
pad fix nails	nails/ 6mm re bar	2×8×50	800.00	0.08
strap rings	6mm re bar	2×6×180	2160.00	0.22
strap clenchers	6mm re bar	2×6×120	1440.00	0.14
strap hooks	6mm re bar	2×6×150	1800.00	0.18
straps	CC5 canvas	2×3×4×65	1560.00	3.94
strap chains	dog chain	2×3×300	1800.00	1.40
saddle/ yoke buffer pad	scrap tyre rubber	2×60×110	120.00	0.20
yoke/ saddle locking pin	scrap tyre rubber	2×90×100	180.00	0.20
yoke/ cart attach strap	scrap tyre rubber	1×70×400	400.00	0.50
			TOTAL =	**11.79**

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete.

2) Make up the U-shaped frame as shown in Figure 5. If you have a G clamp you can use it to hold two pieces of the frame together during welding.

3) Then weld the tie loops and the yoke attachment stub and loop onto the U frames so that the frame looks as shown in Figure 6.

4) Next cut the wooden load pads and round off all the edges so that there are no sharp corners to stick into the donkey. Hammer two nails through each of the pads in the positions shown in the drawings. With some timbers you may need to drill holes for the nails to avoid splitting or burn the holes with a hot nail. Then cut the nails so that about 30 mm projects from the timber as shown in Figure 9.

5) Now mark the position of the holes required to accommodate the pad nails in the steel tubing. These holes should be 15 mm and 65 mm from the ends of the square tube as shown in Figure 7. Blow the holes through with the welder at maximum current setting or use an angle grinder or file or hacksaw.

6) Next you can weld on the pad pivots blocks as shown in

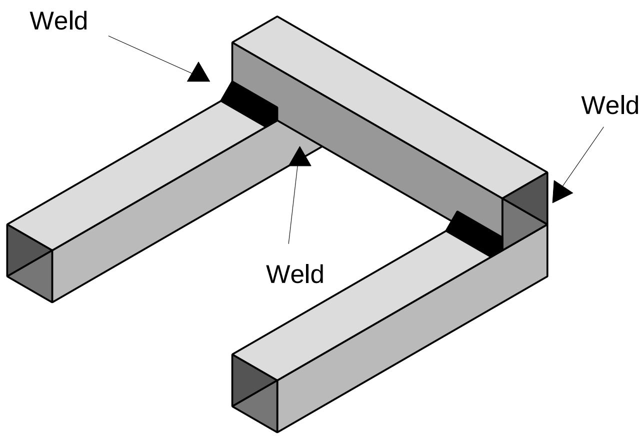


Figure 5: welding of frame cross piece.

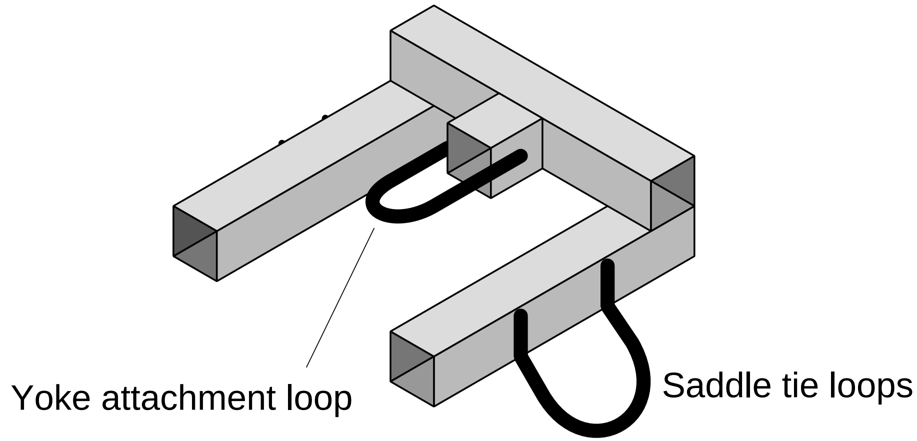


Figure 6: links and yoke loop welded to U- frame.

Figure 7.

7) Now put the nails through the blown holes and weld a piece of nail across the ends of the nails as shown in Figure 10. Welding down inside the tube looks difficult but skilled workers can weld the pads in about one minute.

An alternative way of doing it is to cut slots 70 mm long along the corners where the holes would be as shown in Figure 8. The slots should be 8 mm wide so that the nails are very loose in them. Make up the pads as shown in Figure 9, put the nail loop into the slot in the right place and weld the pad pivot blocks into place across the slot so that they are in the same place as in Figure 7.

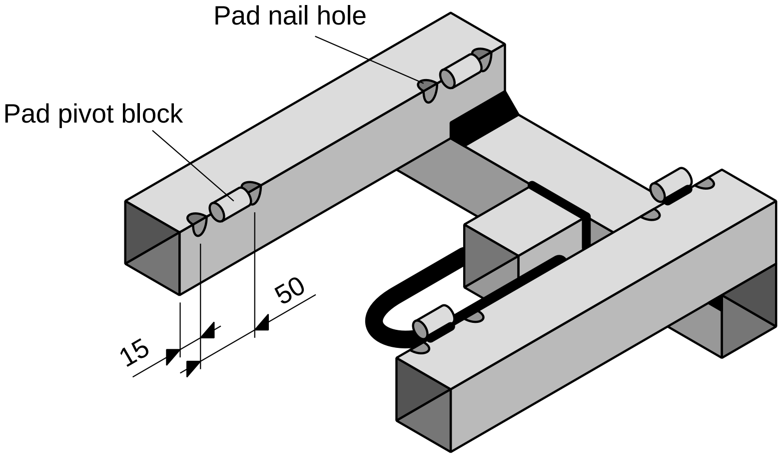


Figure 7: pad nail hole positions and pad pivots.

8) Next you need to make up the six straps to hold the saddles onto the donkeys. The D rings at the end of the straps can be made from 6 mm diameter concrete reinforcing bar as shown in Figure 11. A separate piece of the re-bar is clenched over the strapping using hammer blows to fix the D rings to the ends of the straps as shown.

The straps themselves can be made from heavy canvas or hessian sacking. You should use three or four thicknesses of material for them to make them strong enough and soft enough not to hurt the donkey.

9) Make the strap chain hooks from more 6 mm re-bar as shown in Figure 12 and fit the fixed ends to the saddle tie loops.

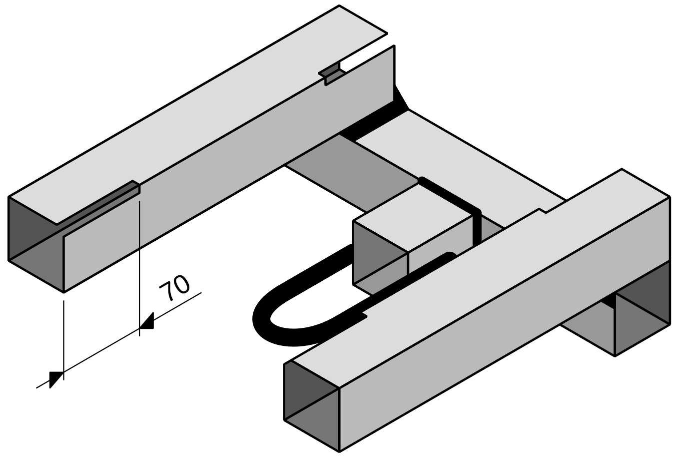


Figure 8: pad nail slots.

10) Cut the yoke to length. You can make the yoke different lengths but we have found that a longer yoke helps the animals turn in a narrow track. Make the slots in the ends with a grinder and welder and file. Weld the central tie loop shown in the drawings.

11) Cut two rubber yoke fixing pins and the buffer pads to the shape shown in the drawings.

12) Cut another piece of scrap tyre rubber about 400 mm long and 70 mm wide. Make a hole about 15 mm diameter about 40 mm from each end. Make a loop of 6 mm diameter steel reinforcing bar about 70 mm long as shown in Figure 14 that will just go over two thicknesses of the rubber strip.

13) Paint and creosote the saddle. You've finished it!

Method of harness use

Harness each animal separately, then fix the yoke to the saddles, then fix the yoke and animals to the cart.

1) First put a blanket or two folded hessian or jute sacks (not plastic) onto each donkey's back to protect them.

   Remember that protecting the donkey will save money because it can work harder if it is comfortable and will not get sick from skin wounds.

2) Put the saddle on so that the fronts of the wood pads are about 100 mm behind the animal's shoulder blades. This

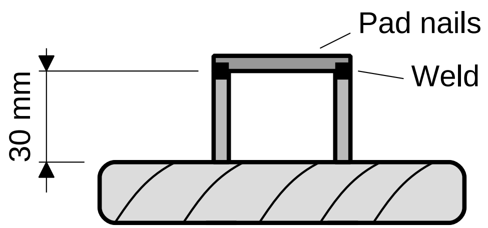


Figure 9: welded pad nails.

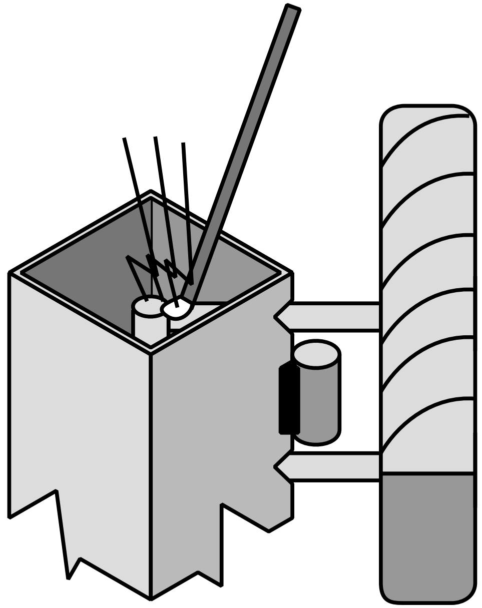


Figure 10: welding re-bar to frames for wooden load pads.

means that the saddle should never come near parts of the animal's back which move.

3) Next hook the breaching strap to the loops hanging from the side of the saddle. It should be tight enough to tend to pull the saddle a little rearwards. Make sure that the breaching strap is pulled up high so that it does not rub the backs of the legs. But it should not be so high that the animal cannot defecate. Tie a piece of thin rope across the animal's back between the rings of the strap to hold the strap up.

4) Now hook the chains for the belly strap onto the hooks fixed to the saddle tie loops. The strap should be 50 mm or 100 mm behind the front legs - check that the legs do not rub on the strap when the animal walks. Tighten the strap so that you can just get a couple of fingers under it between the strap and the animal. This will be much tighter than the other straps.

5) Hook the chest strap to the loop and adjust the tension so that it is a little loose. Use another short piece of rope to hold the chest strap up so that it is just below the wind pipe. The strap goes tight when the animal pulls really hard. We have noticed that the belly strap and breaching strap are nearly enough without the chest strap and so we leave the chest strap a bit loose.

6) Saddle the second animal in the same way. You should be able to saddle an animal in only a few seconds when you

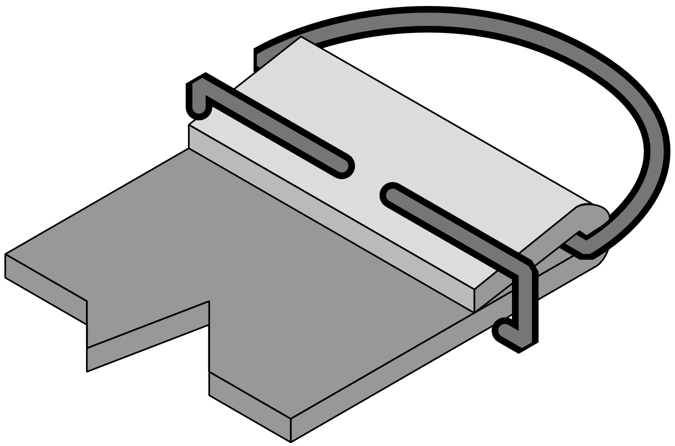


Figure 11: D rings for straps made from re-bar.

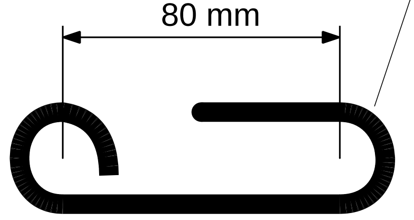


Figure 12: chain hooks for straps.

get practised.

7) Get the two donkeys in position side by side and put the yoke across the two saddles pushing the slots in the yoke over the loops on the saddles as shown in Figure 1. Secure the yoke by pushing the rubber pins into place.

8) Lastly connect the cart to the centre of the yoke using the 400 mm long rubber strap, the 80×15 loop and a bolt as shown in Figure 13.

9) You are ready to go!

Saddle Drawing

You will find drawings of the saddle and yoke on the last pages

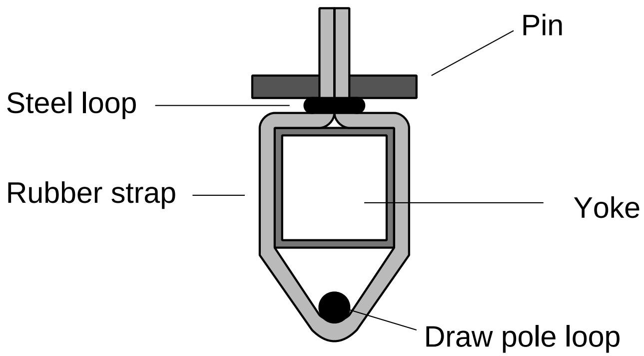


Figure 13: using rubber strap to join cart draw bar loop to yoke.

of this Technical Release.

Other DTU cart developments

The DTU has been working on a range of cart designs for use with both donkeys and oxen. It has designs for wooden and steel framed types. You can make either type of cart in only a few hours, if you are reasonably set up with tools and materials.

The DTU has also been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. It has a system of axles with bearings made from PVC pipe, another with wooden bearings and a third using scrap ball bearings. None of these axles need machining and they only take two men a day to make.

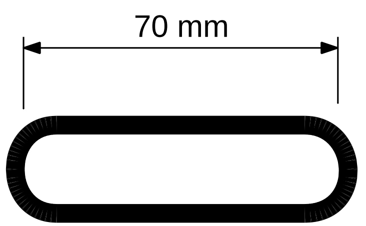
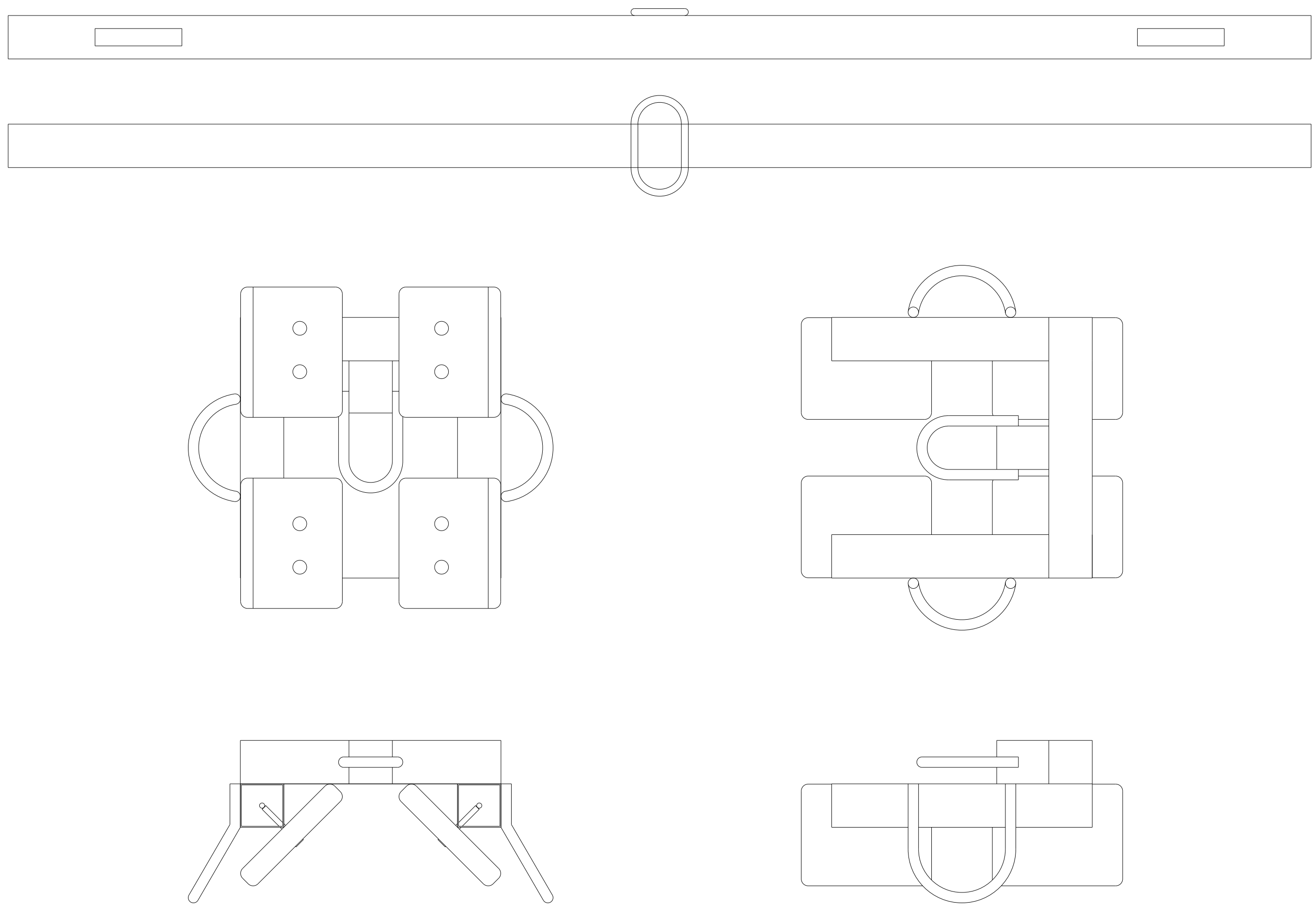


Figure 14: loop for yoke to drawpole strap (R6 steel bar).

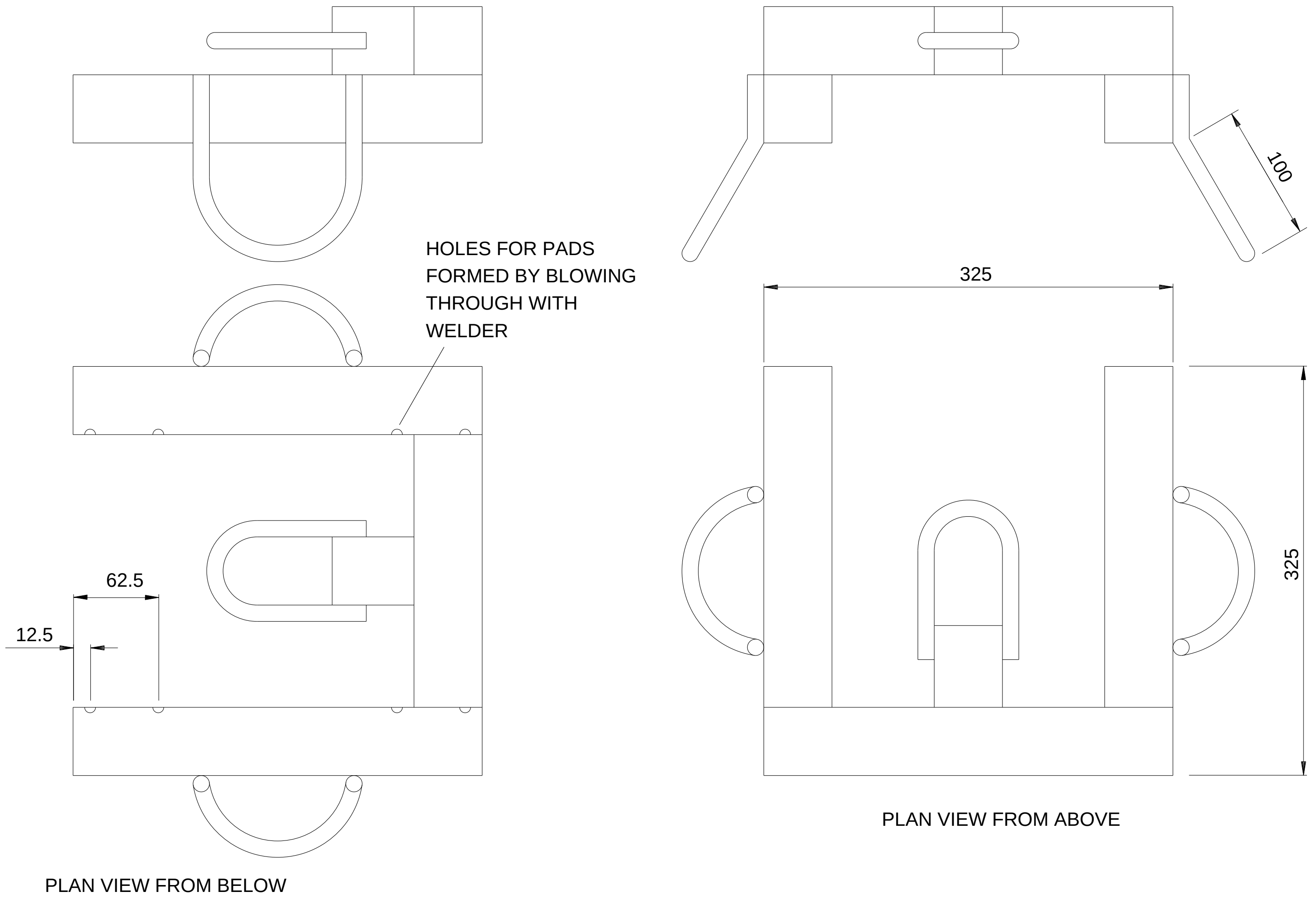
Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

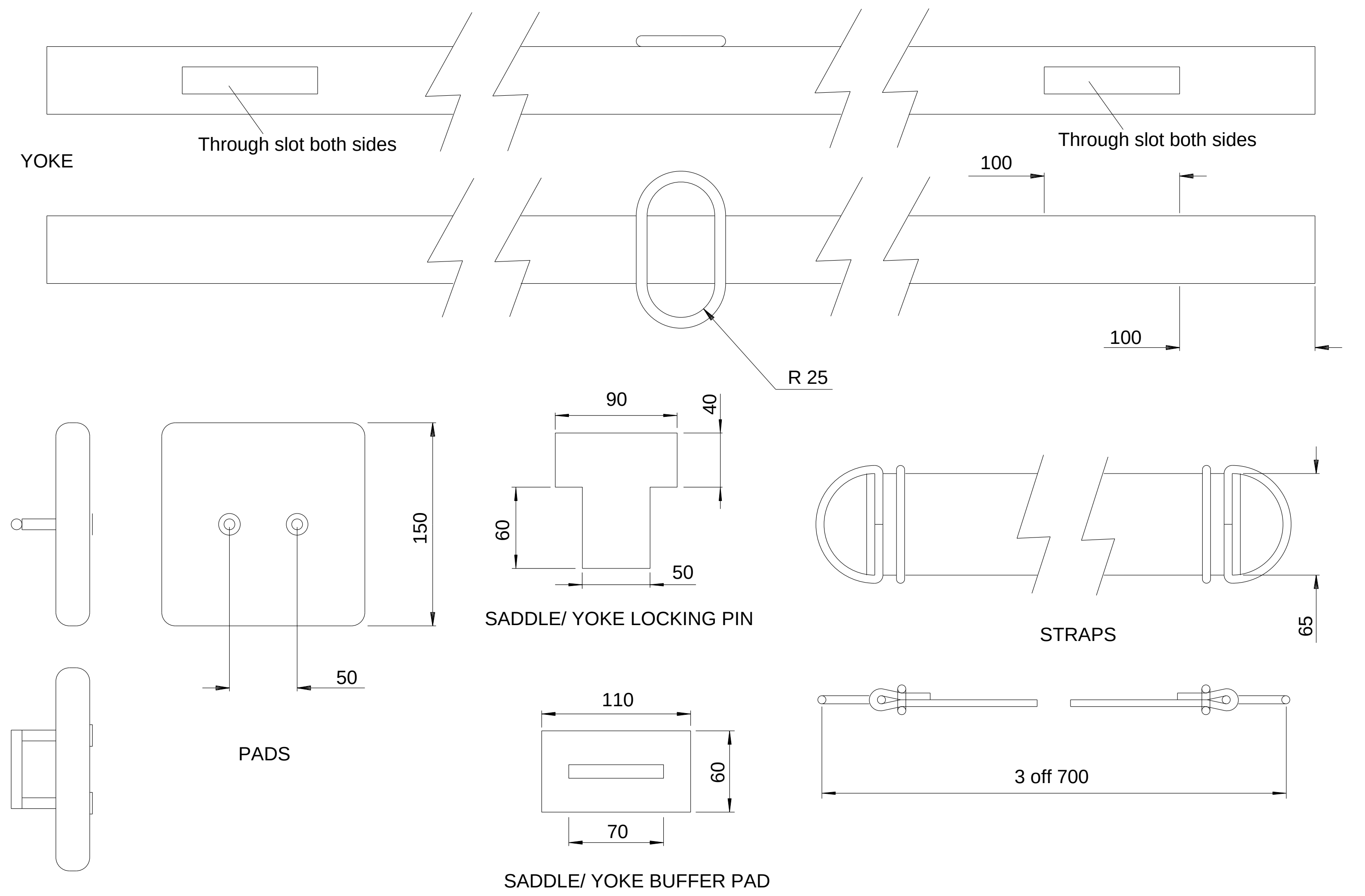
The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



Scale	50 mm	Title	Drawn by	CEO
Date	9/1/99	Donkey saddle	Dwg No	1/3



Scale	50 mm	Title Saddle for donkey carts - frame	Drawn	CEO
Date	9/1/99		Dwg #	2/3



					Scale	50 mm	Title Yoke, saddle pads, straps, for donkeys	Drawn	CEO
					Date	9/1/99		Dwg #	3/3

KENDAT

DTU

Animal Cart Programme

TECHNICAL

41

RELEASE

Twin Ball Bearing Axle for Donkey Carts

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk

KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Novel bearing arrangement for animal carts using scrap (or new) ball-race, running on outside of full width axle. No machining or accurate work required. Bearing may be replaced with different size. Only one bearing per wheel required.

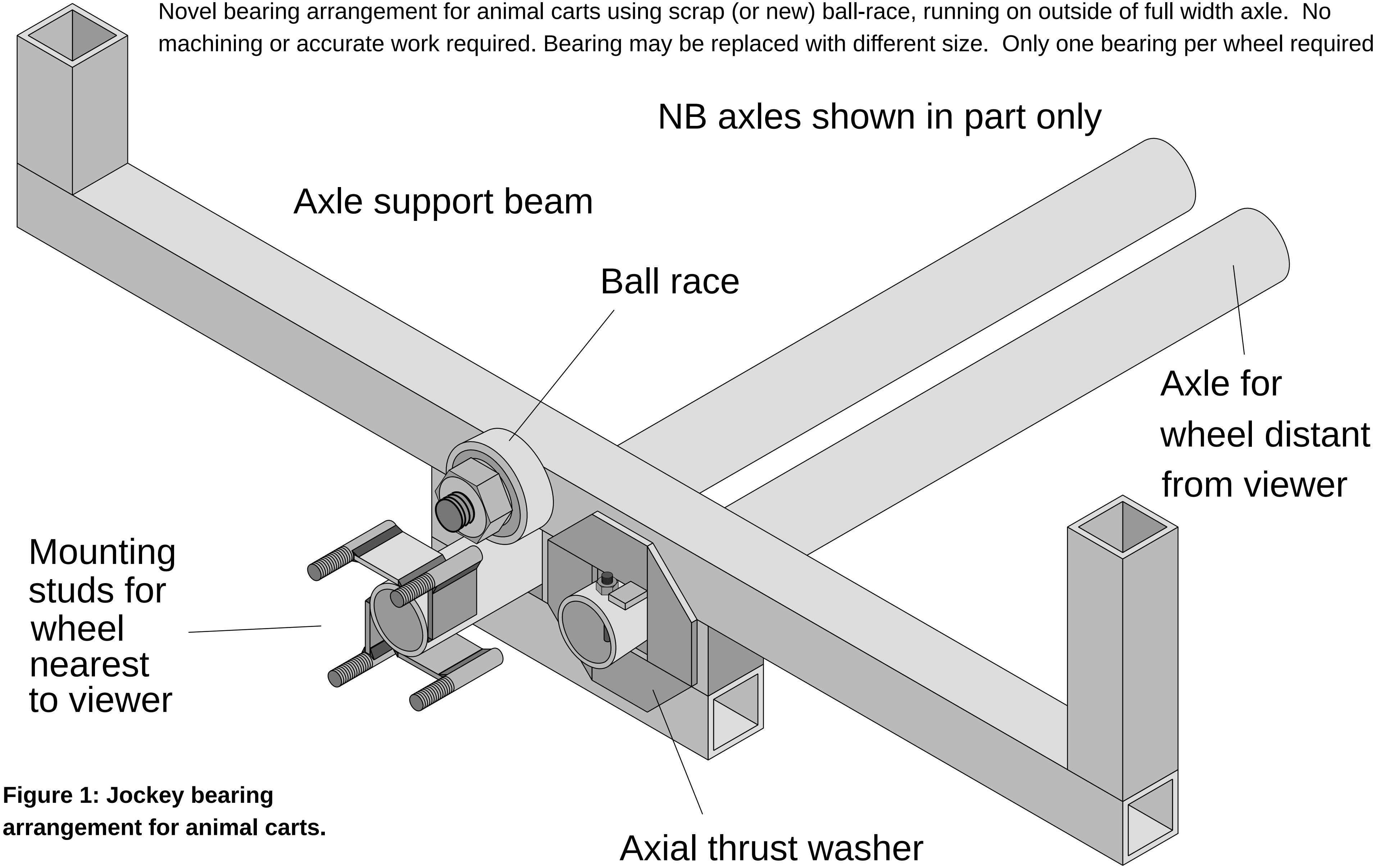


Figure 1: Jockey bearing arrangement for animal carts.

Twin axle system for donkey carts using scrap/ new ball bearings.

Introduction

In this booklet we tell you how to make an axle system for a simple donkey cart from round steel tube and ball bearings. The instructions do not cover how to make the cart itself - you will need to read other Technical Releases from us to find out how to make the carts.

You should find that you can make the axle system for about £40 including the wheels, tubes and tyres. This cost will depend on the cost of the materials and labour. Once you get organised, two men can probably make and fit one cart with axles in half a day. This is quite a lot faster than it takes to find and a scrap car axle and it will be much cheaper.

In other booklets in this series you can find out how to make other low-cost axle systems and carts.

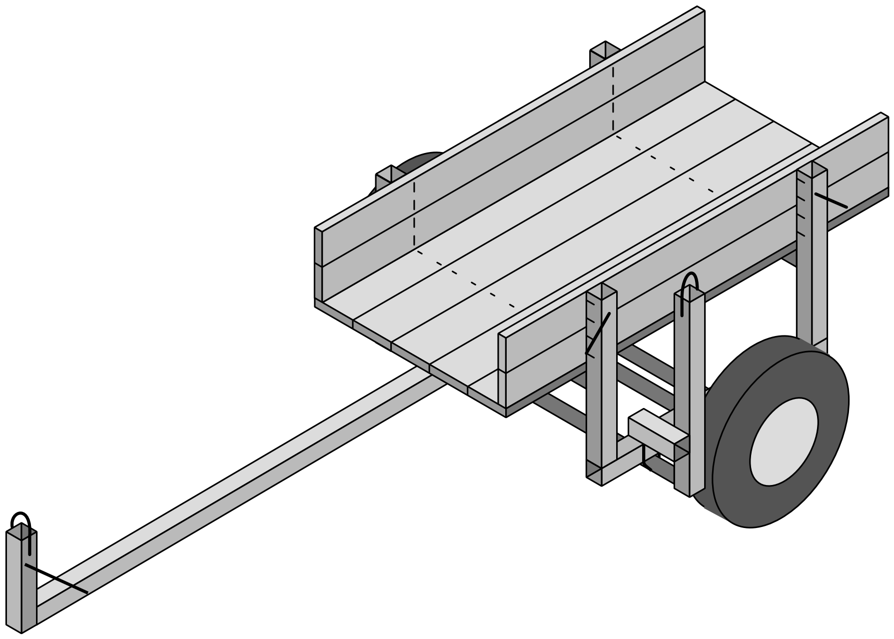


Figure 2: DTU donkey cart fitted with twin axles and PVC bearings.

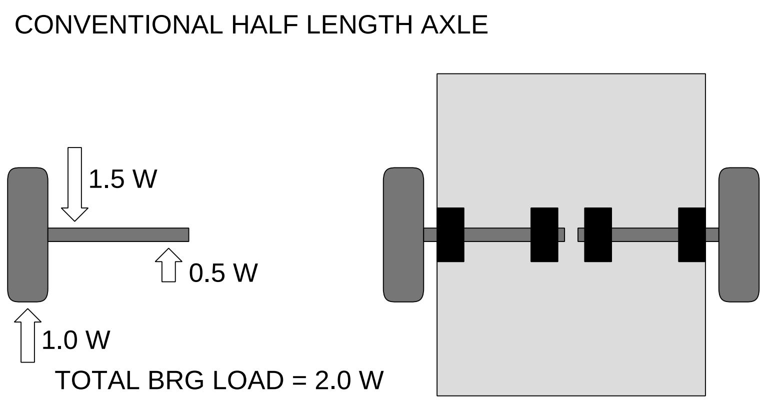


Figure 3: bearing loads in conventional half shaft axle.

Why have twin axles?

There are two types of axle: fixed axles and stub axles. In a stub axle the wheel-hub rotates on a stationary axle. In a live axle arrangement the axle revolves in stationary bearings.

With the stub axle types the bearings must be inside the wheel. This is easy with expensive ball bearings in a machined hub but more difficult to do in a simple workshop. Really you need two ball bearings per wheel as well. You also need to make things quite accurately or make the hubs quite long to stop wheel wobble. Another problem is that the hubs stick out of the wheel and catch on animal and human legs.

If you would really prefer to make a stub axle we have quite a good one using plain bearings made from PVC pipe. This axle also has the advantage that you can take the wheel off without a spanner. We also have a system of making your own roller bearings and we can send you Technical Releases on how to make these axles, but we think the twin axle system here is easier to make and a bit better.

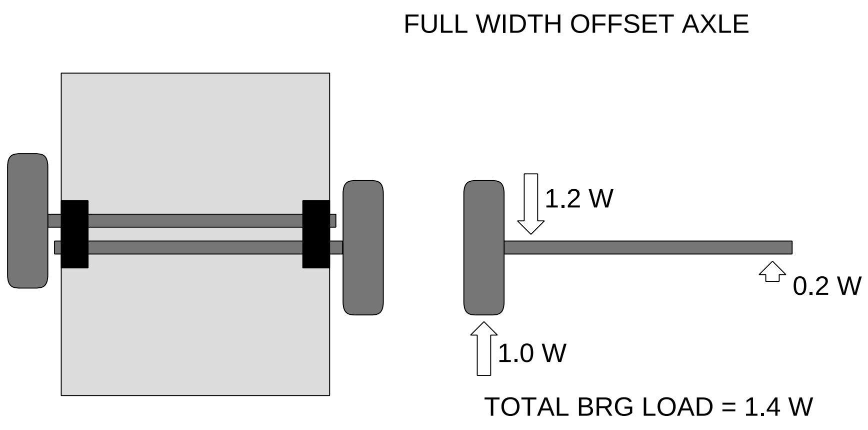


Figure 4: bearing loads in twin offset axles.

Long twin axles reduce bearing loads and require less accurate manufacture. Figure 3 shows the bearing loads of short axles and Figure 4 shows the twin axle method. You will see that bearing loads are about 30% lower. Surprisingly there is no extra steel required either because there would have to be some steel to support the middle bearings anyway.

Easy to make design.

These axles are designed to be constructed without any special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a simple welder, a hacksaw, and

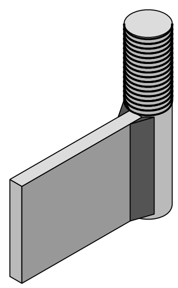


Figure 5: a welded wheel stud and strut fabrication.

a hammer. You might find that a couple of 4" or a 5" G clamps (or something like it) are useful too. We have deliberately designed the axle so that drilling is not required.

We have tested many of these axles in Kenya and Uganda and we have had only a few failures caused by poor welding or incorrect material. We think that they are strong enough, but you can always find someone to break anything. To get a reasonable cost you need to experiment a bit to see how the farmers treat their carts and what they expect them to stand.

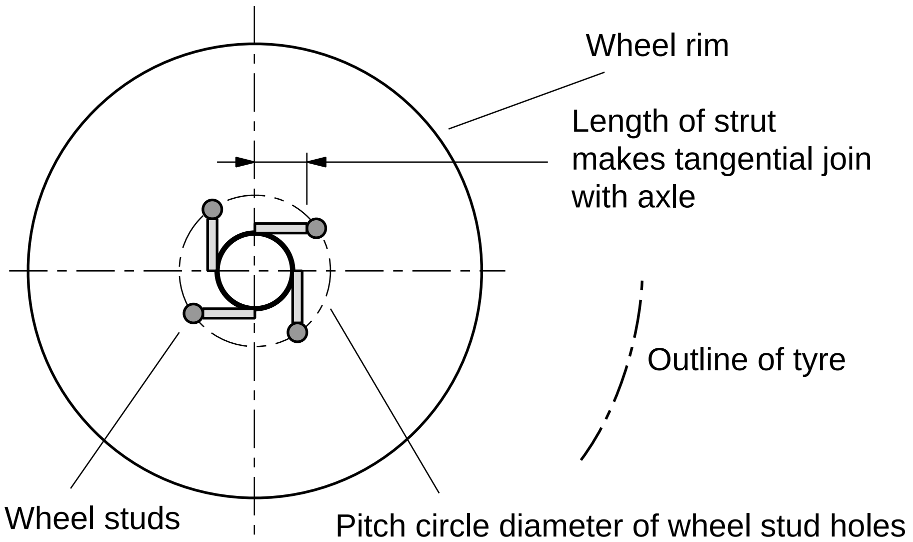


Figure 6: length of wheel strut.

Cutting list and costs

Table 1 shows a cutting list for a complete axle - Recent prices of materials in Kenya are shown converted into £UK.

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete.

2) Start by getting the two ball bearings you want to use and cleaning them. If they have rubber or steel shields it is probably best to leave the shields in place, but if they are open clean the bearings in petrol or diesel fuel or kerosine. Then re-grease them.

TABLE 1: materials for ball bearing twin axle.

component	material	# lengths reqd [#*mm]	total material for one axle [mm]	cost [UK£]
wheel studs	50×M12 nuts and bolts	10	10	2.60
wheel stud struts	6×40 BMS strip	10 × 37	650	0.49
axial thrust washers	6×40 BMS strip	8 × 90	720	0.54
axle cross bolts	75×M12 nuts and bolts	4	4	1.04
axles	1-½" BSP malleable iron pipe	2 × 1500	3000	6.00
axle reinforcements	1-½" BSP malleable iron pipe	2 × 30	60	0.12
main bearing	scrap ball race eg 6205, 6206	2 off reqd	2	5.00
ball bearing mounting bolts	M24×50 or similar	2 off reqd	2	2.00
bearing box sides	50×50×3 mm square steel tube	6 × 53	318	0.73
bearing box top	50×50×3 mm square steel tube	2 × 252	504	1.16
wheel rims, tyres + tubes	na	2	2	25.00
			TOTAL	**44.69**

3) Now you need to get a large nut and bolt for each bearing. Ideally the bolt should just go through the middle of the bearing, but it can be quite loose as long as the bearing can be held very tightly when the nut is tightened.

If you cannot get a nut and bolt you can use a piece of pipe and a welded ring.

4) Next make the wheel stud struts shown in Figure 5. You need to make one of these struts for every stud hole in the wheels you are going to use. Figure 6 shows how to measure the length of the struts. The struts are made from 6×40 flat bar or similar and M12 bolts 50mm or 60mm long. The flat bar should be long enough so that it meets the axle tube tangentially as shown in Figure 6.

5) Once you have made these struts, screw a nut onto each one until it touches the 40×6 metal. Then put the thread through the hole in the rim and screw another nut onto the thread. Tighten this nut lightly with a spanner. Repeat for all the struts so that they all point the same way round the axle, as in Figure 6, and leave a gap for the axle.

6) Now centre the axle in the rim and get it square using a tape measure, a trysquare and a plank or piece of steel resting on the tyre.

Put the wheel rim on the floor and put the axle in place in the middle. You should put something under the end of the pipe to get it in the right position as shown in Figure 7. Get an assistant to hold the top end of the pipe and tell him to

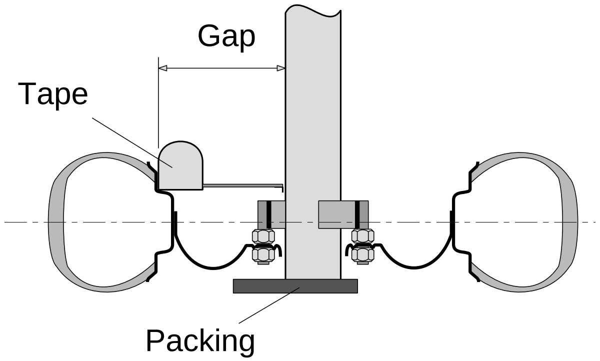


Figure 7: using tape measure to centre axle in wheel.

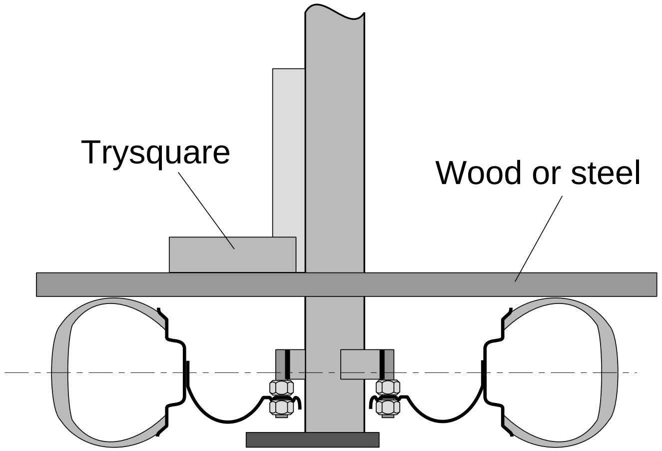


Figure 8: using trysquare to get axle square to wheel.

keep very still! Use your tape to measure from the outside of the pipe to the inside of the rim as Figure 7 shows. Measure in one place and then measure the gap opposite. Move the axle pipe over until it is central. Repeat this for the other direction at right angles. You could use wooden wedges as shown in Figure 9 to hold it.

Now use the trysquare and a piece of wood to get the axle square to the rim as shown in Figure 8. You put the wood on the tyre or rim so that it is flat and you put the trysquare on the wood. You have to move the axle until it is straight with the trysquare and your assistant must hold it without moving. Check it several times - its hard to correct afterwards!

7) Once you have it in position, tack weld the ends of the struts to the axle tube as shown in Figure 9. Then weld the struts on properly. Do as much welding as you can without taking the axle out of the wheel because the metal changes size as it heats and cools and it may move out of place.

8) Next cut six 53 mm pieces of square tubing for the bearing boxes. Mark the centre of the axle support beam of the cart and put marks on the beam 25 mm and 125 mm either side of this. You need to weld the 53 mm pieces on to the axle beams as shown in Figure 10. Weld a 252 mm piece of

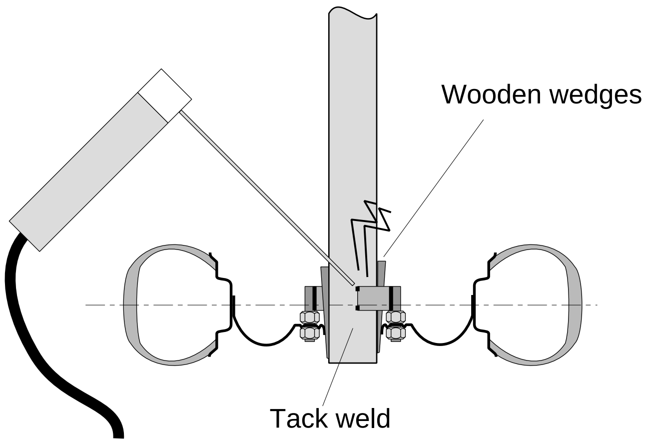


Figure 9: tyre, wheel and axle tube during tack welding stud support struts

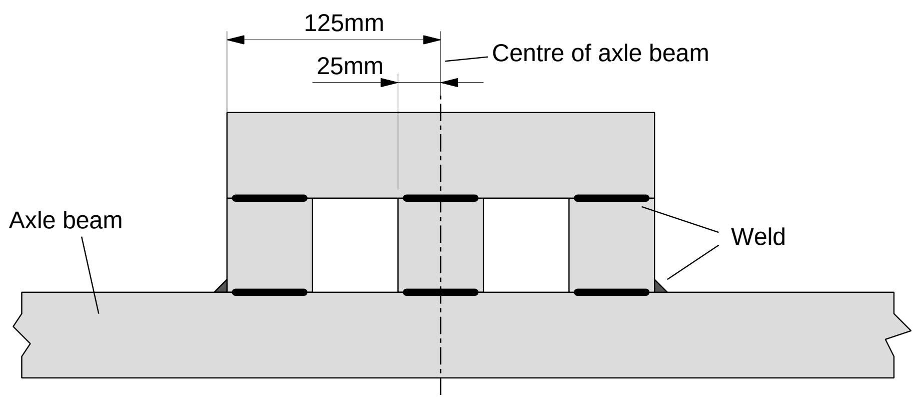


Figure 10: position of bearing support blocks - nb the cart is upside down here.

square tubing across the ends of the 53 mm pieces as shown in Figure 10.

9) Make four axial thrust washers from 40×6 or 40×3 or similar flat bar like those shown in Figure 12. You must remember to weld on a tag made of a 20 mm length of bar to each ring as shown in the drawings. This makes the washers go round with the axle and stops wear in the wrong places.

10) Assemble the axles and thrust washers as shown in Figure 13 but without the crossbolts. Put the wheels onto the axles, lightly tighten the nuts and position the axles so that there are 50 mm gaps between the tyres and the axle beams.

11) Now mark the position of the cross bolt holes. Remember that the nuts will have to be turned so do not make the holes too close to the thrust washers - centre about 15 mm away is fine. Use the welder to blow the holes.

12) Cut the excess axle off about 40 mm from the thrust washer. To mark a line around the pipe to cut it square, wrap a strong piece of paper or thin card around the pipe, get the edge in line and use the edge to guide the felt tip pen or scriber as you mark the line.

13) Mark the place on each axle where the bearing will roll. Cut 30 mm pieces of pipe, slit them so they can be opened and placed on the outside of the axles and weld them in place where you have marked.

14) Now assemble the ball bearings on their bolts and lightly tighten the nut. Position each bearing under the middle of the axle as shown in Figure 11 so that the axle is held 2 or 3 mm away from the axle beam as in General Assembly 1. Then tack or 'spot' weld the ball bearing bolt to the axle beam. Remove the bearings from their bolts and weld fully.

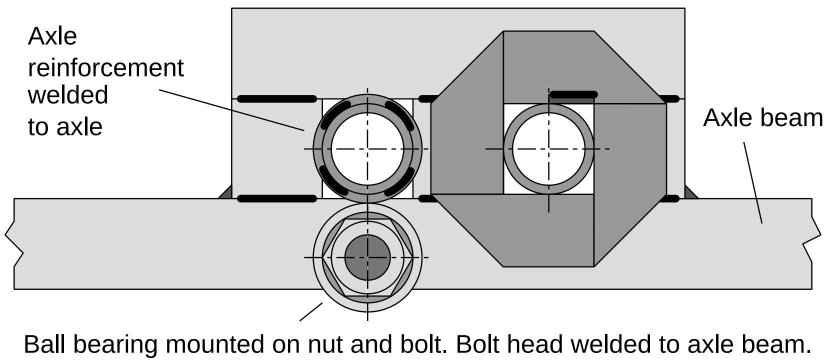


Figure 11: ball bearing mounting and axial bearing washer.

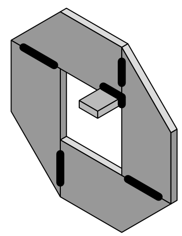


Figure 12: axial bearing washer.

15) Apply grease to the axles where they rub on the axle beams, replace the bearings on their bolts and tighten the nuts, fit the thrust washers and crossbolts and tighten.

16) You've finished it!

Other DTU cart developments

The DTU has been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. It has designs for twin axles with wooden bearings and twin axles with bearings made from PVC water pipe. And it has two systems of fixed axle: one with PVC bearings and another using needle roller bearings which you can make yourself. No machining is necessary for any of these axles.

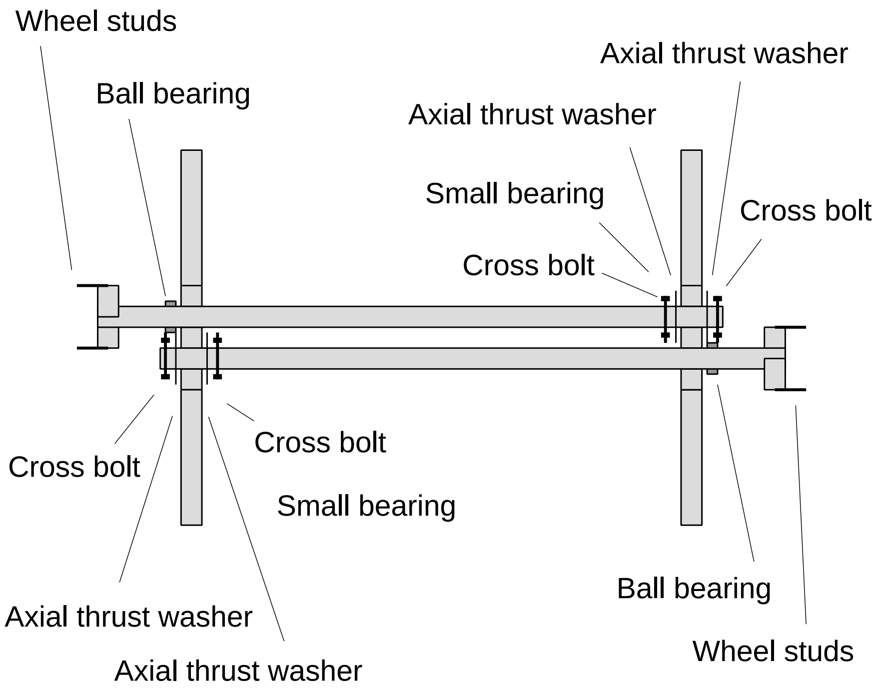


Figure 13: axle and bearing arrangement.

Other hub designs using, for example aluminium castings, have been in production in Nigeria and we are trying to reduce or eliminate the machining in these. Also wheel designs in steel sheet, cast aluminium and timber are under development. We have a design for solid steel rim wheels in which the rim is made from round bar and does not need any hammering.

The DTU has also been working on a range of cart body types for use with both donkeys and oxen. It has designs for wooden and steel framed types. The wooden types are cheaper in material terms, but the steel framed ones are easier to make because the joints are more straightforward - nevertheless you can make either type of cart in only one or two days, if you are reasonably set up with tools and materials.

Drawings

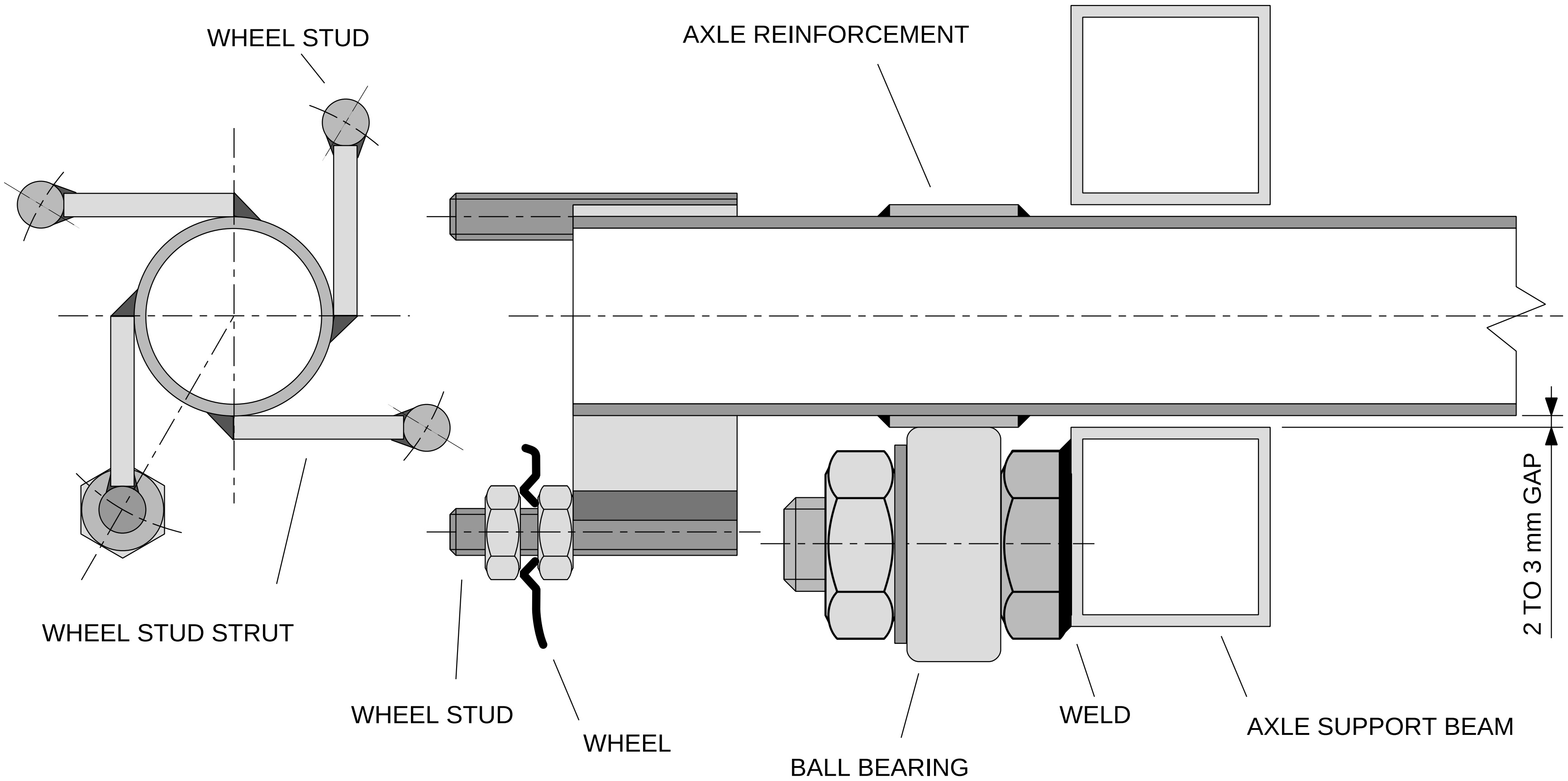
You will find four drawings on the next pages, the first two give a general section view of the axle. The third gives a view of the components of the axle itself and the fourth a drawing of the thrust washer.

Acknowledgements

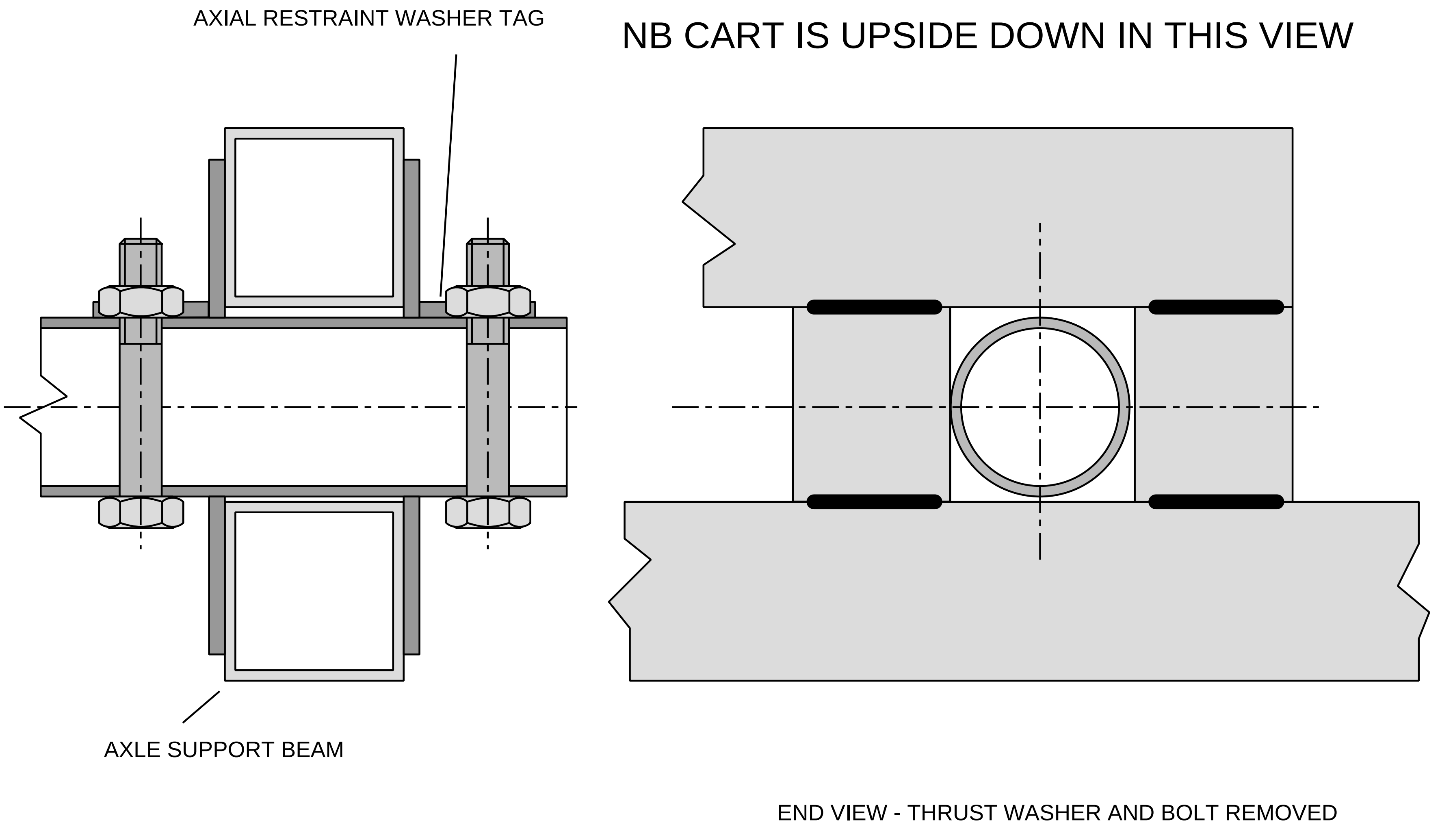
The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.

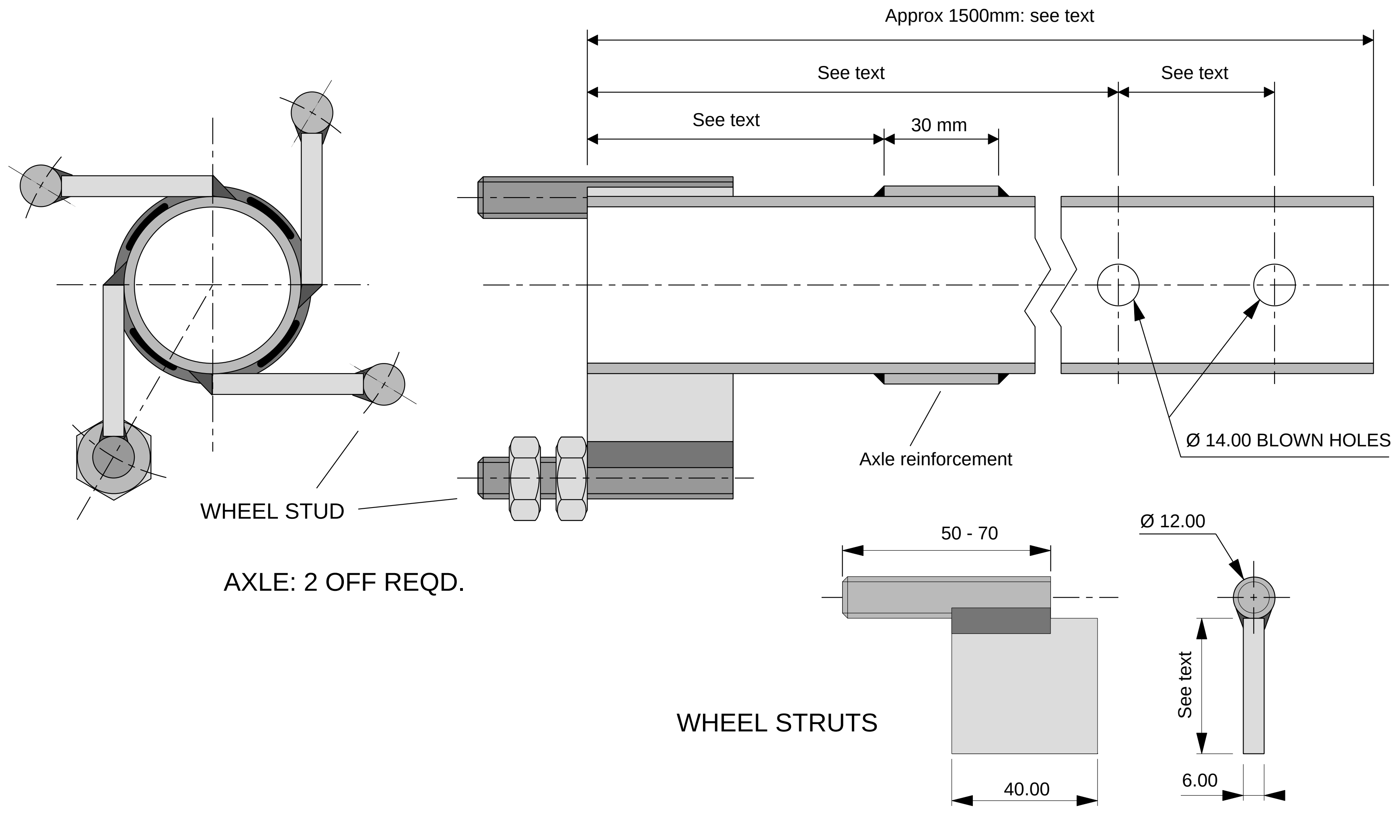
NB CART IS UPSIDE DOWN IN THIS VIEW



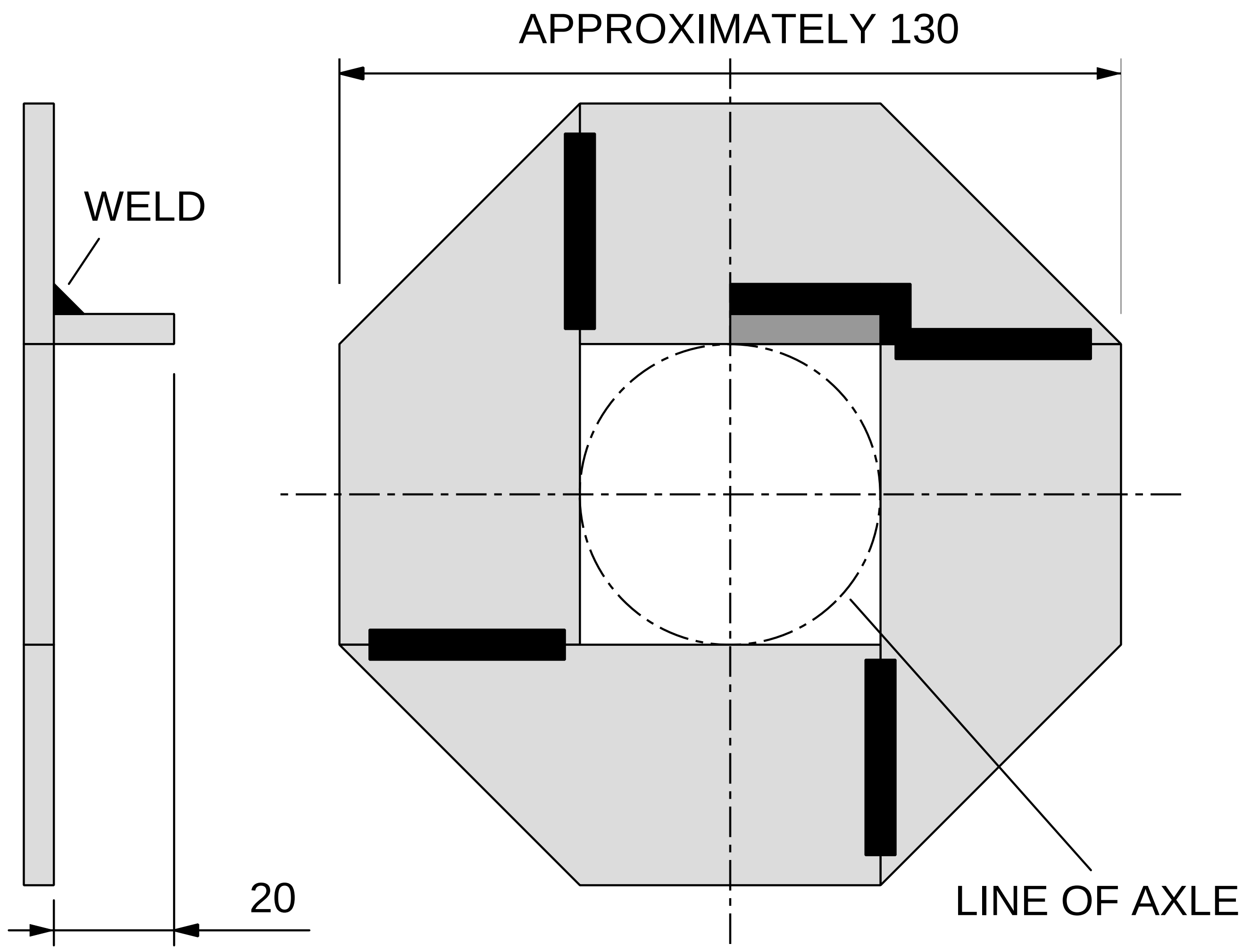
	GENERAL ASSEMBLY DRAWING 1	Scale	10mm	Title BALL BEARING TWIN DONKEY CART AXLE	Drawn by	CEO
		Date	1-4-99		Dwg No.	1/4



GENERAL ASSEMBLY DRAWING 2	Scale	10mm	Title BALL BEARING TWIN DONKEY CART AXLE	Drawn by	CEO
	Date	1-4-99		Dwg No.	2/4



	AXLE COMPONENTS	Scale	10mm	Title BALL BEARING TWIN DONKEY CART AXLE	Drawn by	CEO
		Date	1-4-99		Dwg No.	3/4



AXIAL THRUST WASHER: 4 REQD	Scale	10mm	Title BALL BEARING TWIN DONKEY CART AXLE	Drawn by	CEO
	Date	1-4-99		Dwg No.	4/4

KENDAT

DTU

Animal Cart Programme

TECHNICAL
42
RELEASE

Wooden Flexwheel for Donkey Carts

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: woo
wheel for anin
using old rubl
and timber.

Wooden Flexwheel for Donkey Carts.

Introduction

In this booklet we tell you how to make wooden wheels with scrap rubber tyre rims for donkey carts. The idea for these wheels comes from TDAU and Kasisi Mission in Zambia. This type of wheel cannot be punctured and it is quite easy to make. Unfortunately although the wheel itself is made without steel, the fixing to the axle does need welding.

The instructions here do not cover how to make the cart or the axle - you will need to read other Technical Releases from us to find out how to make these.

You should find that you can make a pair of wheels including the bare steel axle pipe for about £30. This cost will depend on the cost of the materials and labour. Once you get organised, two men can probably make a pair of wheels in two days.

Easy to make design.

This wheel is designed to be constructed without any special tools and jigs, and without any hard-to-get materials. The only tools which you must have are a simple welder, a woodsaw, a hacksaw, and a hammer.

Unfortunately we have only tested one of these wheels in Kenya and Uganda but we had no problems.

Cutting list and costs

Table 1 shows a cutting list for a wheel - recent prices of materials in Kenya are shown converted into £UK.

Construction step by step

1) The first job, is to get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete.

2) Start by cutting the tyre into two pieces around the centre of the tread. You will not be able to cut a tyre with a steel cord in the tread so avoid the steel type.

3) Measure the diameter of the tyre rim hole accurately. This is likely to be about 325 mm for a 13" tyre, 350 mm for a 14" tyre or 400 mm for a 16" tyre. Measure the diameter five or ten times in several different positions and take an average.

4) Measure the outside diameter of the tyre. This will be about 600 mm for the smaller tyres and 650 to 700 mm for the larger tyres.

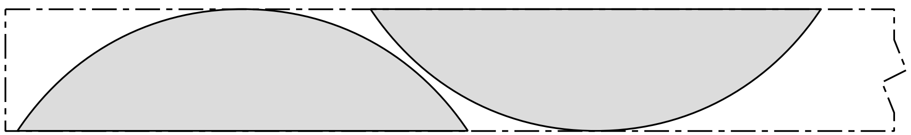


Figure 2: marking out segments to reduce waste.

5) Draw a circle of diameter 30 mm less than the outside diameter of the tyre on a piece of paper or cardboard using a piece of wire to make a compass. On the same centre draw a second circle of diameter equal to the hole diameter. This is to help you mark out the timber, so you should take some trouble over getting it accurate.

6) Now use the cardboard template to mark the timber planks. You can prick through the template into the wood and then join the marks with a pen. Figure 2 shows how to mark out the segments to waste the minimum timber.

7) Cut out the segments. For each wheel you will need to make two of the discs shown in Figure 3. Each disc is three planks thick as you can see. Make sure that the tyre is a snug fit on the single thickness small disc.

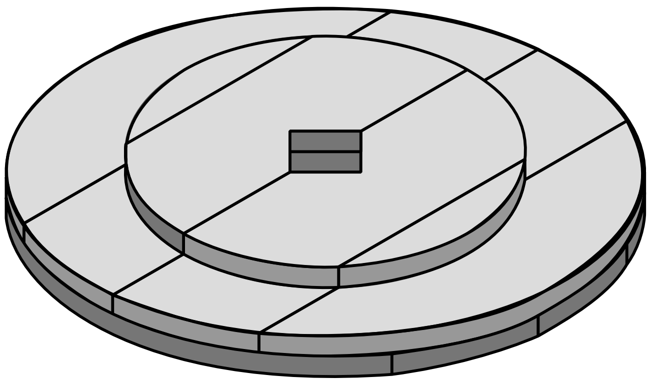


Figure 3: half of one wheel.

8) Nail the segments together with 60 mm nails clenched over. Make sure that the cutouts for the axle all line up - you can put a short piece of pipe into the notch to help align the segments.

9 Carefully mark the position of the bolt holes through the discs and drill the four holes in each disc. It is probably worth making these holes 15 or 16 mm diameter if you are using M12 bolts because it is hard to get all the holes to line up accurately.

10) Cut pieces of 40 × 6 mm steel bar 150 mm long to make the wheel stud struts shown in Figure 4. You will need four for each wheel. To make the long bolts cut the heads off 40 mm bolts and weld on pieces of 12 mm round bar 180 mm long. Check how thick your timber is and change the length here to suit if you need to.

TABLE 1: materials for wooden flexwheels.

component	material	# lengths reqd [#*mm]	total material for two wheels [mm]	cost [UK£]
wheel studs	50×M12 nuts and bolts	8	8	2.08
wheel stud struts	6 × 40 flat bar	8 ×150	1200.00	0.90
axles	1-½" BSP malleable iron pipe	2 × 1500	3000.00	8.23
small timber discs	150×25mm timber	8 × 360 + 8 × 200	4480.00	1.47
large timber discs	150×25mm timber	8 × 580 + 8 × 400	7840.00	2.57
scrap rubber car tyre	size 185×14	2 reqd	2 reqd	4.00
	TOTAL			19.26

11) Next cut the axles from 1-½" black pipe. These will probably need to be about 1600 mm long - it depends on the axle design you are using.

12) Position the axle in a pair of discs without the tyre and fit four stud/ struts. Arrange them to lie around the axle as in drawing 1/3 at the end of this document.

13) Using a trysquare get the axle square to the wheels and get someone to hold it there while you weld the struts to the axle. Make sure that you weld the struts to the axle strongly.

14) Remove the nuts from the studs and the outer wooden disc and fit the two halves of the tyre as shown in drawing 1/3. Replace the nuts and washers and tighten.

15) Repeat for the other wheel. You've finished it!

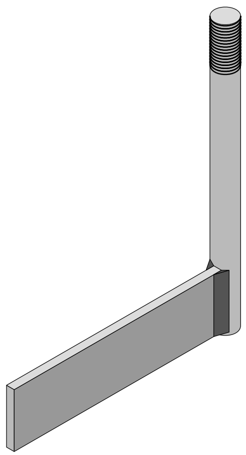


Figure 4: strut and stud fabrications.

Other DTU cart developments

The DTU has been working on new designs of carts and all their components to bring down their costs and make things more locally manufacturable. It has designs for bodies, wheels, hubs, bearings and animal harness all available from DTU as Technical Releases.

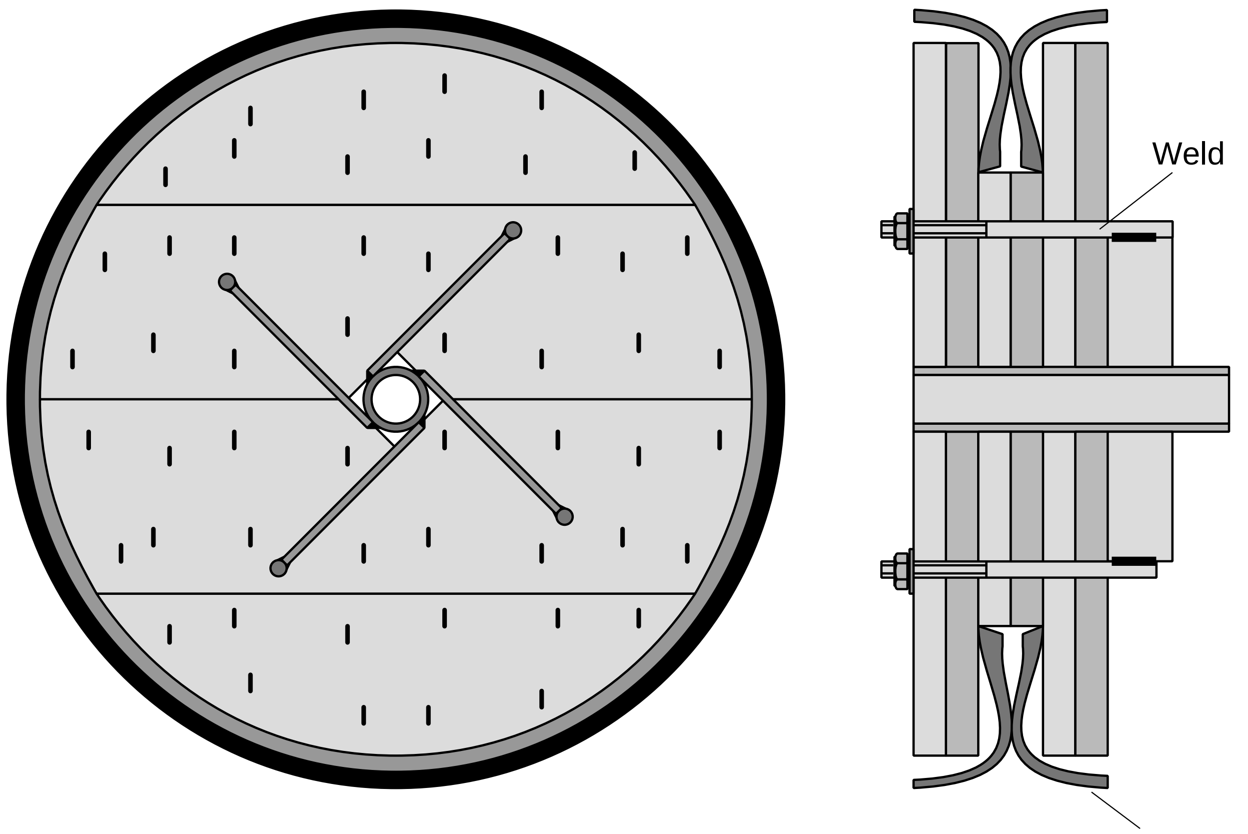
Drawing

You will find a drawing of the wheel on the next page.

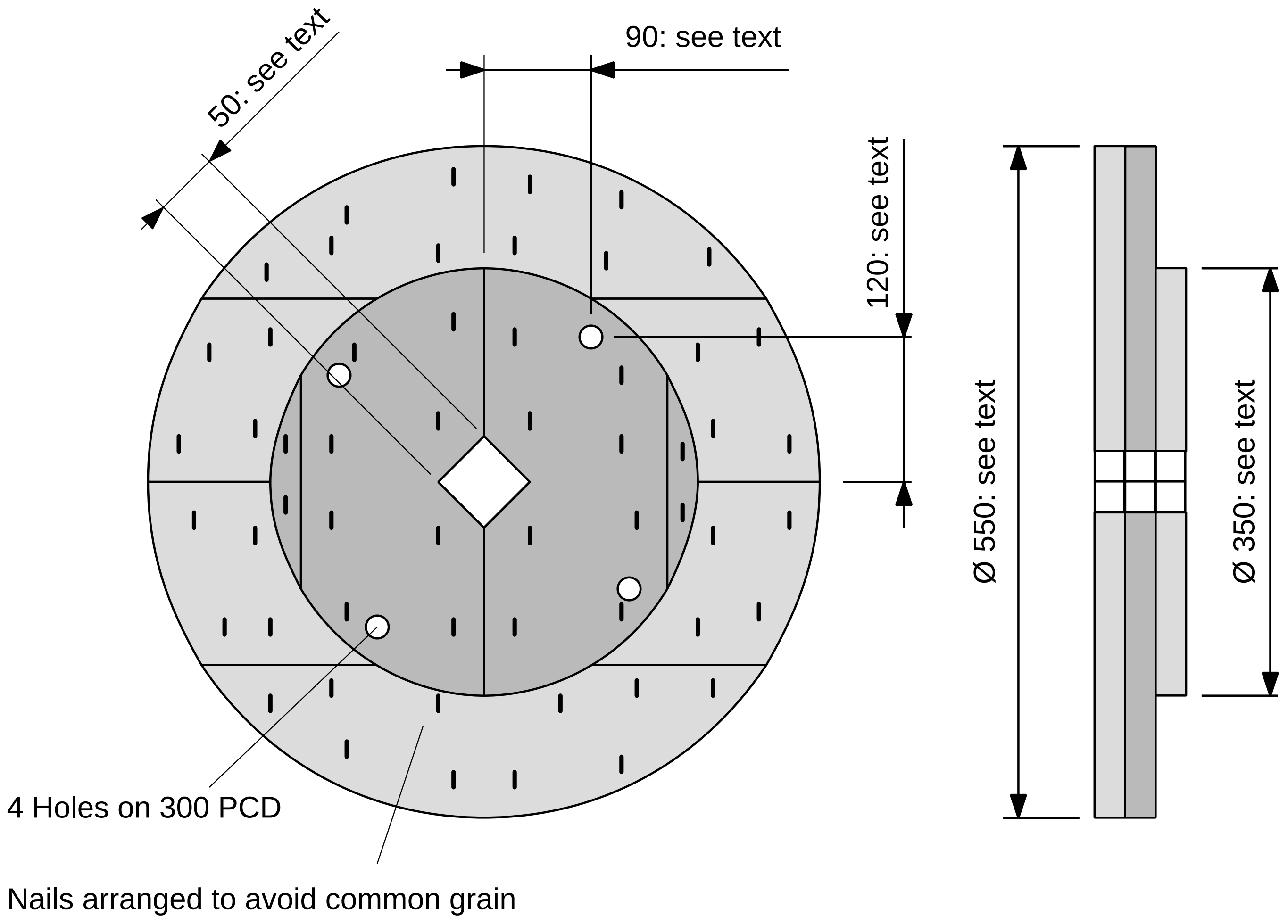
Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

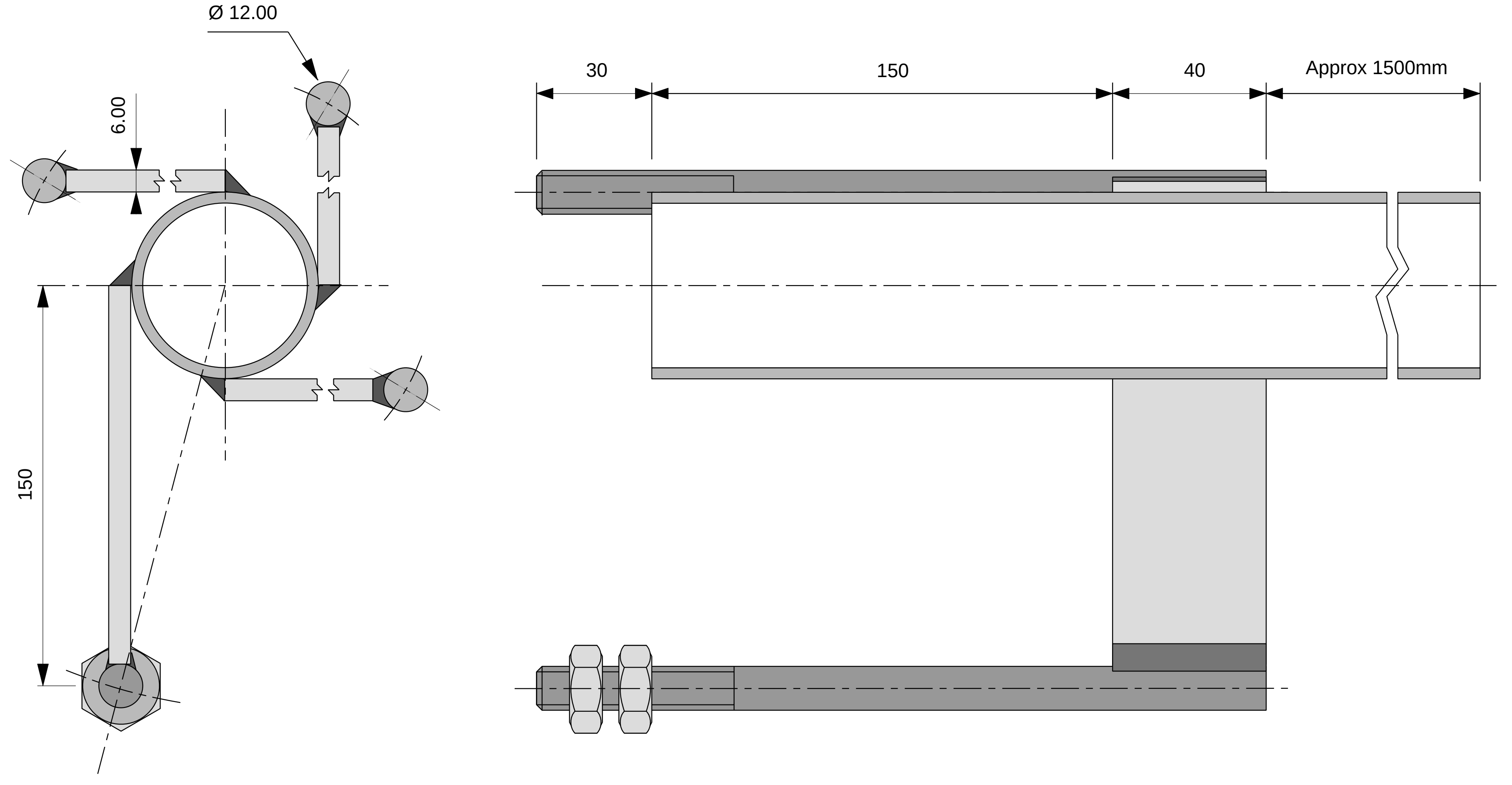
The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



	General Arrangement	Scale	10mm	Title WOOD FLEXWHEEL FOR DONKEY CARTS	Drawn by	CEO
		Date	13-4-99		Dwg No.	1/3



Wood components	Scale	10mm	Title WOOD FLEXWHEEL FOR DONKEY CARTS	Drawn by	CEO
	Date	13-4-99		Dwg No.	1/3



AXLE COMPONENTS	Scale	10mm	Title WOOD FLEXWHEEL FOR DONKEY CARTS	Drawn by	CEO
	Date	15-4-99		Dwg No.	3/3

KENDAT

DTU

Animal Cart Programme

TECHNICAL

43

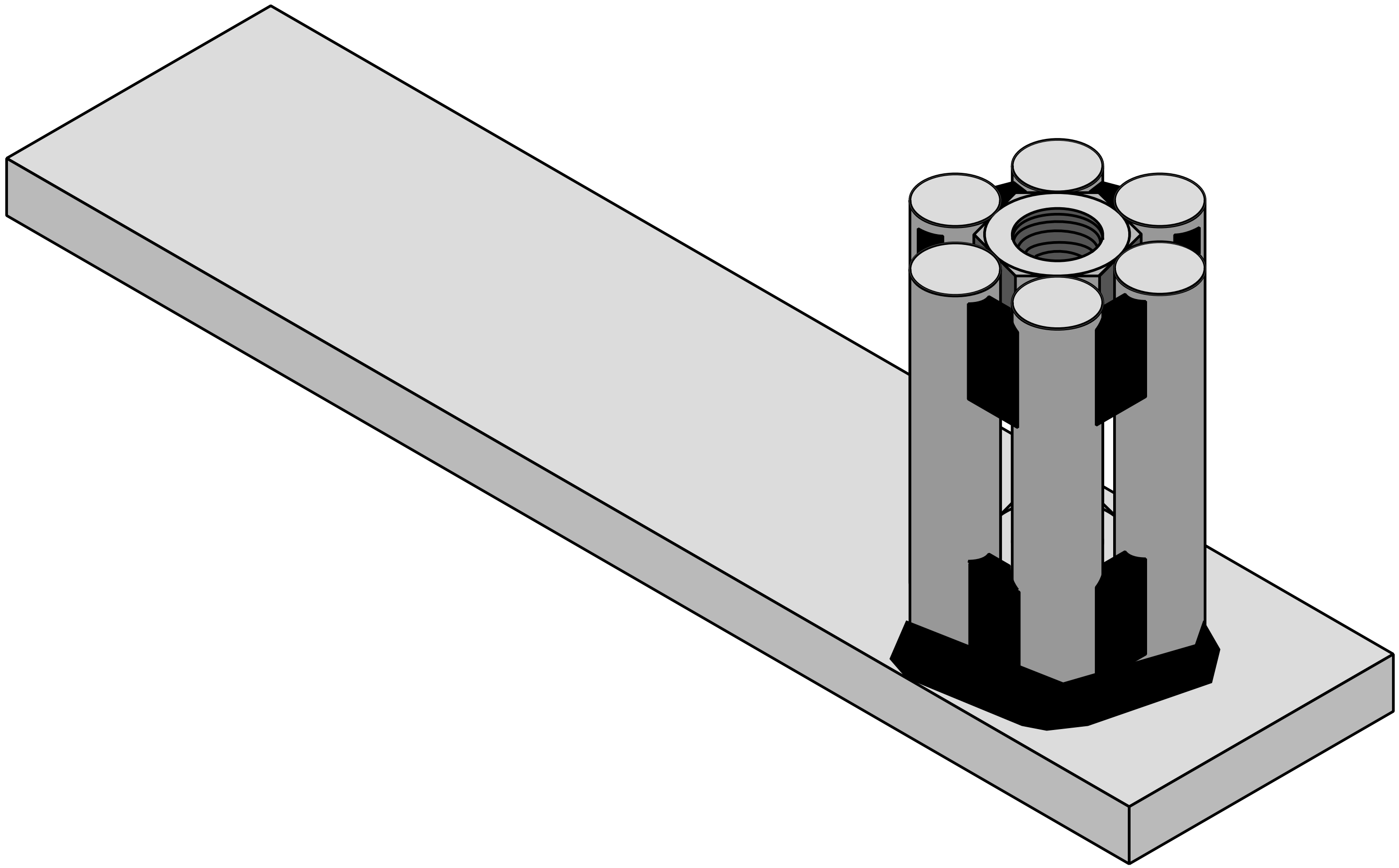
RELEASE

Wheel Spanner for Animal Carts

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk

KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: spanner for wheel nuts made from round and flat steel bar and welding.



Wheel Nut Spanner for Animal Carts.

Introduction

In this booklet we tell you how to make a spanner for M12 wheel nuts from round bar and a bit of welding.

You should find that you can make a spanner for less than £1 and you can do it in only an hour or less.

The only tools which you must have are a simple welder, a hacksaw, and a hammer.

Cutting list and costs

Table 1 shows a cutting list for a wheel - recent prices of materials in Kenya are shown converted into £UK.

Construction step by step

1) Cut the round bar into six 50 mm lengths. File the ends square if you are not good a sawing accurately.
2) Cut some strips of thin steel from a tin or drinks can. The strips should be about 65 mm long and 10 mm wide. You will need at least two of them.
3) Wrap one of the strips of thin steel around the six sides of an M12 nut. Bend the steel so that it stays in place without being held.
4) Put the nut and steel strip on a flat surface and arrange the round bars around it as shown in Figure 2.
5) Put a pipe clip around the six rods outside the nut and tighten the clip so that it grips the nut and steel strip tightly inside the rods.you cannot easily get a pipe clip use a piece of wire to bind the rods together.
6) Bend a second steel strip around a nut and put it and the nut inside the rods at the other end. Put another pipe clip around them. Check that all is square and straight.
7) Carefully weld the six rods together using small (tack/ spot) welds. When you have it all joined together you can remove the nuts and steel strip and weld heavily to join it all together strongly. You do not need to weld all the way along the rods, a good weld at each end and on in the middle is all that is needed.

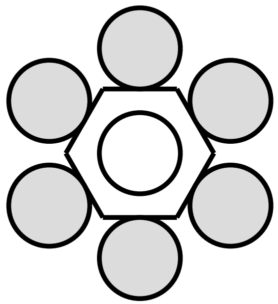


Figure 2: arranging round bars around nut.

8) Make a handle from a piece of flat bar or square pipe and weld it onto one end of the spanner to make a handle.

9) You've finished it!

DTU cart developments

The DTU has been working on new designs of carts and all their components to bring down their costs and make things more locally manufacturable. It has designs for bodies, wheels, hubs, bearings and animal harness all available from DTU as Technical Releases.

Drawing

You will find a drawing of the wheel on the next page.

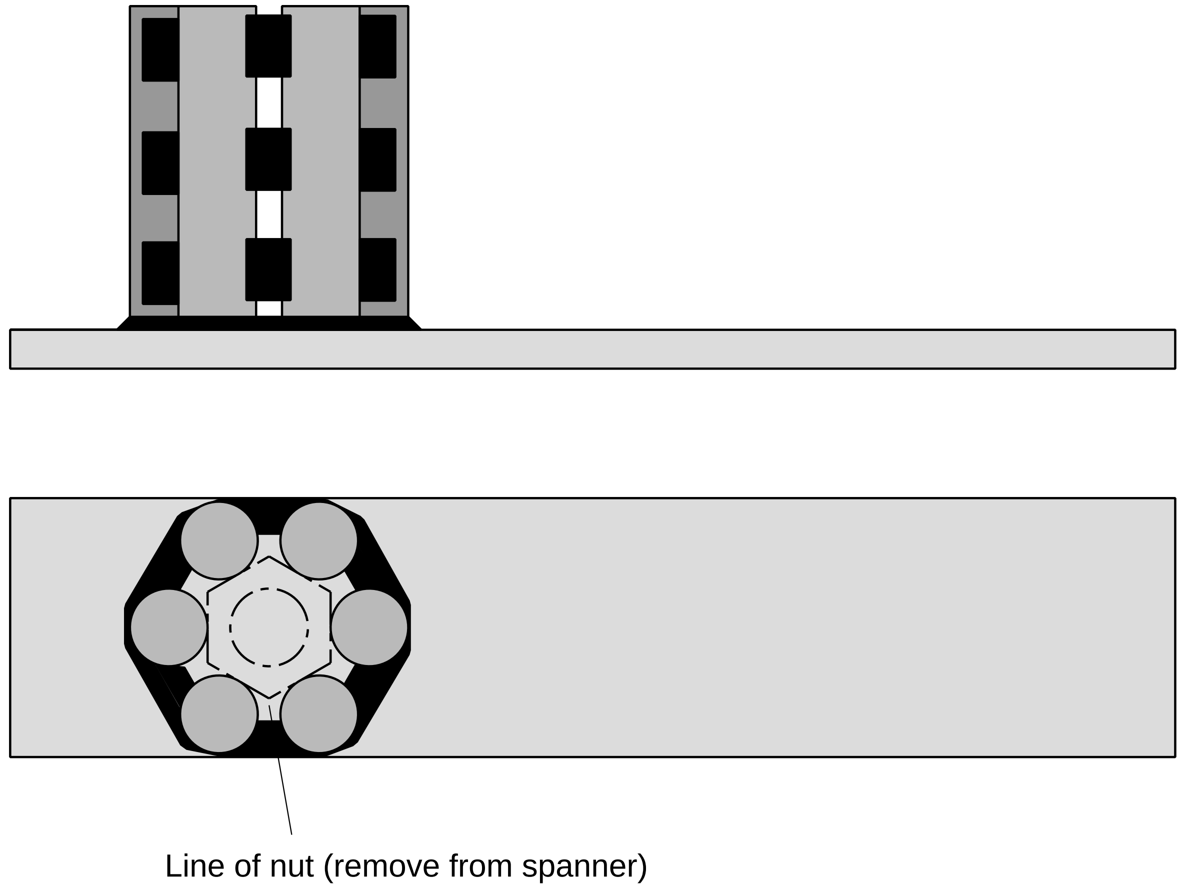
TABLE 1: materials for wheel spanner.

component	material	# lengths reqd [#*mm]	total material for two wheels [mm]	cost [UK£]
wheel studs	50×M12 nuts and bolts	8	8	2.08
wheel stud struts	6 × 40 flat bar	8 ×150	1200.00	0.90
axles	1-½" BSP malleable iron pipe	2 × 1500	3000.00	8.23
small timber discs	150×25mm timber	8 × 360 + 8 × 200	4480.00	1.47
large timber discs	150×25mm timber	8 × 580 + 8 × 400	7840.00	2.57
scrap rubber car tyre	size 185×14	2 reqd	2 reqd	4.00
	TOTAL			19.26

Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



	General Arrangement	Scale	20mm	Title	SPANNER FOR WHEEL NUTS	Drawn by	CEO
		Date	16-4-99			Dwg No.	1/1

KENDAT

DTU

Animal Cart Programme

TECHNICAL
44
RELEASE

As cast aluminium wheel with integral roller bearing

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk

KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: unmachined cast aluminium wheel for animal carts using simple pipe rollers as a low cost roller bearing.

This photograph shows one half only of the wheel. Also missing is the axle - just a piece of 2" pipe (outside diameter 60mm) and the thrust washers.

Note the polish on the rollers and the scuff marks in the aluminium casting where the ends of the rollers rub.

As Cast Aluminium Wheel for Animal Carts.

Introduction

This booklet describes work carried out at Warwick University on as cast aluminium wheels with integral roller bearings for animal carts. This work grew out of work we had performed in Nigeria.

We have made several wheels to the design in the drawings in this document and tested them in a machine which makes them revolve under load. Figure 1 shows one half of one wheel. We ran them for many days and then took them apart to see how much they had worn. We were surprised that they had worn only a little.

We think that they could be used in some countries where scrap aluminium is cheap and where there is good enough communication to allow manufacture in a few workshps around a country and a trade in wheels to develop. Unfortunately we thought that these conditions did not exist in Kenya and Uganda.

We expect that you could make a pair of these wheels including the bare steel axle pipe for about £30. This cost will depend on the cost of the materials and labour.

Easy to cast design.

We have designed this wheel to be easy to cast. You need only to feed the molten aluminium to three or four places around the rim and if you make the gates quite big (about 30 mm by 20 mm) you may be able to avoid using feeders. All the components of the wheels and axle can be made without machine tools though you may need to drill the holes for the bolts joining the two halves of the wheel together if you cannot cast them.

Construction

We have not developed these wheels and so we do not have proper instructions for their casting. We cast our wheels in a special casting laboratory at our university. But we did do some quite complicated casting in Nigeria and we are sure that it would be possible to do this work in most other developing countries. Certainly we identified several companies who could have cast the wheels in Kenya.

Unfortunately therefore you will have to get local expert casting workers to make a suitable wooden pattern from the drawings and then make the castings. After that you can make up the rollers and the rest of the axle yourself.

1) The first job, is to get all the material together including the castings and clear a space to work. Ideally you will be able to work on a flat area of concrete.

2) Clean up the castings and cut off all the excess lumps from the casting process. Remember that the inner tube will press on the casting so you don't want sharp points left to make punctures.

3) Line up two wheel halves and drill six or eight bolt holes through as shown in the drawings. It is probably worth making these holes 13 or 14 mm diameter if you are using M12 bolts because it is hard to get all the holes to line up accurately.

4) Cut lengths of 1/2" steel pipe to act as rollers. You should file the ends of the rollers smooth and square to the length so they revolve smoothly.

5) Make the big washers from 5 mm or 5 mm steel plate. You do not need to make them very accurately but they must go over the axle and they must not have any bumps on the sides. So you could make the hole in the middle with a welder as long as you file the bumps off afterwards.

6) Cut the axle to length and assemble the wheels and washers on it in the right place. You do not need to put the rollers in at this stage. Mark the position of the cross bolts in the axle and blow the holes with an electric welder or drill them.

   A useful way of cutting pipe square at the ends is to wrap a piece of paper or cardboard around the pipe and push it until the edge of the paper is level all the way round and the paper is tight on the pipe. Then mark around with felt tip pen. Finally cut to the line.

7) Next remove the components, clean everything and put it all together with some oil or grease.

8) You've finished it!

TABLE 1: materials for cast aluminium wheels.

component	material	# lengths reqd [#*mm]	total material for two wheels [mm]	cost [UK£]
aluminium castings	scrap aluminium	28 kg	28.00	14.00
wheel studs	75×M12 nuts and bolts	4	4	1.04
axle thrust washers	6 × 40 flat bar	16 ×140	2240.00	1.68
axles	2"" BSP malleable iron pipe	1 × 1500	1500.00	4.87
rollers	1/2"" BSP malleable iron pipe	2×11×225	4950.00	6.19
			TOTAL	**27.78**

Other DTU cart developments

The DTU has been working on new designs of carts and all their components to bring down their costs and make things more locally manufacturable. It has designs for bodies, wheels, hubs, bearings and animal harness all available from DTU as Technical Releases.

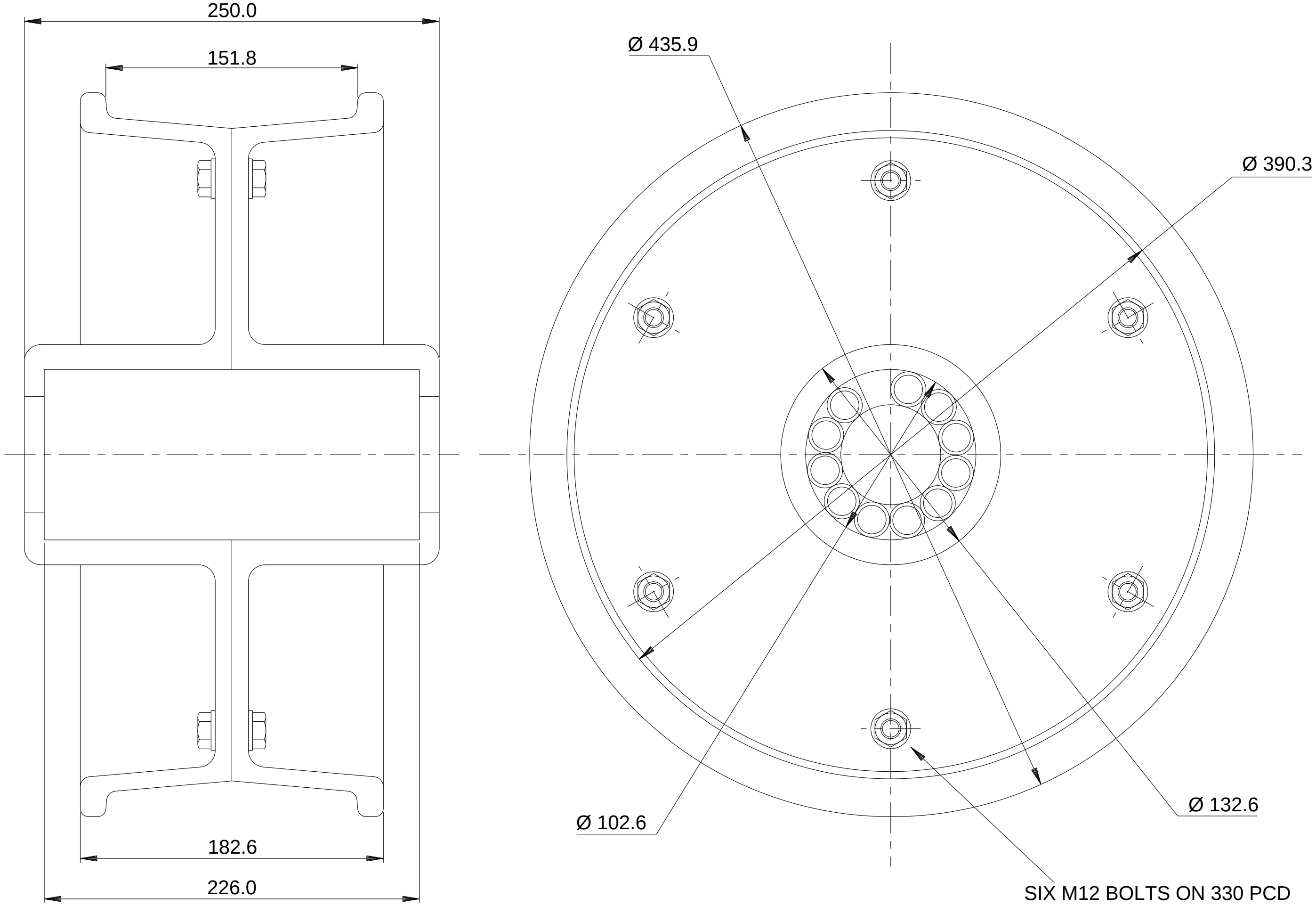
Drawing

You will find drawings of the wheel and axle on the next pages.

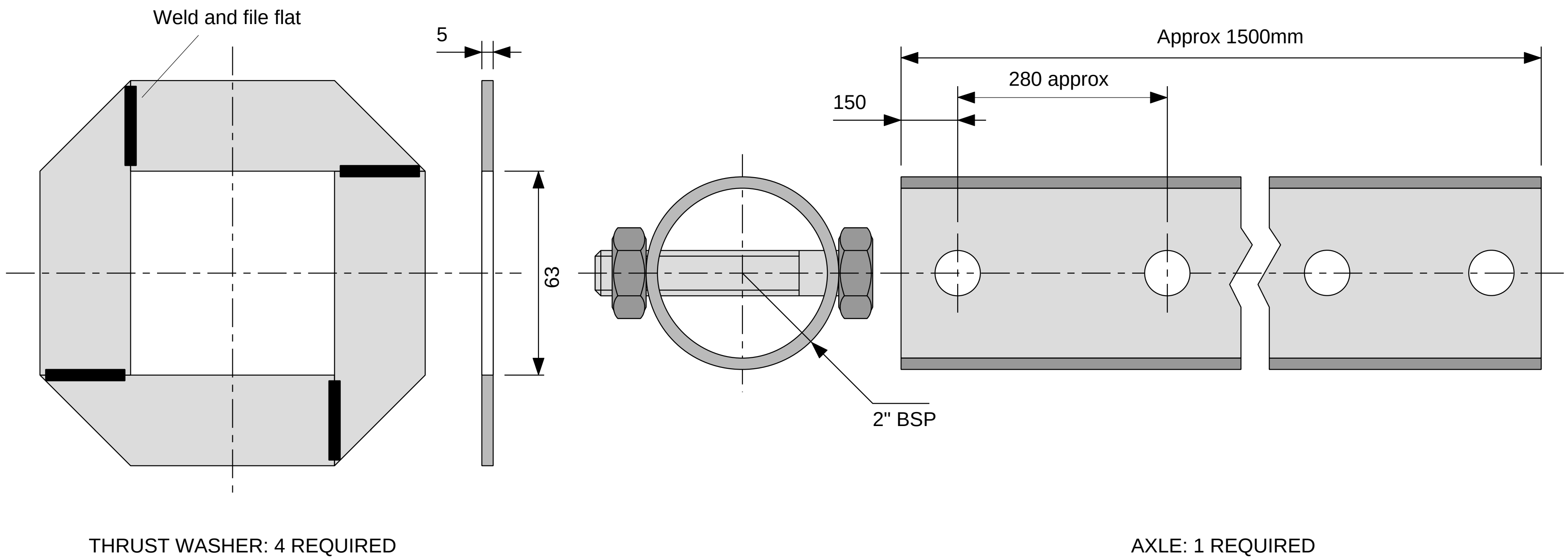
Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



WHEEL ASSEMBLY	Scale		Title AS CAST ALUMINIUM WHEEL + BEARING FOR ANIMAL CARTS	Drawn by	CEO
	Date	15-4-99		Dwg No.	1/2



	AXLE COMPONENTS	Scale		Title AS CAST ALUMINIUM WHEEL + BEARING FOR ANIMAL CARTS	Drawn by	CEO
		Date	15-4-99		Dwg No.	2/2

KENDAT

DTU

Animal Cart Programme

TECHNICAL

46

RELEASE

Straight Hame Collar for Donkey Harness

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)1203 523523 extn 2339, fax: +44 (0)1203 418922, email: dtu@eng.eng.warwick.ac.uk

KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: straight hame collar for donkeys - wooden version.

Straight Hame Collar Harness for Donkeys made from Steel Tubing, or Timber

Introduction

This technical release covers the manufacture of a straight hame collar for donkeys. Collars allow the highest draught (pull) from a particular animal. Collars can be used to pull carts or agricultural implements. This chapter shows both a steel and a wooden design.

You should find that you can make the wooden collar for one donkey for less than £$_{UK}$1 and the steel one for about £$_{UK}$3, depending on the cost of the materials and labour. Once you get organised, a man can probably make a colar in a few hours.

Idea Behind Design

Collars are used in many countries to harness animals for draught (pull). Most of the designs we have seen have curved hames (the wood or metal parts pressing on the sides of the animal's necks), but some countries have used straight pieces of wood here and our experience suggests that this will work adequately if not well - depending on the shape of the donkey. Most of the donnkeys we have seen in Kenya, Uganda and South Africa would be fine with straight hames.

Special tools and jigs and hard-to-get materials are not required for these designs - the only tools which you must have are a woodsaw and hammer - and for the steel design, a simple welder and hacksaw.

Cutting lists and costs

Tables 1 and 2 show the cutting lists for complete collars -

Figure 2: steel version of straight hame donkey collar.

recent prices of materials in Kenya are shown converted into $£_{UK}$ to assist international comparison.

Construction step by step (steel version)

1) Get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete or have access to a bench and vice.

2) Cut two pieces of 50 mm diameter steel pipe 450 mm long.

3) Using a hacksaw, angle grinder or welder set to maximum current, cut the slots for the hame cross bar as shown in Figure 3.

4) Make a top ring for each hame by winding some 8 mm round bar around a piece of the tube and weld it to the top of each hame to close off the end of the slots.

TABLE 1: steel straight hame collar materials cutting list.

component	material	# components	total mat [mm]	mat cost [$£_{UK}$]
main frames	50 round tubing	2*450	900.00	0.97
padding clips	8 mm re bar	2*2*40	160.00	0.02
U loops	8 mm re bar	2*3*125	750.00	0.07
top link	8 mm re bar	2*250	500.00	0.05
strap hooks	8 mm re bar	2*150	300.00	0.03
top link pins	scrap tyre rubber	1*8*70	10.00	0.10
			TOTAL =	**1.25**

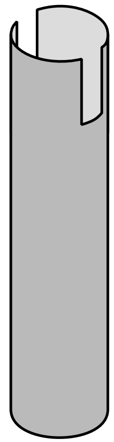


Figure 3: slot cut in top of steel hame.

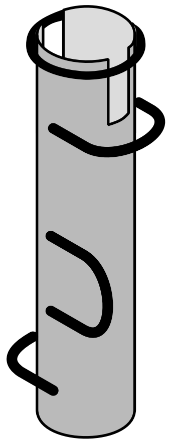


Figure 4: attachment loops and top ring welded to steel hame.

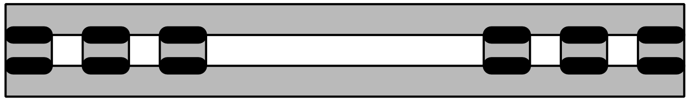


Figure 5: welding of hame cross bar.

5) Make up the hame cross bar shown in Figure 5.

6) Cut some scrap tyre rubber to make pegs to stop the hame cross bar pulling through the hames.

7) Cut 125 mm lengths of 8 mm diameter round steel bar and bend them into U shaped loops.

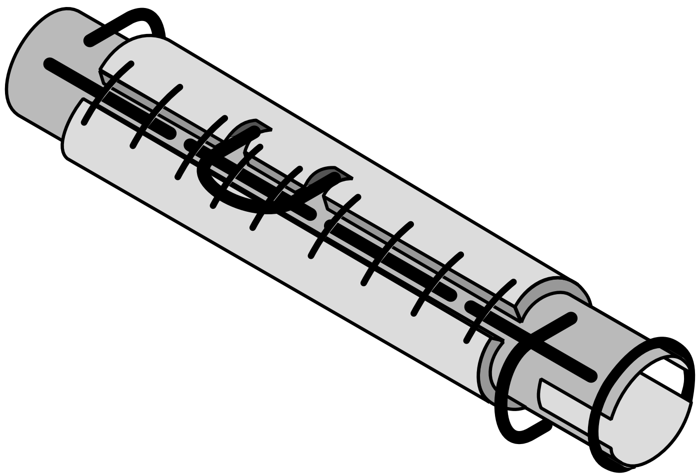


Figure 6: padding tied to hame with wire or cord.

8) Weld the U loops onto the tubes in the places indicated in the drawings.

9) Either:

wind the padding rope around the tube and fix the ends,

or fix padding cloth, blanket or carpet to the hames using wire or cord as shown in Figure 6.

10) Make up some attachment hooks as shown in Figure 8.

Construction step by step (wood version)

1) As always get the material together and clear a space to work. Ideally you will have a flat area of concrete or have access to a bench and vice.

2) Cut two 450 mm long pieces of timber about 50 mm in

TABLE 2: wood straight hame collar materials cutting list.

component	material	# components	total mat [mm]	mat cost [£UK]
hames	50 round timber	2*450	900.00	0.18
padding clips	6mm re bar	2*2*40	160.00	0.02
U loops	6mm re bar	2*3*200	1200.00	0.12
top link	6mm re bar	2*250	500.00	0.05
strap hooks	6mm re bar	2*150	300.00	0.03
top link pins	scrap tyre rubber	1*8*70	10.00	0.10
			TOTAL =	**0.49**

diameter.

3) Make a U-shaped loop from a piece of 8 mm round steel bar for each hame pull loop.

4) In each piece of wood drill an 8mm diameter hole 175 mm from one end and another 225 mm from the same end, making sure that both holes are in line with each other.

5) Hammer the pull loop into each hame and clench over the ends of the loops to hold them into the hame. The clenched ends must be sunk into the timber so they do not press through the padding into the animal.

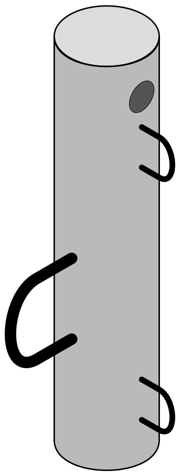


Figure 7: wood hame with hole drilled for bolt type crossbar.

6) Hammer a U-nail into the side of each end of each hame in line with the pull loop as shown in the drawings.

7) Either:

cut a slot in the top ends of the hames for the crossbar and make up the crossbar and two rubber pegs to put through the holes in the crossbar.

or:

drill a hole in each hame 40 mm from the top for a crossbar made from a long bolt (Figure 3·11 shows how a long bolt can be made from a short one) and make the long bolt. Figure 7 shows a wooden hame with a drilled hole before padding has been fixed.

or:

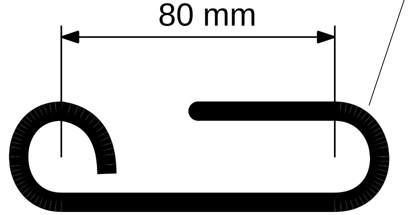


Figure 8: chain hooks for straps.

make up leather straps and buckles as in Figure 1. The leather straps can be put through the U-nails to hold them in place.

8) Cut a piece padding (carpet, blanket cloth etc) and nail to the hame as shown in the drawings.

Collar adjustment and method of use

(Remember that protecting the donkey will save money because it can work harder if it is comfortable and will not get sick from skin wounds.)

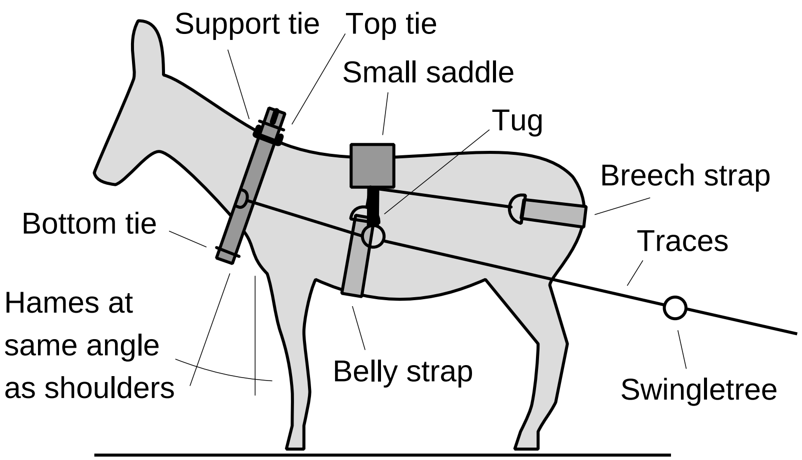


Figure 9: Collar position on donkey with traces pulling at right angles to hames.

1) Adjust the top fixing between the two hames so that it is in approximately the right position (once the collar has been used on a particular donkey this will probably not need further adjustment unless the condition of the donkey changes ie it get fatter or thinner).

2) Slacken or undo the fixing between the bottoms of the two hames.

3) Place the collar over the neck of the donkey and adjust the bottom fixing rope across between the bottoms of the two hames so that it is in approximately the correct position. (If you are using rope for this tie do no tie it properly yet.)

4) Now check and adjust both top and bottom fixings between

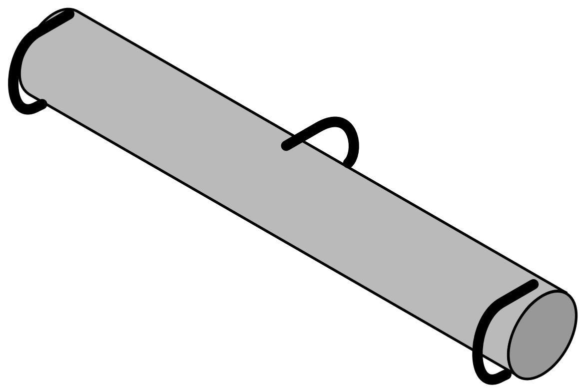


Figure 10: swingletree: the traces will be tied to the ends, and the implement tied to the central loop.

the two hames until the collar is in the right vertical position and the angle between the hames fits the donkey's neck. (If you are using rope for the ties, now make the knot properly.)

5) Tie the top support rope or strap so that it carries the some of the weight of the collar on the donkey's neck.

6) Adjust the lengths of the tugs (the ropes or straps hanging from the saddle if you use one) so that the traces (the ropes or chains which will pull the cart or implement) come back at right angles to the animal's shoulder blades (see Figure 9).

Saddles

With a suitable agricultural implement (plough or weeder etc) It is sometimes possible to pull without using a saddle, but it is usually easier to use a one. This is because ideal angle for the traces is rarely the same as for the implement and using a saddle these can be adjusted independently.

Swingletrees

Whether a saddle is used or not, a swingletree will be needed. A swingletree can be made using the same types of materials as the steel hames. The tube will need to be about 700 mm or 800 mm long depending on how fat the donkey is. You should make sure that the swingletree is long enough so that the traces do not normally rub on the sides of the donkey when they are tight and the donkey is working. Figure 10 shows a steel swingletree. It is also quite easy to make a wooden one.

If you are going to pull a cart you must use a saddle as shown in Figure 9 because the cart will need some up or down force to stabilise it and a donkey's back needs the protection that a saddle gives.

When pulling a cart fix the breeching strap and the tugs to the shafts. Lead the traces to a swingletree on the front of the cart.

Other DTU cart and harness developments

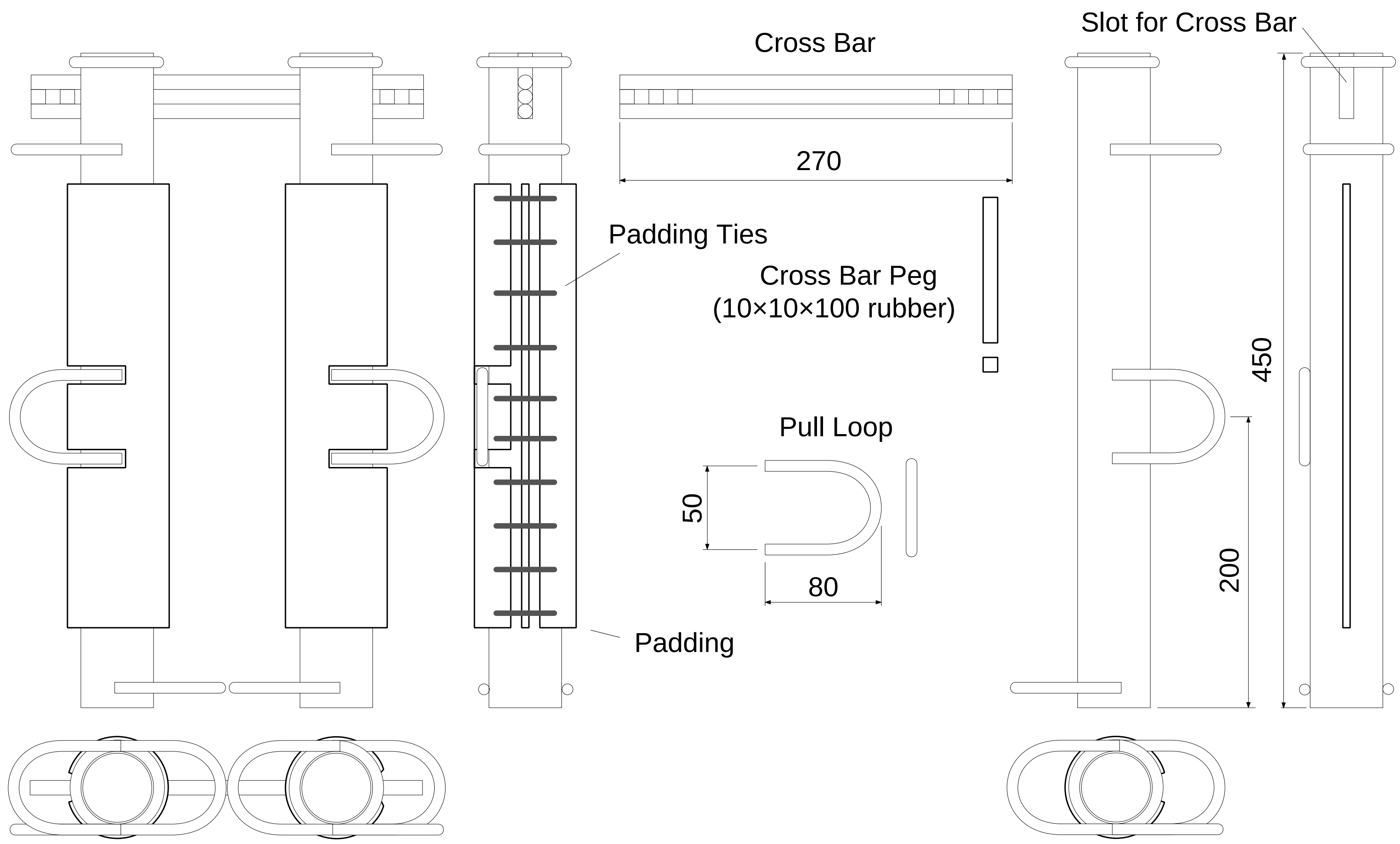
The DTU has been working on a range of cart designs for use with both donkeys and oxen. It has designs for wooden and steel framed types. You can make either type of cart in only a few hours, if you are reasonably set up with tools and materials.

The DTU has also been working on new designs of wheels, hubs and bearings to bring down their costs and make things more locally manufacturable. It has a system of axles with bearings made from PVC pipe, another with wooden bearings and a third using scrap ball bearings. None of these axles need machining and they only take two men a day to make.

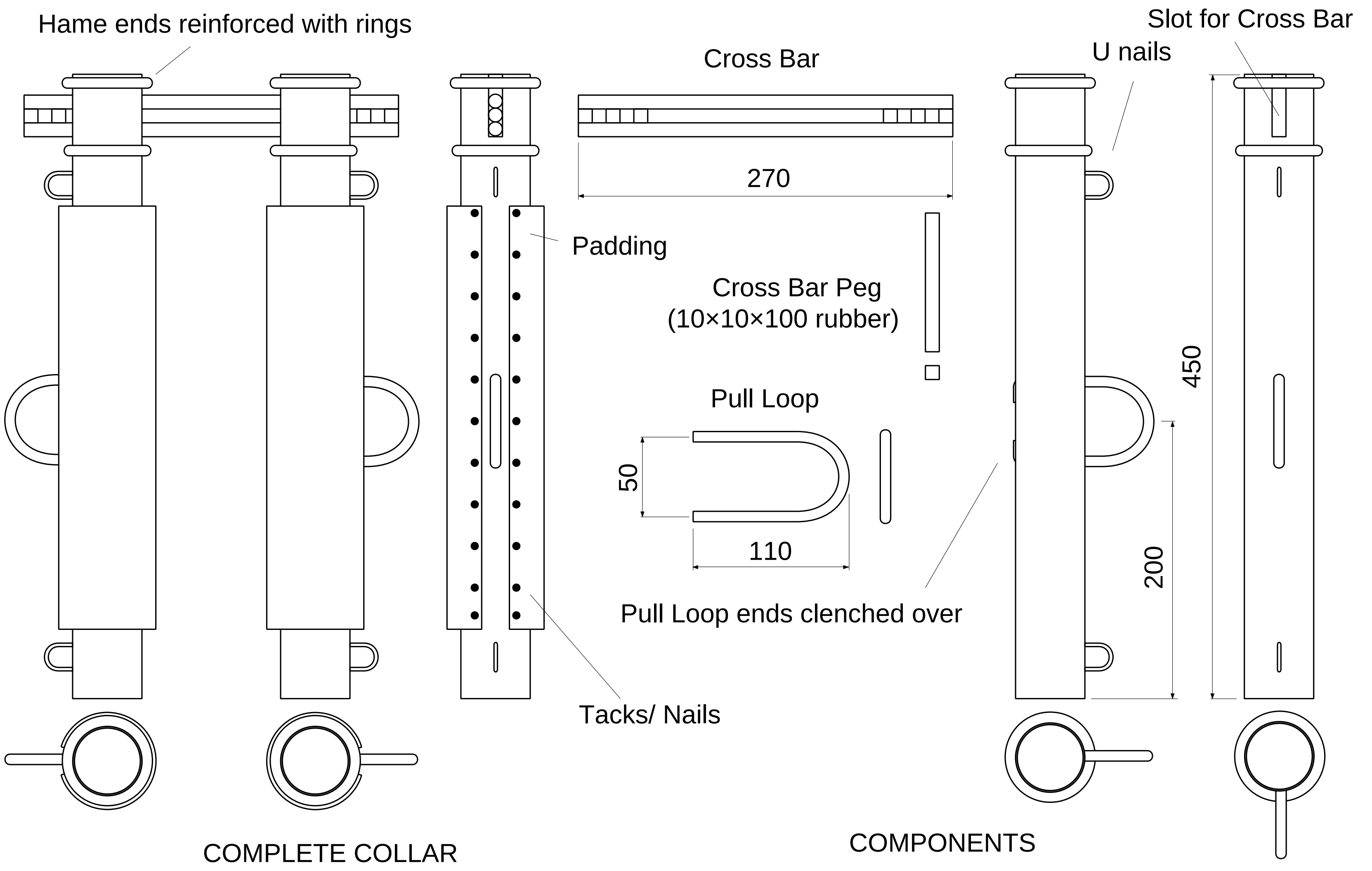
Acknowledgements

Initial trials with these collars were suggested and greatly assisted by Dr Dag Austbo and Mr Martin Aeshlimann from the National Agricultural University of Norway and were hosted by Dr Bruce Joubert in South Africa.

The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajaido, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



Scale	25 mm	Title	Straight Hame Donkey Collar of Steel	Drawn by	CEO
Date	25/12/99			Dwg No	1/1



					Scale	25 mm	Title	Straight hame Donkey collar of Wood	Drawn by	CEO
					Date	25/12/99			Dwg No	1/1

KENDAT

DTU

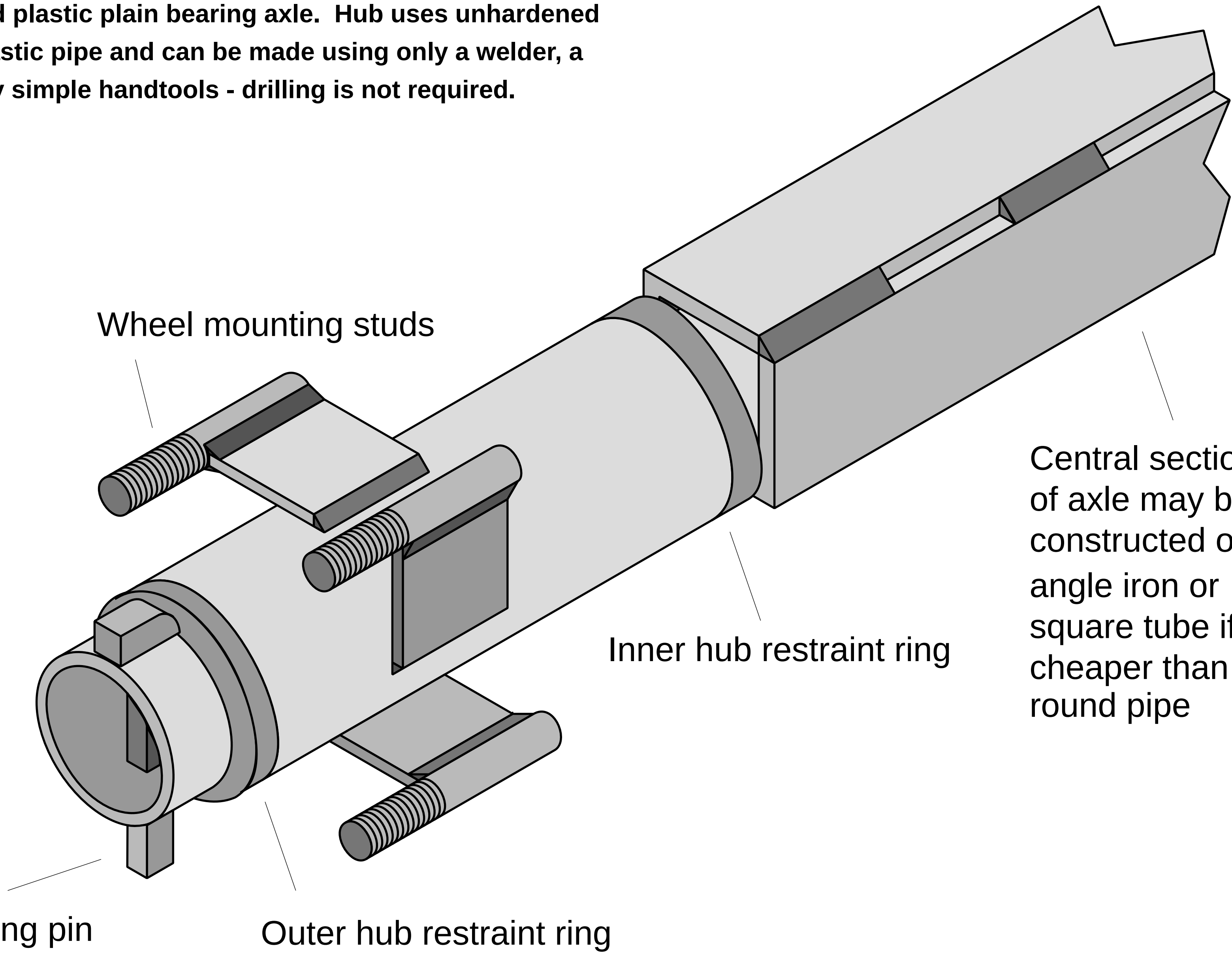
Animal Cart Programme

TECHNICAL
47
RELEASE

PVC PLAIN BEARING OX CART AXLE

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)203 523523 extn 2339, fax: +44 (0)203 418922, email: esceo@eng.warwick.ac.uk
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: pipe and plastic plain bearing axle. Hub uses unhardened mild steel and plastic pipe and can be made using only a welder, a hacksaw and very simple handtools - drilling is not required.



PVC Plain Bearing Ox-Cart Axle

Introduction

Not enough farmers in Africa have animal carts. Those who have carts can take their produce to places where they can get the best prices. They can also get into town and buy fertilizer and better seeds and move things around their farm easier. The trouble is that carts are too expensive for many farmers. The question is what can be done about it?

Carts are made in many different places. Some carts are made in factories in industrial countries and some are made in factories in Africa, but most are made by local blacksmiths or carpenters using scrap car and 4WD axles. In many countries these people cannot get enough axles to meet the demand so the price is high. Another problem is that the axles are often so worn that they do not last long. Lots of farmers take the (differential) unit out of the axle too, which makes the axle break sooner and lets the dirt in.

What you need is an axle which blacksmiths and fabricators can make with fairly simple tools - without having to get parts machined. There are usually blacksmiths and fabricators in the small market towns used by the farmers. Experts think that having the cart maker close to the farmer is a good thing because they can talk to each other easily and sort out any problems. And of course if the cart is made locally, it can be repaired locally, so there will not be problems with spare parts.

Idea behind design

The idea behind the design of axle described in this technical release is to allow construction without the use of machine tools (drills, lathes and milling machines), and to use materials which should be readily available. The materials can be used 'as bought' - no hardening of any of the components is needed. The only tools which you must have are a welder and a hacksaw, but a file and a vice are also very handy. Of course if you do have power tools - especially a power hacksaw or angle grinder with cutoff wheel - things can be made much faster. This axle is suitable for a wide range of production methods so that if you have to make many of them you can make special tools to make it quicker.

The hubs in this design use 2½" BSP pipe on 2" BSP axles so that they will fit many scrap 4WD wheels. Most wheels have a hole in the middle about 75 mm diameter, or a bit bigger. Sometimes the holes are smaller and the wheel will not fit on the hub, but you can sometimes saw or file the hole bigger if you have to. Another way of avoiding the hole in the wheel problem is to use live axles as described in Technical Releases 36, 37 and 41.

It is best to make these axles with a plastic pipe sleeve bearing between the axle and the hub tube. The best plastic is probably polythene, but this is hard to get in short lengths so we have used PVC in Kenya. Search around the stores in your area to see what is available. If the only steel pipes you can find for the axle and hub tube do not have enough gap between them for a plastic bearing then you can still get good performance without using the plastic sleeve, but the axle will wear a bit faster and the cart will be a little harder to pull. Alternatively you can enlarge the hub tube a little see the **modifications** section.

You will see from Figure 1 that the wheels are fixed to the axles by struts rather than by thick metal discs as on most axles. Putting the fixing studs on struts like this is much easier and cheaper. It also means that you might be able to fit a slightly different wheel by bending the struts a bit to fit. Or if that does not work you can even cut nearly through the welds and then weld them in the right place. You could even cut the struts right off and weld on a different number if your wheels have a different number of holes.

Performance

Many farmers and water carriers in Kenya have used axles similar to these axles on their carts for three years or more without even a plastic bearing. Of course you need to clean the axle and hubs out and re-grease them every month at least and if you use them heavily you should grease them every day. Actually this is easy because all you need to do is remove the hub restraint pin through the end of the axle, take the wheel and hub off, put some grease on the axle and replace everything. It should only take a few seconds.

Length of axle

You need to decide how long to make your axle. Of course you can make the axle any length you like to suit your cart, but most carts will need an overall axle length of between 1500 to 1800 mm. You will need to make the track of the wheels so that the wheels do not rub on anything. You should keep the wheels and tyres about 50mm away from any part of the cart. If the cart

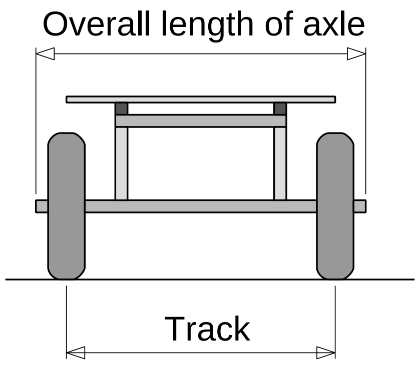


Figure 2: track width of cart.

is to use roads with deep ruts in them it may be a good idea to make the track of the cart the same as the ruts so that the cart stays level on these roads.

In the **cutting list and cost** section you will see that you can make the axle with a central section of some other material than the 2" BSP if this is very expensive. We have assumed that you will use square box tubing but you could use two pieces of angle iron to make a square tube.

To calculate how long to make the axle decide how big you want the **track** to be (see Figure 2) and add 310 mm. This distance will now be the **overall length of the axle** (see Figure 2 for this also). If you are going to use square box tubing or angle iron for the central section of the axle then you should subtract 300 mm from the **track** and make the central section this long (we have shown this to be 1200 mm long in the cuting list but you will probably find you need to make it a little different).

Table 1: materials and costs.

component	material	number & length reqd [#× mm]	materials cost in Kenya [£UK]
central axle	60 mm square box tubing	1×1200	3.82
hub stub axles	2" BSP malleable iron pipe	2×350	2.37
hub outer tube	2½" BSP malleable iron pipe	2×200	2.50
PVC plastic brg	2" PVC plastic pipe	2×250	0.47
axial thrust rings	10mm or 12mm square bar	4×154	0.52
thrust ring tags	10mm or 12mm square bar	4×154	0.02
hub restraint pegs	10mm or 12mm square bar	90×2	0.10
wheel studs	M12 × 50 mm bolts	10×50	5.00
wheel stud struts	6×40 steel flat bar	8×37	0.47
TOTAL COST =			14.80

Cutting list and costs

The table shows a cutting list for a complete axle - two wheel hubs with stub axles joined by a square tube section in the middle. We have shown this way because round pipe is very expensive in some countries. But if pipe is reasonably cheap where you are then make the whole axle out of one piece of pipe.

Recent prices of materials in Kenya for the axle are shown in $£_{UK}$. The 2" BSP (British Standard Pipe) is about 61mm outside diameter, or a bit less, with a wall thickness of about 3.6mm. The 2½" BSP is about 76mm outside diameter, with a wall thickness of about 3.6mm.

These instructions deal with making an axle to the design shown in the drawings. If you find that you cannot get the right sizes of material you might still be able to make an axle with other sizes. See the **Modifications** section later in this booklet.

Construction step by step

1. The first and probably most difficult job, is to get some suitable pipes. Then get everything else togerther and clear a space to work - ideally on concrete.

2. You will probably find that no plastic pipe fits properly between the axle pipe and the hub tube but all you need to do is slit a piece along its length and open it up a bit or close it down until it fits. A better way is to make a helical saw cut in the plastic pipe so it is like a spring. Then it will open up or close down to the size easily and it will not tend to wear the area around the slit.

3. When you have got the right pipe sizes, you can cut the two hub pipes each 250 mm long and the axle pipe about 1500 mm or 1600 mm long, or if you are going to do it with a central section in square pipe or angle iron, you need to cut the central section about 1200 mm long plus the two stub axles each about 500mm long. If you need to support a simple one piece pipe axle an easy way is to weld short pieces of round bar onto it as shown in Figure 4. If you make these about 300 mm long they will strengthen the axle quite a lot as well.

   Figure 5 shows how you can weld the stub axle tube into the square box tubing. You will need to cut slots in the sides of the square tube so that it can bend in or out to accept the stub-axles. When you are welding the stub axles in, make sure you tack weld them in first and check that they are straight before you do the final weld.

4. Next you need to make a hole near each end of the axle pipe for the hub restraint pin. To do this without using a drill cut a slot in each side of each end of the axle. Make the slots about 30 mm long to start with and about 10 mm wide. You want the slots just the right length so that with the hub

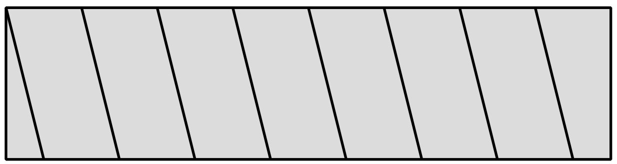


Figure 3: helical sawn cut in plastic pipe bearing sleeve.

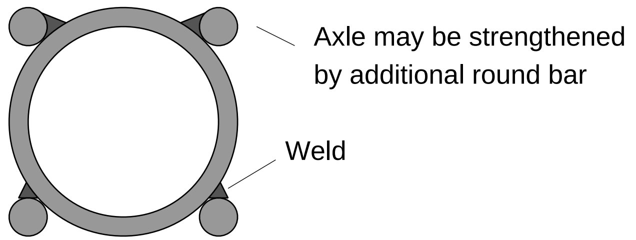


Figure 4: method of adding round bar to support pipe axle and strengthen it if needed.

restraint pins in place the hub tubes are not pinched tight. It does not matter of they are a bit loose.

5. The next step is to weld the stud bolts or bits of threaded rod onto the struts. If you have a vice it makes welding the bits together easier because you can clamp the pieces together while you tack weld them. You need to make one strut for each wheel stud unless you are using five or six stud wheels on a single donkey cart. If so you could use only three studs per wheel if the cart user wants only light

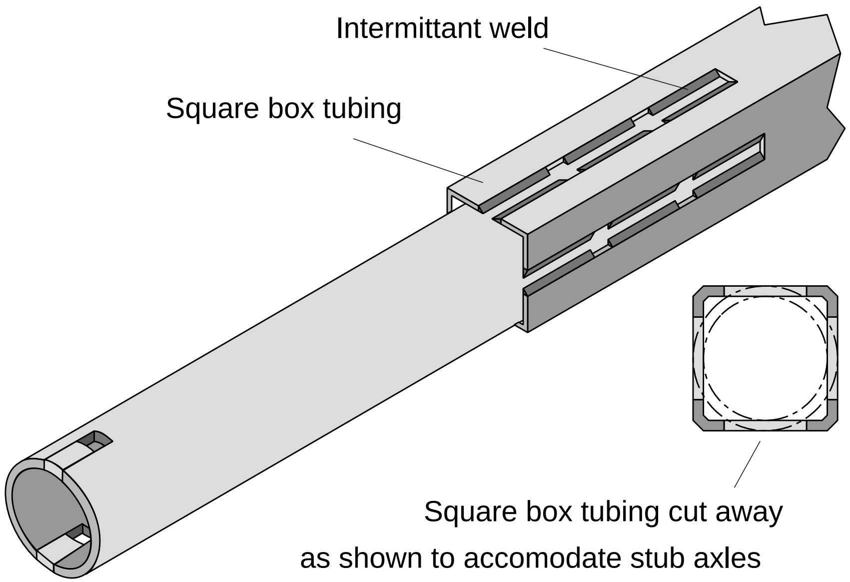


Figure 5: method of welding stub axles into square tube.

use from the cart.

We have used this method of fixing wheels successfully with the old VW wheels which have very large holes in the middle.

When you've made up the struts you can put a nut on each stud and then put the studs through the wheels, and put a second nut on each stud. Then get everything even and straight with the hub pipe in place as well in preparation for welding the struts to the hub tube. You want to get the middle of hub pipe level with the middle of the tyre, as is shown in Figure 7. Most wheels need the studs to be about 60mm offset and this is what is on the drawings.

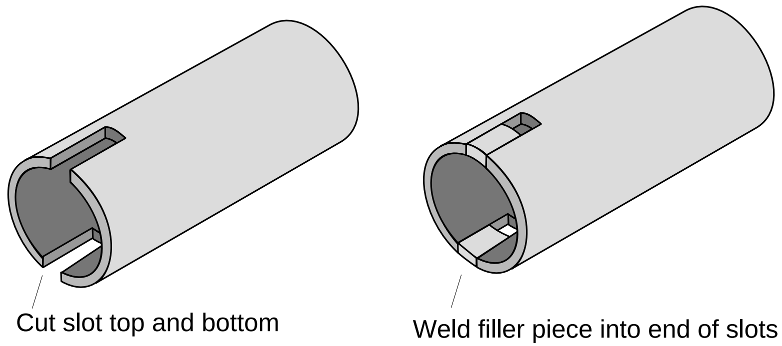


Figure 6: method of making hole for hub restraint pin without drilling.

wedges as shown in Figure 8 is a good way to do it.

When you are happy, tack weld the struts to the hub pipe. Then remove the wheel and wedges so you've got room and finish the welding of the struts to the hub tube. Repeat this for the other hub. If you are going to make several axles you can make up a simple jig, rather than the wedges, to hold everything for welding. We have used a piece of plywood with a central hole to fit snugly over the hub tube and four holes for the studs. In other words its a bit like a dummy wheel.

It is best to check that the hubs, and stub-axles still go together when you've finished welding because sometimes weld contraction can pull it all out of shape and make it all too tight. You might need to file off some high spots inside the hub. If you can get it together without a hammer you'll be ok because it will wear to the right shape.

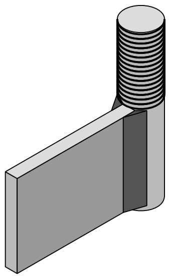


Figure 7: stud welded to the wheel struts.

6. Nearly there! Now take the wheels off the hubs and make up four rings (called hub restraint rings on the drawing) from 6 mm or 8 mm square steel bar (or round bar if you cannot find square). One ring must be welded about 230 mm from each end of the axle pipe or from the ends of the stub axles to stop the hubs going too near the centre of the axle.

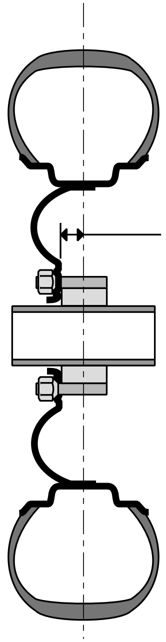


Figure 8: cross section of tyre wheel and hub tube showing centering of hub tube in wheel.

7. Now put it together! Smear some grease onto the plastic bearing pipe (if you are using one) and onto the axle and put the hub tube and plastic bearing onto the axle. Then put the outer hub restraint ring on and secure it with the hub restraint pin. Do the same for the other hub.
8. Paint everything!
9. You've done it!

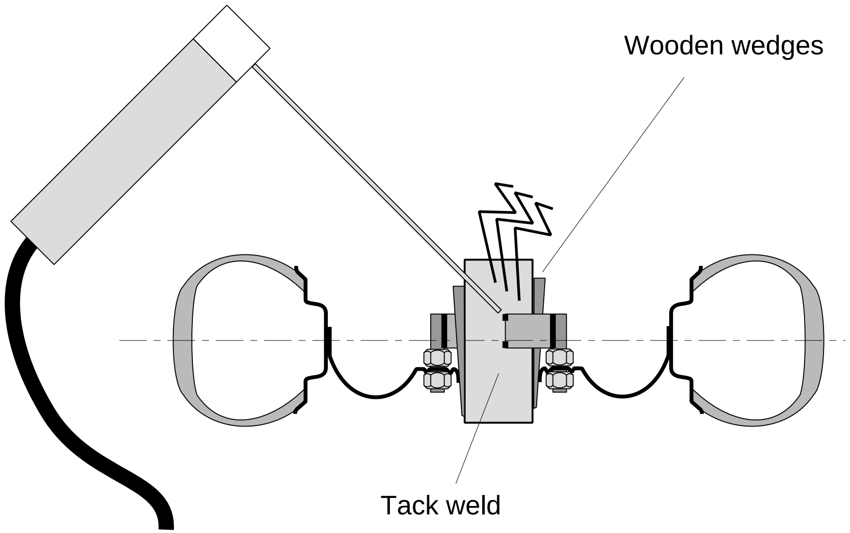


Figure 9: cross section of tyre wheel and hub tube during tack welding of stud support struts

Modifications

Obviously the axle has to be strong enough to carry the cart, so it should be made from pipe bigger than about 50mm outside diameter. You must also make sure that the pipe has a wall thickness of more than about 2.5 mm.

The hub pipe also must have a wall thickness of 2.5mm or more. And it must have a bore (or inside diameter), which goes easily over the axle with enough room for the plastic bearing and some grease. There can be quite a lot of clearance (slackness or looseness) between the hub and the axle (say up to 3mm) - it does not have to be tight. If you cannot get steel pipes with enough clearance for a plastic pipe bearing then the axle will work quite well without it, but you should grease it more often.

Pipes can be made slightly bigger (up to 1mm bigger) by forcing a short piece of round bar of the right diameter through them with a press. Another way to do it is to saw the pipe along its length and open it up to the right size and then weld it. You can also make it a bit smaller like this by cutting a wider slot and squashing the pipe down. Try to clean the weld back flush with the tube using a file if you can as this will give longer life. It does not matter if

If you find that you cannot get anything like the materials talked about in the cutting list then maybe you can adapt the design a bit. If the hole in the middle of the wheels is big then you stand a better chance of finding a combination of pipes and rollers that will fit. You can often cut a bit out of the middle of the wheel to make the hole bigger. The hole in Land Rover wheels is big and you can get 4" pipe into them. Of course the shaft does not have to be a pipe - it could be solid and then it could be a bit smaller, say 30mm diameter if the steel is high quality.

Another idea that we have tried is to use a hardwood as the hub and even the wheel. If you think about it, the wear on something which is rolling must be less than when something is sliding over it, so a wooden bearing should be better than a sliding one. Some bearings we have tried have had a steel ring fitted inside so that the rollers roll on this steel. We have also tried making these rings from round bar like wire so that it's like the rollers roll on the inside of a spring. This seemed to work quite well.

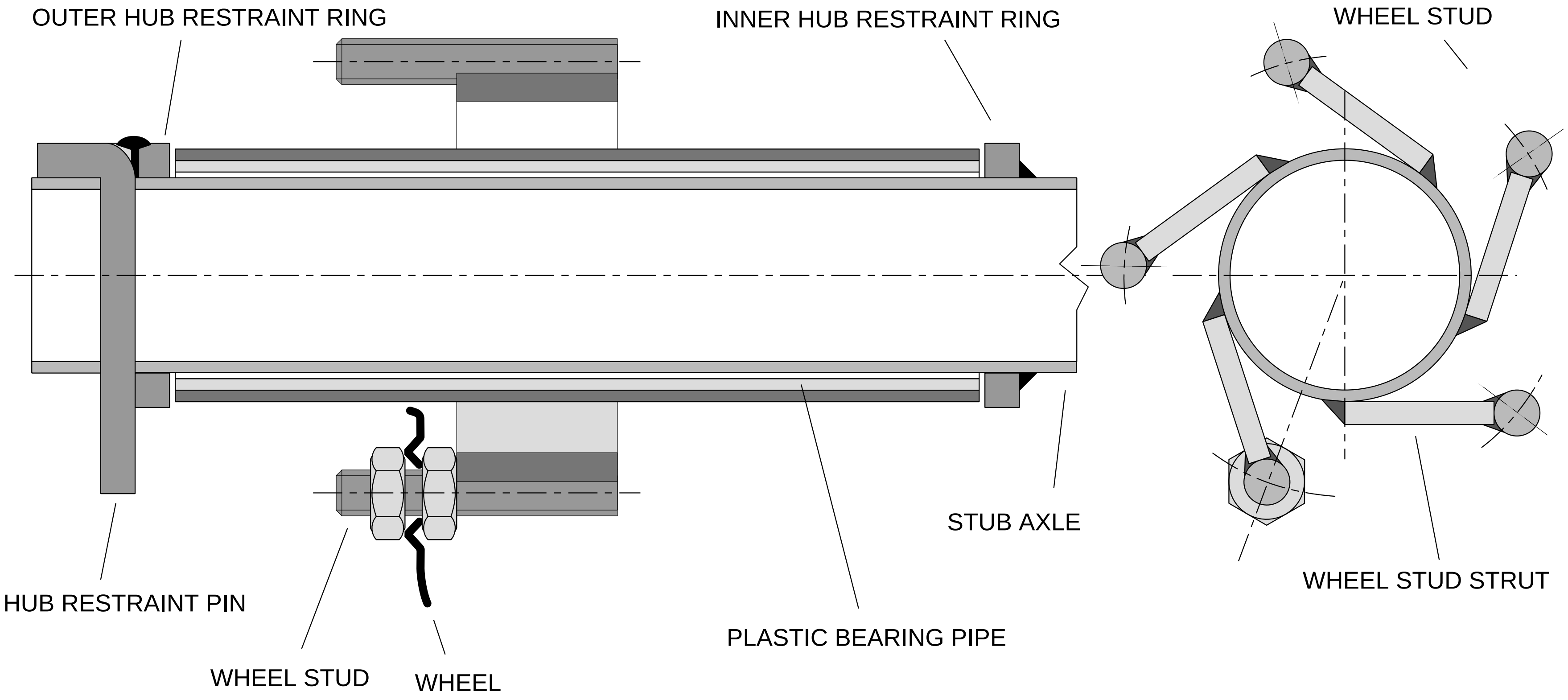
Other DTU cart developments

Other methods of hub design using aluminium castings, for example, which might need no machining, are under development at Warwick and wheel designs in steel sheet, cast aluminium and timber are also in manufacture or under development. A range of designs for donkey and ox carts made of steel and wood, is also available, some of which are in production in Kenya and Uganda.

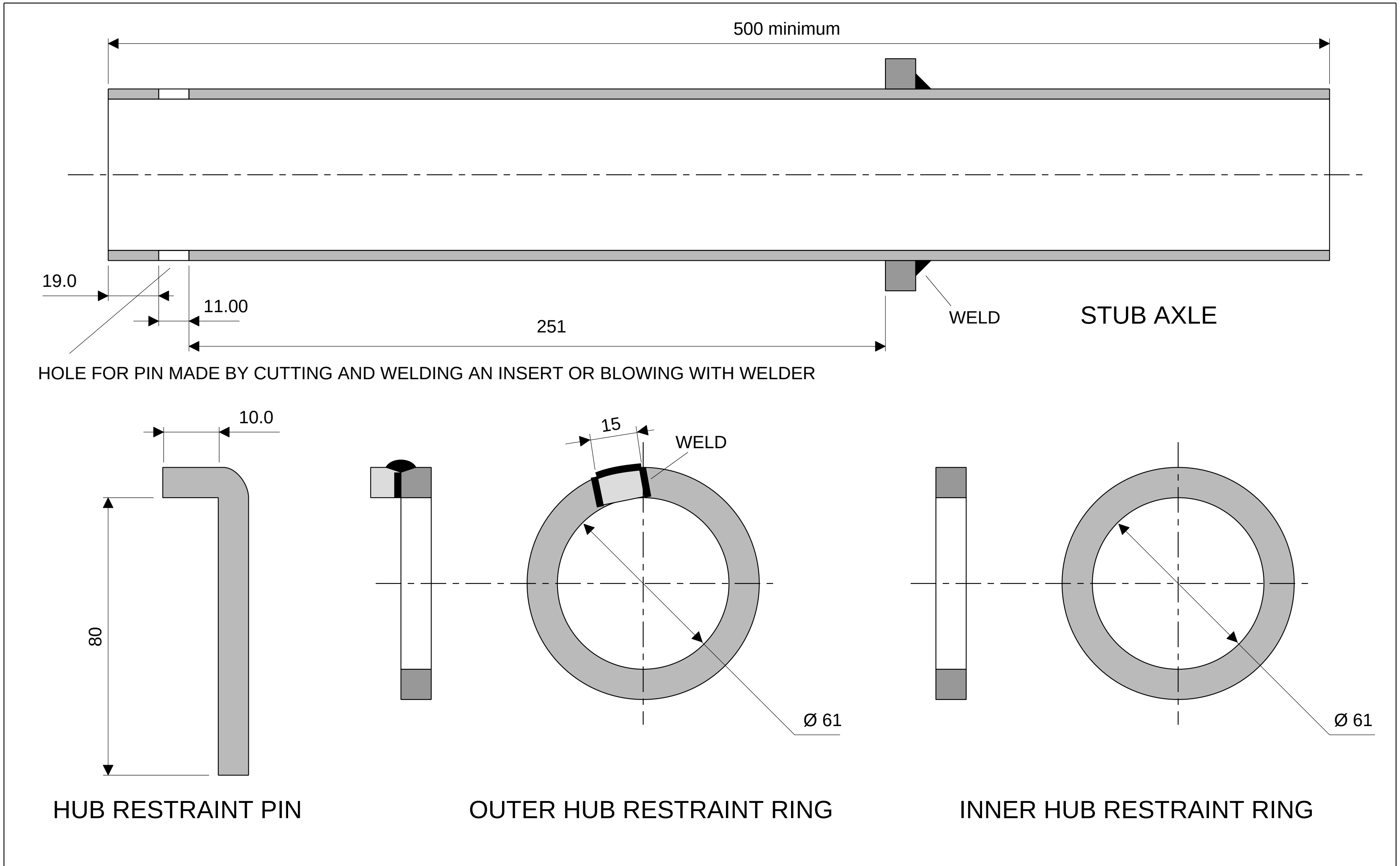
Acknowledgements

The DTU is grateful to the DFID (British Government) for the financial support necessary to carry out the research and development project under which this product was developed.

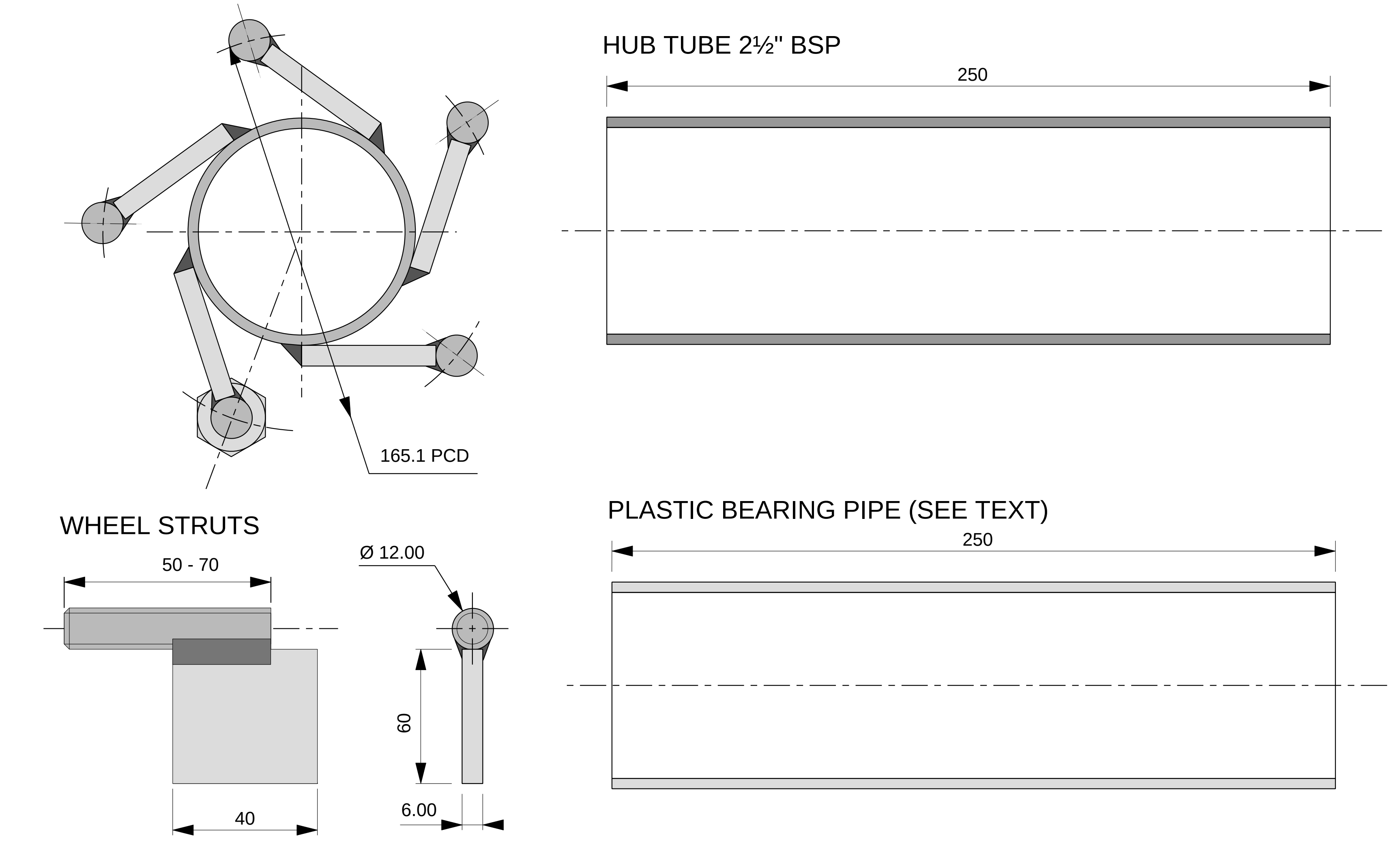
The DTU would also like to thank Dr Pascal Kaumbutho of KENDAT in Kenya and Mr Joseph Mugaga of TOCIDA in Tororo, Uganda for their very considerable help with this project. A large number of other people and organisations have contributed to the success of the project, most notably Mr Anthony Ndungu in Kajiado Kenya, Mr JD Kimani in Kikuyu Kenya and Mr Joseph Gitari in Wanguru Kenya in whose workshops most of the development work of this project was performed. Thanks are due also to Mr Stanley Lameria in Kajiado, Mr Patrick Gitari in Wanguru and Mr Mathew Masai in Machakos for their assistance.



	GENERAL ASSEMBLY DRAWING	Scale	10mm	Title PVC PLAIN BEARING OX CART AXLE	Drawn by	CEO
		Date	1-1-99		Dwg No.	1/3



	HUB AND AXLE COMPONENTS	Scale	10mm	Title PVC PLAIN BEARING OX CART AXLE	Drawn by	CEO
		Date	1-1-99		Dwg No.	2/3



HUB TUBE, PLASTIC BEARING & WHEEL STUDS	Scale	10mm	Title PVC PLAIN BEARING OX CART AXLE	Drawn by	CEO
	Date	1-1-99		Dwg No.	3/3

KENDAT

DTU

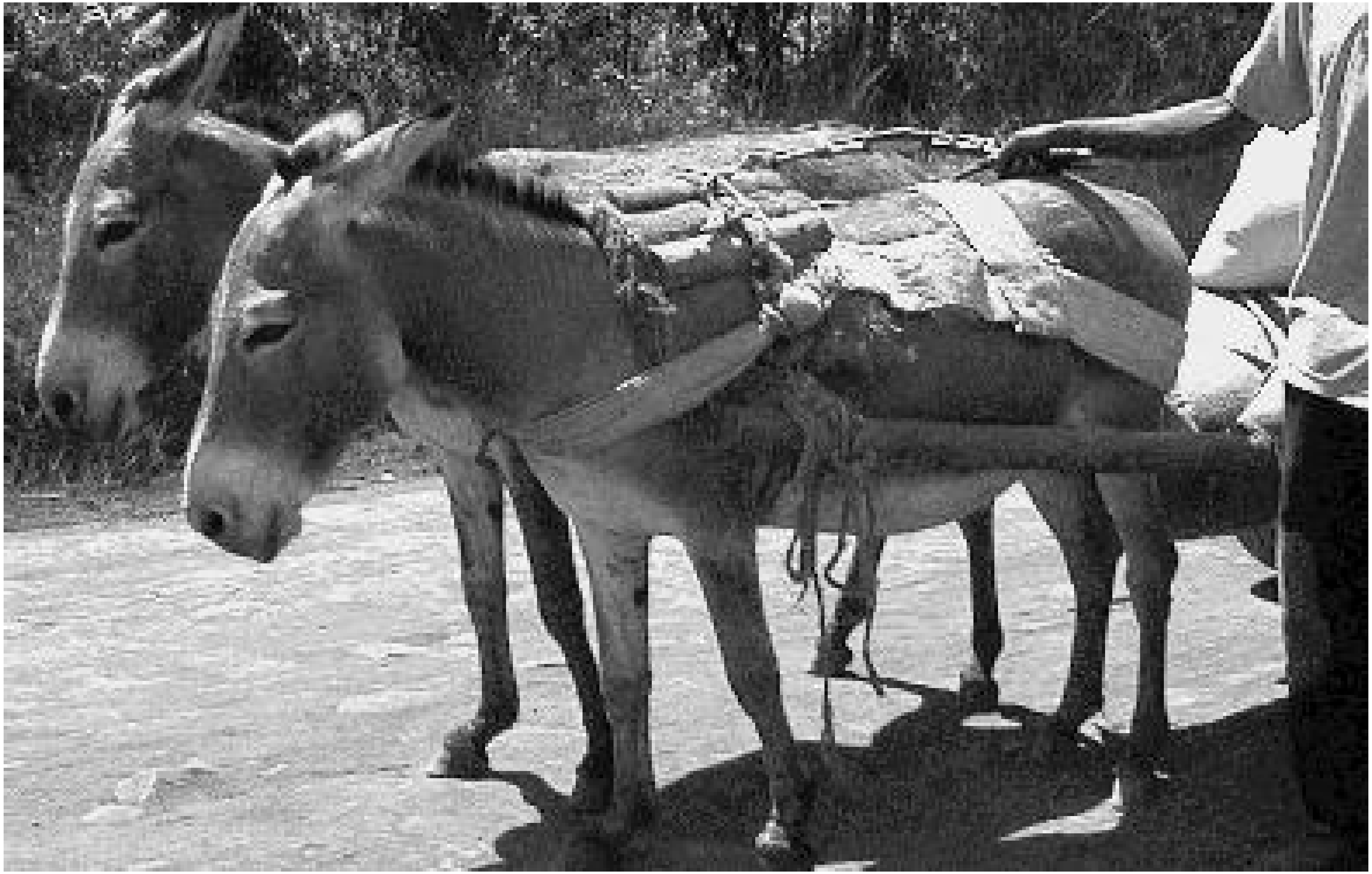
Animal Cart Programme

TECHNICAL
48
RELEASE

Simple Chain and Wood Saddle for Donkeys

Development Technology Unit, Department of Engineering, University of Warwick, Coventry, CV4 7AL UK, tel: +44 (0)1203 523523 extn 2339, fax: +44 (0)1203 418922, email: dtu@eng.eng.warwick.ac.uk
KENDAT, PO Box 61441, Nairobi, Kenya, tel/fax: +254 2 766939, email: kendat@africaonline.co.ke

Figure 1: locally made chain and wood block saddle for donkeys.



Chain and wood block saddle for donkeys.

Introduction

This chapter covers a simple but effective saddle design for donkeys pulling carts, though it can be used for other purposes as Figure 2 shows. Saddles are necessary in a cart harness because vertical loads must be applied by the animal to stabilise the cart. We have adapted the saddle shown here only slightly from a design used by many farmers in the Mwea area in Kenya.

This saddle is extremely easy to make. All you need is some light chain (made from 2mm or 4mm steel wire) some wood and some fencing nails (U-nails). The only tools which you must have are a woodsaw and hammer. You will probably find that you can make the saddle for one donkey for about £$_{UK}$3 for a version for use with agricultural implements or about £$_{UK}$6 if you need to make up the belly and breeching straps for use with carts. You can probably make a saddle in only a couple of hours.

Idea behind chain saddles

Saddles are used to apply downwards load from a cart or from an agricultural implement onto donkeys. Saddles must be used because donkeys' backs are easily damaged by things put on them. Fat donkeys will be much more resistant to injury, but donkeys in poor condition have much thinner muscles on their backs and this tends to expose their backbones. Any hard object (even a strap) just stretched over their backs can cause damage as the skin over their backbone is compressed. After some time this pressure will damage the skin and infection can start which can kill the donkey.

Figure 2: simple version of chain saddle used with steel version of straight hame donkey collar.

A properly shaped saddle will protect the vulnerable area of skin and put the load onto the muscles which cover part of its back. Figure 3 represents an animal in good condition and Figure 4 shows the vulnerable area of skin above the backbone. A saddle must be shaped on the underside to keep clear of this vulnerable skin.

Cutting list and costs

Tables 1 shows a cutting list for a saddle with recent prices of materials in Kenya converted into $£_{UK}$.

TABLE 1: steel straight hame collar materials cutting list.

component	material	# components	mat cost [$£_{UK}$]
chain	dog or round wire chain	1	1.40
wooden blocks	100 × 50 mm roughsawn	4 × 250	0.40
padding	blanket/ sacks	2	0.80
U-nails	U-nails	¼ kg	0.60
strap hooks	6mm re bar	2×150	0.03
strap rings	6mm re bar	6×180	0.11
strap clenchers	6mm re bar	6×120	0.07
strap hooks	6mm re bar	6×150	0.09
straps	CC5 canvas	3×4×65	1.97
strap chains	dog chain	3×300	0.70
		TOTAL =	**6.14**

Construction step by step

1) Get all the material together and clear a space to work. Ideally you will be able to work on a flat area of concrete or have access to a bench and vice. It is also easiest if you have the donkey standing by to try the saddle fitting out and position things.

2) Cut four pieces of 100 × 50 roughsawn timber 250 mm long and round off all the corners with a file or carpenter's plane.

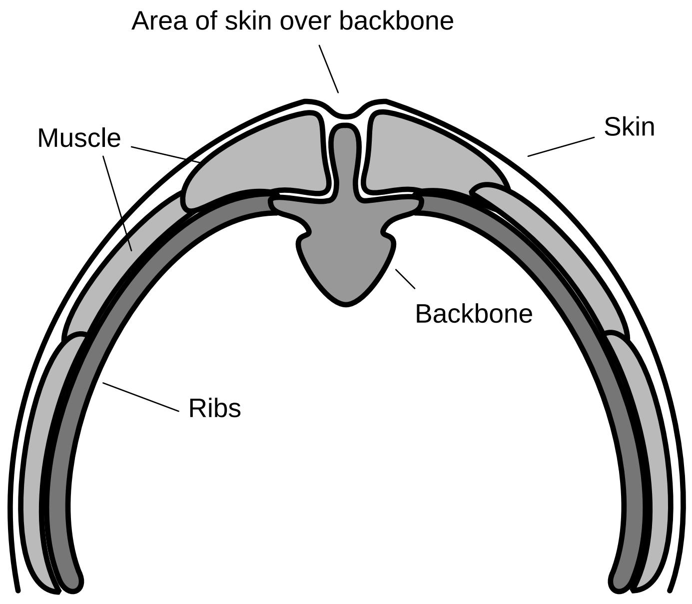


Figure 3: skin over the backbone of donkey in good condition is not vulnerable.

3) Using U-nails, fix each of the top wooden blocks 75 mm either side of the centre of the chain and the other two blocks a further 75 mm away from them as shown in Figure 5.

4) Make up the padding from blanket or natural fibre sacks (DO NOT USE PLASTIC SACKS). You can either leave this padding separate from the saddle or you can nail it on to the wooden blocks. If you nail it, you must use another layer of cloth under the the nailed padding to make sure the nails do not touch the animal.

5) It may be easier to fix the saddle to the rest of the harness if you make up two of the hooks shown in Figure 6 and fit one to each end of the chain.

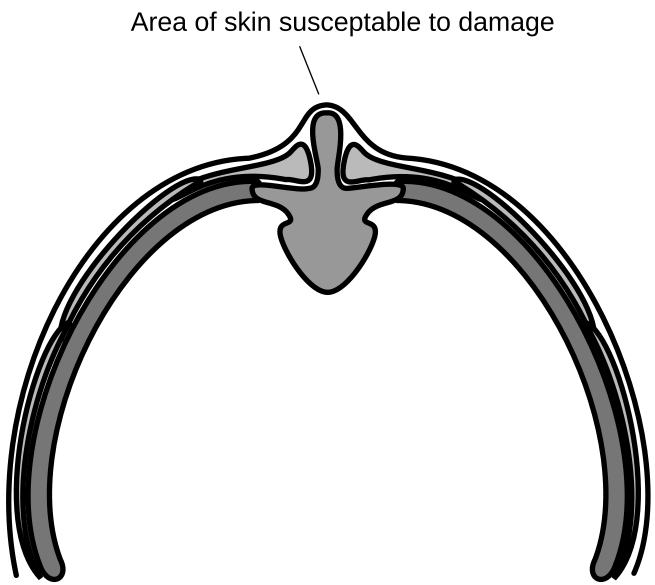


Figure 4: skin over backbone of donkey in poor condition is very vulnerable.

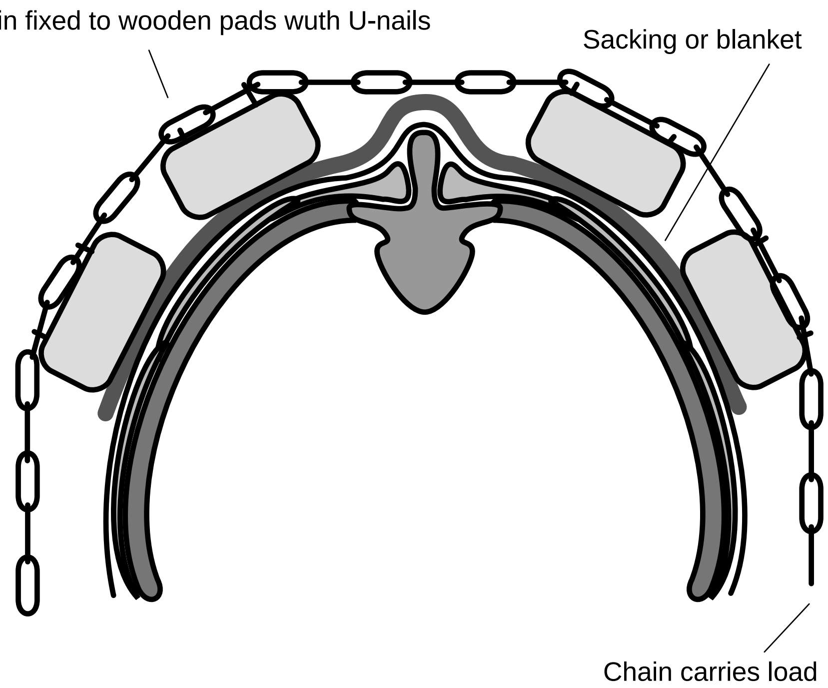


Figure 5: height of wooden blocks keeps chain away from backbone. Note sacking/ blanket padding.

6) If you are going to use belly, breeching and breast straps for cart use, make up six more of these hooks and fit one to the chain between the wooden blocks on each side, and one to the chain just below the lower wooden blocks, as shown in Figure 7.

7) If you are going to use the saddle for pulling carts make up a belly strap, a breeching strap and a breast strap to hold the saddles onto the donkeys. The D rings at the end of the straps can be made from 6 mm diameter concrete reinforcing bar as shown in Figure 8. A separate piece of the re-bar is clenched over the strapping using hammer blows to fix the D rings to the ends of the straps as shown.

   The straps themselves can be made from heavy canvas or hessian sacking. You should use three or four thicknesses of material for them to make them strong enough and soft enough not to hurt the donkey.

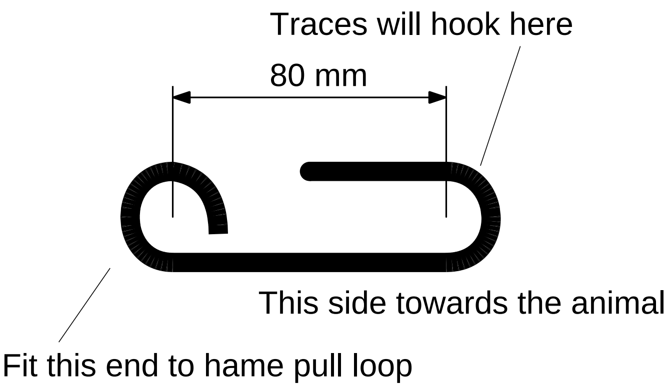


Figure 6: chain hooks for straps.

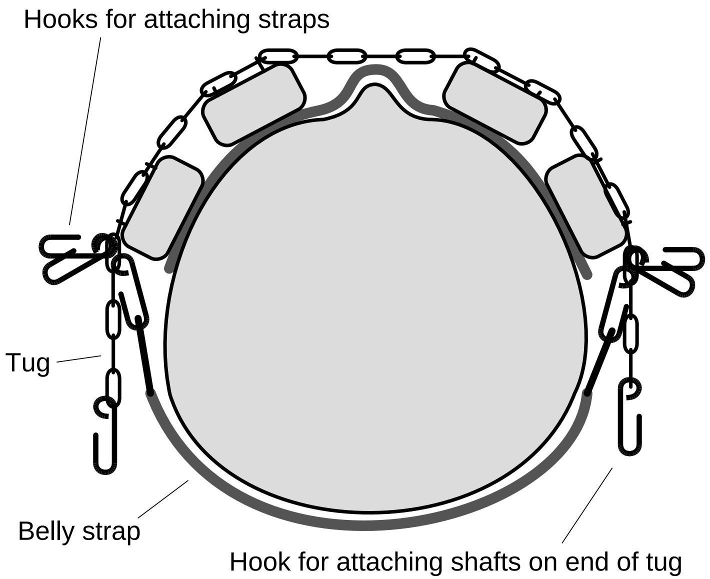


Figure 7: arrangement of attachment hooks for straps and shafts on saddle chain.

Using the saddle with agricultural implements

(Remember that protecting the donkey will save money because it can work harder if it is comfortable and will not get sick from skin wounds.)

Agricultural implements normally demand high draught and this is usually only available using a collar harness of the type described in our Technical Release 46. Breast straps are difficult to use with most donkeys because their chest are poorly proportioned for this work. It is even worse if you want them to pull hard.

It is usually easier to use a saddle with agricultural implements (plough or weeder etc) because the ideal angle for the traces is rarely the same as the ideal angle for the implement and, using a saddle, these can be adjusted independently. Figure 9 shows the general layout of harness when using the saddle with a collar.

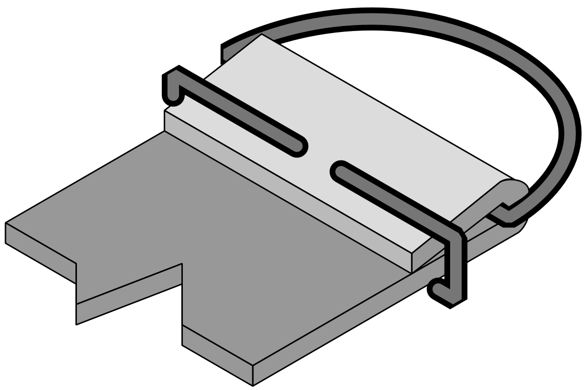


Figure 8: D rings for straps made from re-bar.

1) Place the saddle on the donkey's back using a separate piece of blanket or sacking under any nailed on padding.

2) Check that there is plenty of room under the chain you must be able to get two fingers under the chain - it MUST NOT TOUCH THE DONKEY'S BACK! Figure 10 shows a poorly arranged saddle with one of the wooden blocks nearly over the backbone.

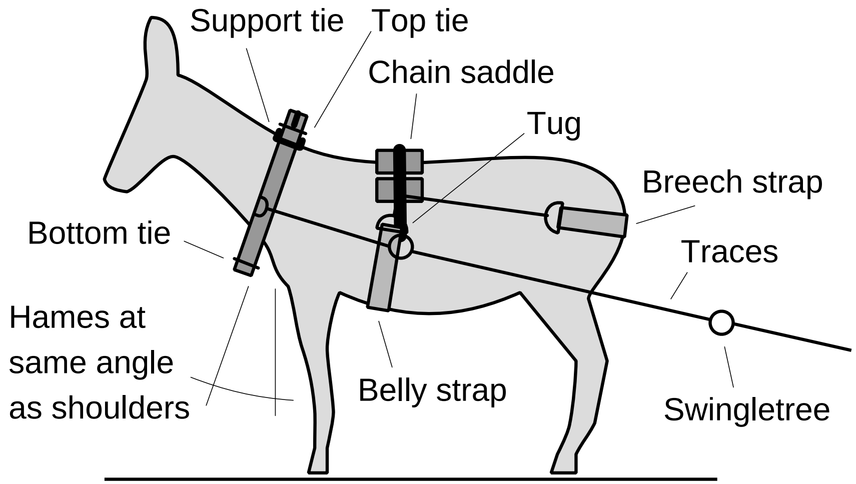


Figure 9: saddle and collar position on donkey with traces pulling at right angles to hames.

3) Adjust the lengths of the tugs (the ropes or straps hanging from the saddle if you use one) so that the traces (the ropes or chains which will pull the cart or implement) come back at right angles to the animal's shoulder blades (see Figure 9).

4) Fit the breeching strap if you find that the saddle rides forward as you work, but you will usually find that the backward pull on the tugs keeps it in place.

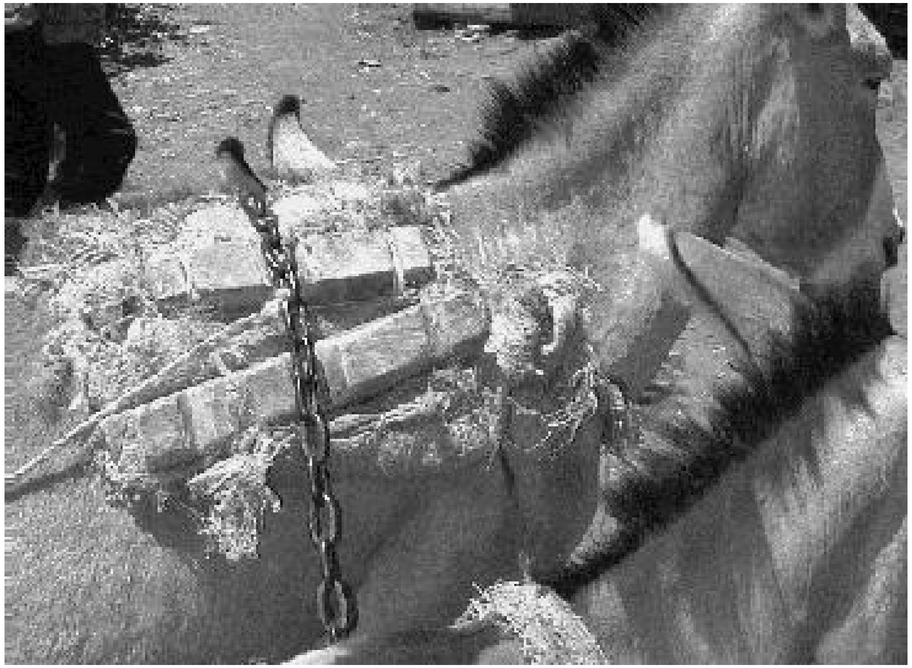


Figure 10: inadequate height of wooden blocks and poor positioning endangers skin over backbone. The sacking/ padding is also poor.

Saddle use with carts

(Remember that protecting the donkey will save money because it can work harder if it is comfortable and will not get sick from skin wounds.)

If you are going to pull a cart you must use a saddle because the cart will need some up or down force to stabilise it. A donkey's back needs the protection that a saddle gives against this force. If people just use neck yokes in your area you will usually see damage on the necks and shoulders of their donkeys.

Figure 11 shows a typical arrangement of breast band, saddle,

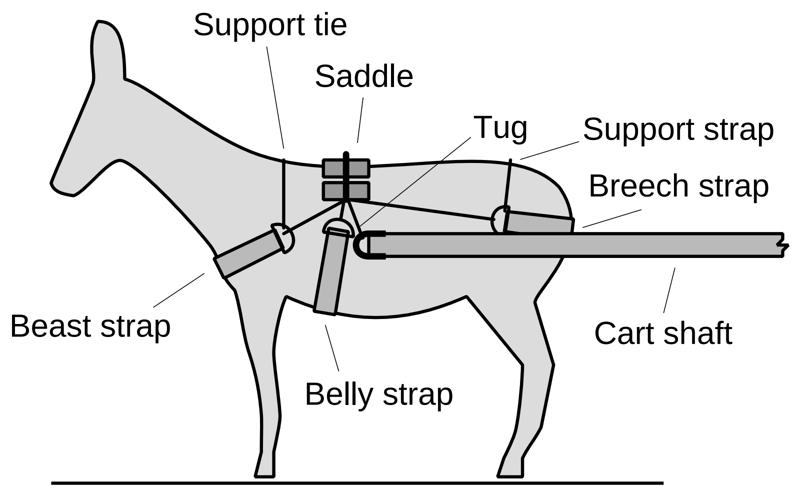


Figure 11: attachment of saddle to cart and use of breast strap.

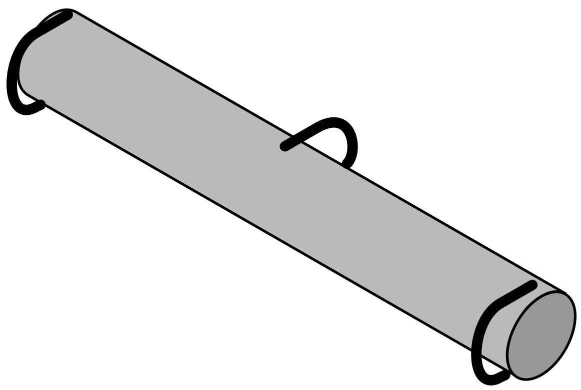


Figure 12: swingletree: the traces will be tied to the ends, and the cart or implement tied to the central loop.

belly strap, breeching strap, tugs and cart shafts.

Using a breast band to pull the cart is usually acceptable, but if the cart is very heavy then you should use the collar described in our Technical Release 46.

1) Place the saddle on the donkey's back using a separate piece of blanket or sacking under any nailed-on padding.

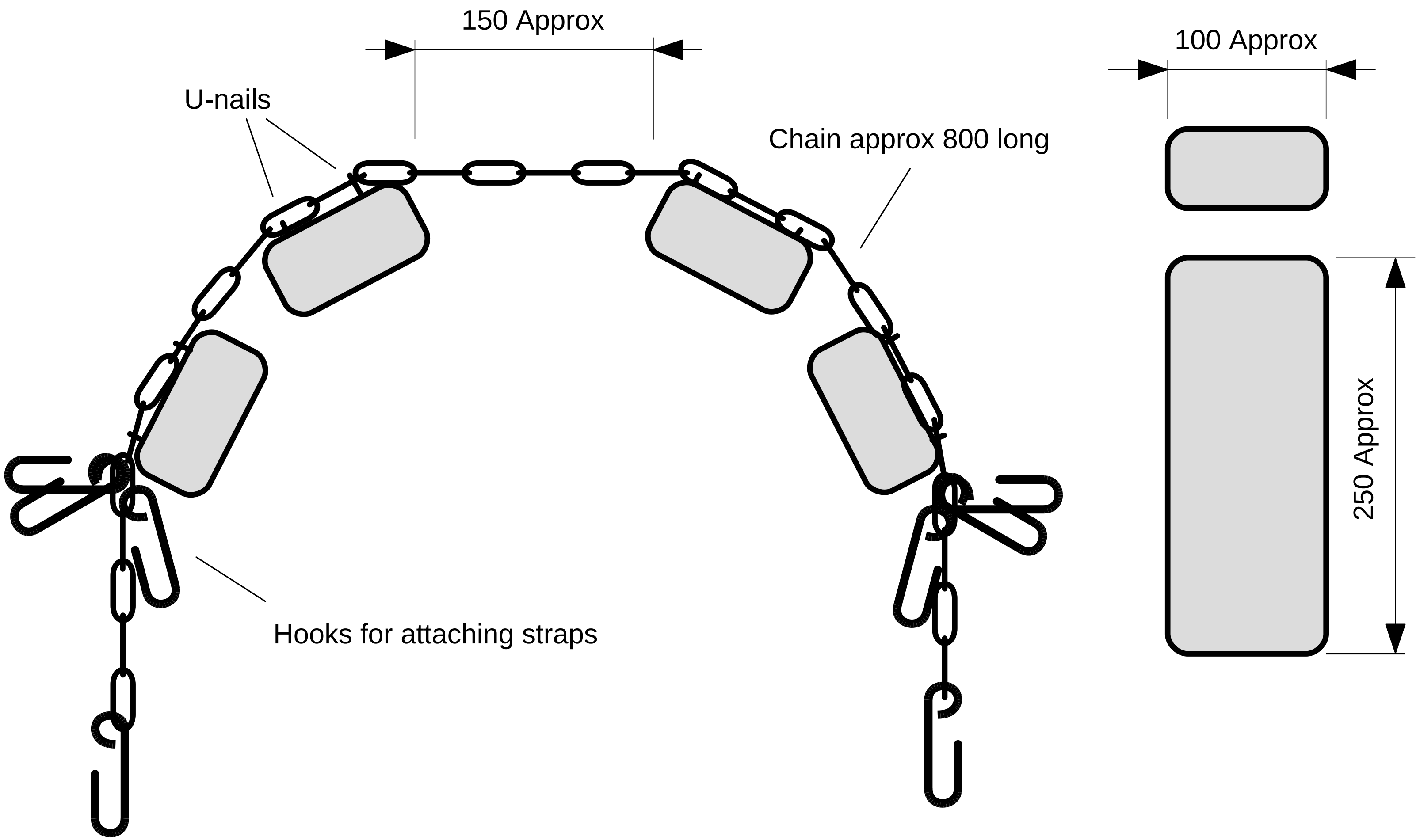
2) Using two of the hooks shown in Figure 6 attach the belly strap. It should be quite tight so that it does not move during work.

3) Check that there is plenty of room under the chain over the backbone: you must be able to get two fingers under the chain - it MUST NOT TOUCH THE DONKEY'S BACK!

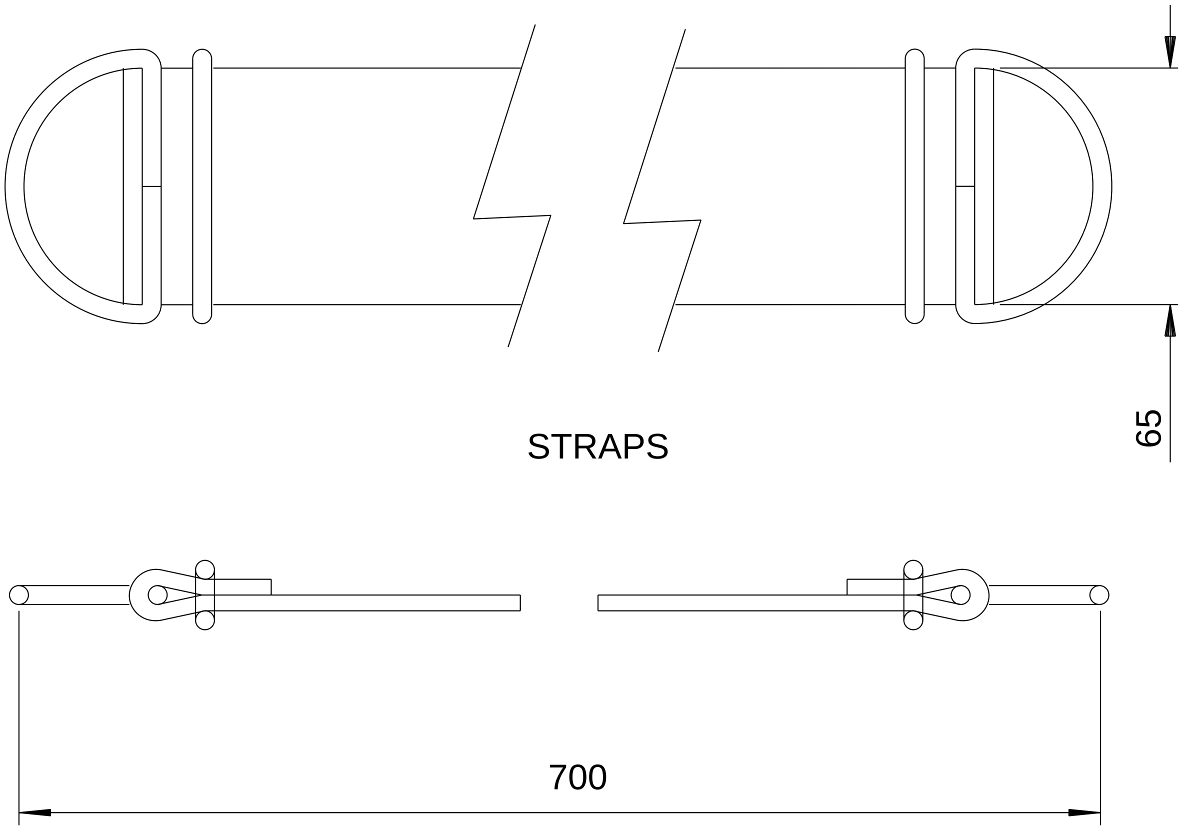
4) Attach the breeching strap to two of the hooks just below the lower wooden blocks. Tie a piece of rope or cord across from the D-ring in one end of the breeching strap to the D-ring of the other so that the strap will not fall out of position if the strap goes slack. The breeching strap will be nearly loose.

5) Now bring the shafts of the cart either side of the donkey and attach the tugs to the shafts so that the cart is level.

6) Lastly attach the breast band to another pair of hooks below the wooden blocks of the saddle or lead traces (pieces of rope or chain) from the ends of the breast strap to a swingletree on the front of the cart. A swingletree is shown in Figure 12. Use of the swingle tree is kinder to the animal because it allows a little movement of the breast strap with the movement of the animals chest as it moves. Make sure that the traces do not rub against the sides of the animal.



Scale	25 mm	Title	Drawn by	CEO
Date	25/12/99	Chain saddle for donkeys	Dwg No	1/1



					Scale	25 mm	Title	Drawn	CEO
					Date	25/12/99	Straps	Dwg #	2/2

Design of Rainwater Storage Tanks for use in Developing Countries

Stephen Turner

EDAT Year Four

Department of Engineering
University of Warwick

27th April 2000

SUMMARY

This project is to investigate the problems associated with ferrocemnet water storage tanks in developing countries, with the aim of giving the engineer a series of practical tips that will help with tank construction in the field. Using the findings of the project a series of a construction rules have been produced. The aim is to overcome the problems that are particular to constructing ferrocement water tanks in hot dry climates.

In rural areas of many developing countries, there is a scarcity of water. Traditionally, rainwater collection has provided valuable source household water. Therefore there is a need to provide simple, economical storage facilities that can be constructed with semi or unskilled labour. Approximately 80% of the cost of a ferrocement tank is the construction material. Due to the high cost the majority of water tanks are financed through donor funding. To enable self-sufficient production material costs need to be reduced. This project looks at efficient ways of reducing material inputs.

The first stage is to carry out structural analysis. For this thin shell theory has been used. Excel spreadsheets have been produced to allow the designer to vary the range of tank configurations and material characteristics. Initial findings from the analysis show that cylindrical tanks with curved walls can withstand greater loads than cylindrical tanks with vertical walls. Comparing a Thai jar (figure 2.7(b)) style water tank to standard cylindrical tank (figure 2.7(a)) of a similar volume material inputs can be reduced by approximately 30% (assuming the material is homogeneous).

Little is known about the mechanics of shrinkage, therefore this has been the focus of the experimental work carried out on ferrocement tanks. The author's new physical test shows how shrinkage can be reduced through the use of reinforcing. A range of reinforcing systems has been tested and results show that a thin-wire steel square mesh is the most effective of the three examined.

The report investigates how the environment in which the tank is constructed plays an important role in the degree of shrinkage and cracking. It is shown that tanks constructed in a hot and dry environment and tanks that are allowed to set at different rates are much more susceptible to cracking.

NOTATION

E	Youngs modulus
g	Gravity
h	Tank height
H_o	Edge loads consisting of shear forces at tank base
M_o	Edge loads consisting of bending moments at tank base
M_x	Bending moment
N_q	Hoop force in tank wall
N_f	Meridional force in tank
PR	Poisson's ratio (used in spreadsheet)
P_c	Portion of the load retrained by cantilever
P_r	Portion of the load retrained by hoop stresses and radial constraints
P_x	Total outward pressure load to be restrained
p_r	Pressure of fluid
Q_x	Shear force
r	Tank radius
r_1	Radius of curvature of the meridian in doubly curved tank
r_2	Radius of curvature of the second principal curve in doubly curved tank
S	Surface area of doubly curved tank
t	Tank wall thickness
v	Poisson's ratio
V	Volume of tank
w	Radial deflection
x	Intermediate point on the tank wall
D_H	Horizontal deflection around the diameter of tank
e_f	Strain in plane of the meridional force
e_q	Strain in plane of the hoop force
g	Intermediate point on tank wall in the x direction
r	Fluid density
x	Intermediate point on tank wall in the y direction

1.0 INTRODUCTION

1.1 A Brief Introduction to Rainwater Harvesting

A good quality RainWater Harvesting (RWH) system provides people with access to an on-site water supply, either next to their homes or at local public buildings such as schools and health centres. Rainwater has been collected and used for drinking water throughout the centuries, but in recent years they have fallen out of favour as they are considered old-fashioned. Ideally, the RWH collection system should involve basic construction techniques, be inexpensive to maintain, and have a long functional life span (Pacey 1986). If the system is designed well, it should provide a good safe source of drinking water at a relatively low cost when compared to the mains supply.

The RWH system provides a good alternative water supply option, especially for rural areas, where the following characteristics apply:

– it operates independently, and therefore gives people access to drinking water without them being dependent on a grid supply which can be unreliable,

– alternate sources of water do not provide sufficient quantities of potable water,

– the available sources of water are of a poor quality, such that the construction and maintenance of expensive treatment plants would be prohibitive,

– the cost of supplying grid water is too high,

– rainwater catchment area i.e. roofs, tend to be larger per capita in rural areas compared to urban,

– pollution levels in rural areas tend to be lower when compared to the towns and cities, making the water more suitable for direct human consumption without treatment.

Other benefits of RWH include:

– it reduces soil erosion (especially in the hilly areas)

– it requires a reduced amount of valuable energy inputs compared to the grid supply,

– it localises the process of water collection, which results in a reduction of the amount of civil engineering works compared to grid connection.

There are many aspects to RWH, each of which must be studied and managed correctly, if the overall system is going to run efficiently. They may be classed as follows;

– water usage management,

– water quality and other health issues,

– water collection hardware (storage tanks),

– financial considerations.

In many 'westernised' countries the most common way of obtaining a rainwater storage tank is to purchase it ready-made from the factory. When looking at rainwater harvesting from the perspective of the rural poor, the factory made tanks are unlikely to be a realistic due to their high cost and transportation difficulties. The problem should be looked at from a self-help emphasis within the community. However, since tank construction is a skilled task, any self-help effort must involve specially trained individuals, even if the most basic tasks are left to the householders themselves.

Assistance should not only be in the form of technical skills, but also in the supply of raw materials and help in the method of payment. Several approaches to the development of necessary skills have been used in rainwater projects. Training may be offered to the village craftsmen, as in Kenya (Pacey and Cullis 1986), or to community workers or individuals chosen at village meetings who are given special training as 'village technicians' (Ichikawa 1995).

The design of the rainwater storage tank is not merely an abstract engineering problem, it is related to the type of assistance, the sort of materials and other resources that are locally available. In places where satisfactory rainwater storage jars or tanks are already available, assistance may come in other forms, such as, offering advice on what type of tank to buy, financial advice, health, or other hardware advice such as gutter construction.

Self-sustainability should be the ultimate aim of any RHW project, and where possible it should be independent of any external subsidy. Ideally the storage tank should be able to be constructed by local craftsmen, where possible using locally available materials, and funded by either the individual or community. Self-sustainability is not only the ability of individuals or communities to pay for and build their own tanks, but also to maintain them, so that the benefit the tank offers is permanent.

Once the tank is constructed, its performance should be monitored. Attention must be paid to deficiencies in maintenance, such as keeping gutters clean, as well as any other defect the tank may develop. As well as practical advice on tank construction and maintenance, other factors should be addressed such as water management and health issues regarding the stored water.

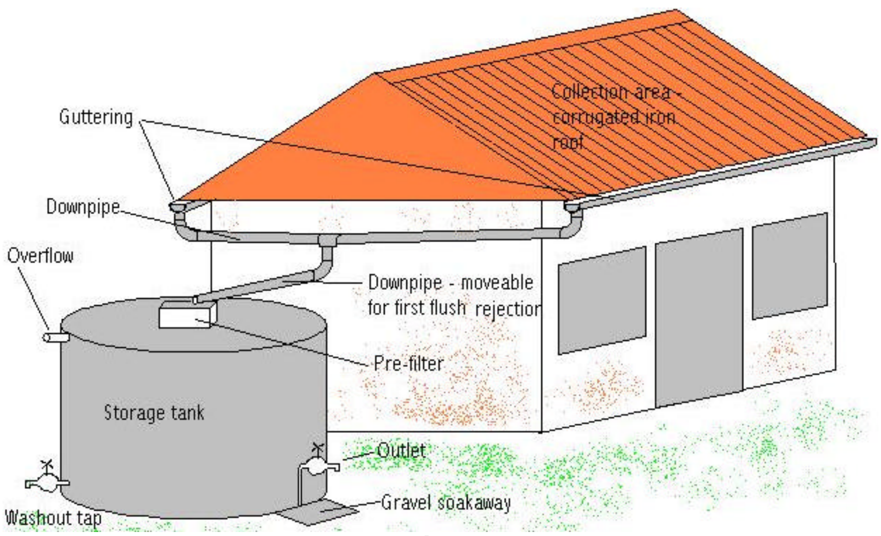


Fig. 1.0 Domestic rainwater harvesting system (DTU 1998).

From the engineering perspective, there are a number of RWH technologies that can be improved upon. These technologies, which involve the water collection and storage side of RWH, can be divided up into a number of key elements (see figure 1.0).

Tank sizes vary depending on ownership, domestic water storage tanks range from 1 to 10m^3 see figure 1.1, community water tanks vary from 10 to over 100m^3 (figure1.2). The main size limiting factor for domestic tanks is cost. For community tanks cost is a factor, but they are also size limited by catchment area and rainfall patterns. For typical ferrocement tanks constructed in Kenya costs vary from 50$US\$/m^3$ for a 11m^3 tank to 26$US\$/m^3$ for a 46m^3 tank (Gould 1999).

Fig. 1.1 A 8m^3 domestic ferrocement storage tank in Sri Lanka

Fig. 1.2 A 46m^3 community ferrocement storage tank in Uganda

As well as the above factors, ease of manufacture is an essential part of any good design. The majority of RWH projects are in rural areas, which may lack the resources and infrastructure that is available to the urban designer. Levels of construction skills may also be limited. This being so, ease of manufacture is a very important area, where possible the tank should be manufactured using a limited range of materials and tooling.

1.2 The Project

The project will investigate the problems associated with building water storage tanks in developing countries, with the final aim of giving the engineer a series of practical tips that will help with tank construction in the field.

The project will only investigate water storage tanks built above ground, where all the forces are carried by the tank walls. Water storage tanks can be constructed from a multitude of materials, but this project will concentrate on ferrocement. Ferrocement is a form of thin cement mortar reinforced with layers of continuous and relatively small diameter mesh. It is usually made from a mortar of Portland cement and sand applied to steel reinforcement which is often provided in the form of small aperture wire mesh, typically 15*mm* x 15*mm*, see figure 1.3. Ferrocement is a low-level technology and is labour intensive, it is therefore ideally suited for water tanks in rural areas of developing countries. Ferrocement is well suited for thin wall structures such as water tanks because the distribution and dispersion of reinforcement provides good cracking resistance, higher tensile strength-to-height ratio, ductility, and impact resistance.

Fig 1.3 Ferrocement tank under construction

The materials, which are usually imported into the area from nearby towns, can be relatively expensive. The cost of this material often puts water storage tanks out of reach of many people in the rural sector. It is therefore important for the designer to investigate where and how construction materials can be reduced. To reduce material inputs it is important to know how large the forces are and where they act and also to know if it can be constructed from local available materials.

Section two of the project investigates how structural analysis can be carried out to establish the forces in the tank. To achieve this the theory of thin walled shells is used. Also, in section two, an Excel spreadsheet has been developed to allow the designer to study any range of tank shapes and construction materials.

A reduction in construction material through reduced wall and base thickness makes the tank more susceptible to additional problems which include shrinkage. Section three investigates the different types of shrinkage and how they effect cement based materials. It also looks into the degree to which shrinkage may be aggravated in less developed countries where the environment tends to be harsher. As well as dry weather, developing countries often suffer with a shortage of construction skills and poor quality raw materials. Different ways to reduce the additional stresses which shrinkage induces will be examined. This includes looking at tank design and education in the appropriate use of the raw materials.

Section four examines the mechanics and development of cracks in ferrocement and how they effect the durability of the tank. There is a limited amount of literature on the mechanical properties of ferrocement, so in section five a series of practical experiments are carried out to investigate some of these properties. It is essential to ascertain the tensile strength of the materials as well as looking into the effects of cracking caused by shrinkage, therefore three tests will be performed. They are,

- Tensile strength testing,
- Unrestrained (free) shrinkage,
- Restrained shrinkage.

Section six discusses the significance of the findings, section seven offers practical tips for tank construction and in section eight there are recommendations for future work.

2.0 STRUCTURAL ANALYSIS

The aim of this section is to study the theory of how stresses develop in structures and then to use the theory to write a spreadsheet to give ‘real’ results.

When analysing the loading on a water tank, it can be considered as a thin walled shell structure because the overall radius is large compared to the wall thickness, usually the ratio is greater than 10:1. These shell structures can be classified as shown in figures 2.0(a) and 2.0(b),

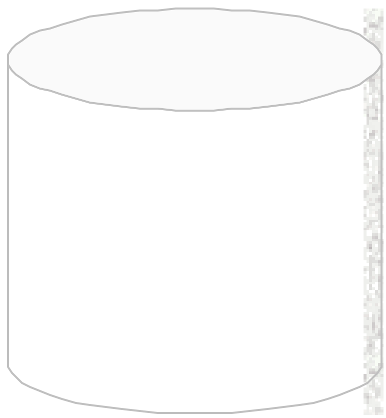


Fig 2.0(a) Singly curved shells

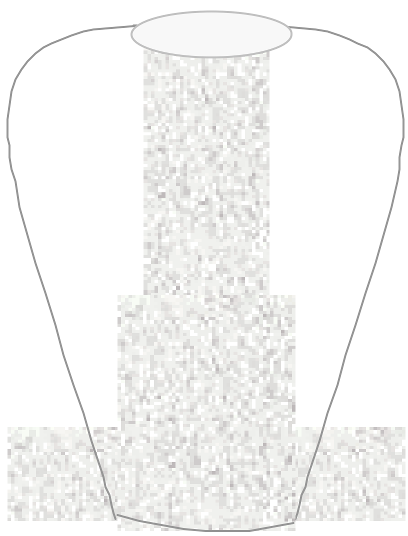


Fig 2.0(b) Doubly curved shells

The two main theories used when dealing with thin shells are,

- Membrane theory,
- Bending theory.

This project uses both membrane and bending theory. Initially the membrane theory will be used to calculate the stresses in the tank wall when there are no boundary conditions i.e. the tank walls are free to move, see figure 2.1(a).

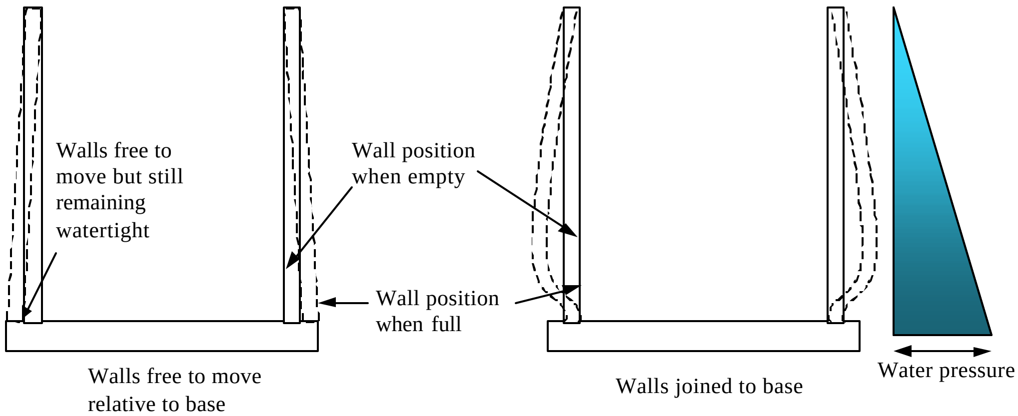


Fig. 2.1(a) Membrane theory

Fig. 2.1(b) Combined membrane and bending theory

In figure 2.1(b), the wall and base are monolithic i.e. the wall and base are continuous. As the wall is restrained bending stresses are set up in the wall. There now exists a complex combination of bending, shear and hoop stresses. Gray and Manning (1960) state that if the wall is not free to move at its base, then the loading caused by the water pressure is counteracted by a combination of hoop and cantilever resistance. It can be seen in figure 2.1(c) that as the base of the wall is restrained the hoop stress at the base is zero and the maximum hoop stress is now experienced higher up the wall. Bending theory is used to calculate this additional loading on the tank wall. The profile of the load distribution line is governed by the profile of the tank.

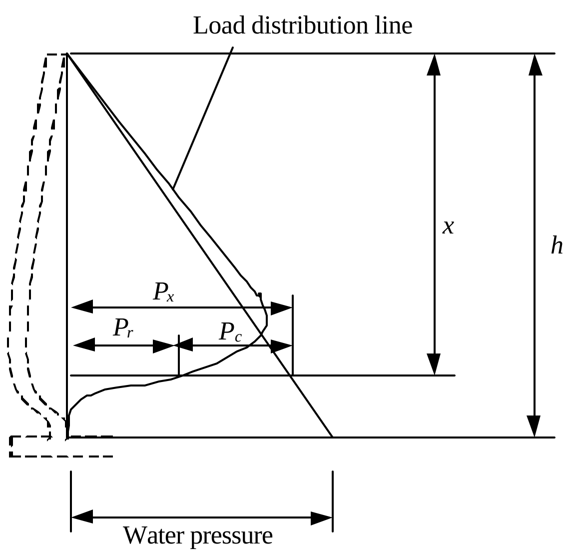


Fig 2.1(c) Typical load distribution for tank with a monolithic base (Gray and Manning 1960)

These theories are further simplified as only axisymmetric loading is considered. These loads are assumed to act at only the middle surface of the shell (wall), i.e. they pass through the centre of the structure. It is also assumed that the construction material is homogenous, isotropic, and linearly elastic, obeying Hooke's law.

2.1 Singly curved shells

Membrane theory

The governing equation for stress in singly curved shells, is;

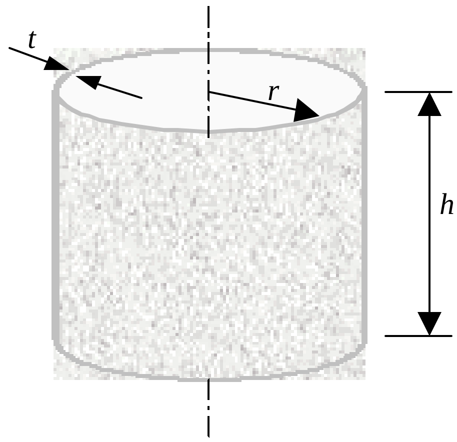


Fig. 2.2 Hoop force in singularly curved shell

$$N_q = p_r.r \tag{1}$$

To find the hoop stress (s), the hoop force is divided by the wall thickness (t) (Shigley 1983),

$$s_q = \frac{p_r.r}{t} \tag{2}$$

In this type of shell all the forces are resisted by the 'hoop' forces in one plane.

Tanks designed with singly curved shells, are probably the most common style of tank in current use as they are relatively easy to design and construct. Their main disadvantage is their weakness, they can only resist loads on one axis i.e. hoop forces.

Bending theory

As previously stated the membrane solution alone could not satisfy compatibility conditions at the boundaries. Bending theory can be further simplified by assuming that the base is solid and does not deflect. Using the spreadsheet it is possible to calculate the minimum depth which the base needs in order for it to be assumed to be solid and inflexible. The effects of edge loads have to be superimposed on the membrane solution. For both singly and doubly curved shells, these edge loads consist of shear forces (H_o) and bending moments (M_o).

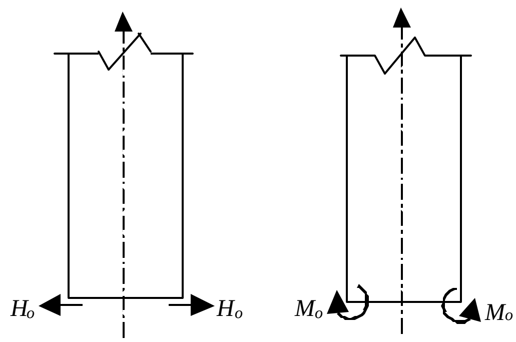


Fig. 2.3 Edge loads, H_o shear and M_o moments

These edge loads induce additional forces,

- M_x is the bending moment,
- Q_x is the shear force,
- $N_{\boldsymbol{q}}$ is the hoop force,
- w is the radial deflection.

these can be calculated from;

$$N_q = \boldsymbol{r} r \left[h - x - he^{-lx/r} \cos \frac{\boldsymbol{l}x}{r} + \left(\frac{r}{\boldsymbol{l}} - h \right) e^{-lx/r} \sin \frac{\boldsymbol{l}x}{r} \right] \qquad (3)$$

$$M_x = -\frac{\boldsymbol{r} rt}{\sqrt{12\left(1-\boldsymbol{n}^2\right)}} \left[\left(\frac{r}{\boldsymbol{l}} - h \right) e^{-lx/r} \cos \frac{\boldsymbol{l}x}{r} + he^{-lx/r} \sin \frac{\boldsymbol{l}x}{r} \right] \qquad (4)$$

$$Q_x = \frac{\boldsymbol{r}\, t\, \boldsymbol{l}}{\sqrt{12\left(1-\boldsymbol{n}^2\right)}} \left[\left(\frac{r}{\boldsymbol{l}} - 2h \right) e^{-lx/r} \cos \frac{\boldsymbol{l}x}{a} + \frac{r}{\boldsymbol{l}} e^{-lx/r} \sin \frac{\boldsymbol{l}x}{r} \right] \qquad (5)$$

$$w = \frac{\boldsymbol{r}\, r^2}{Et}\left[h - x - he^{-lx/r}\cos\frac{\boldsymbol{l}x}{r} + \left(\frac{r}{\boldsymbol{l}} - h\right)e^{-lx/r}\sin\frac{\boldsymbol{l}x}{r}\right] \qquad (6)$$

where;

$$l^4 = 3\left(1 - n^2\right)\left(\frac{r}{t}\right)^2$$

The effects of the additional loading forces are localised around the shell wall/base intersection. All the equations contain a multiplication term $e^{-l x/r}$, which means the effect will decay exponentially with distance moved away from the base. For a full derivation of the formulae see Flügge (1967).

2.2 Doubly curved shells

The structural analysis for doubly curved shells is more complicated than that of the singly curved shell. These shells have curvature in two planes, figure 2.4. This allows them to resist loads by generation of forces in the two planes. They are generally more efficient than singly curved shells. The two main forces are,

- the meridional force (N_f),
- the hoop force (N_q).

And the two main radii of curvature are,

- radius of curvature of the meridian (r_1),
- radius of curvature of the second principal curve (r_2).

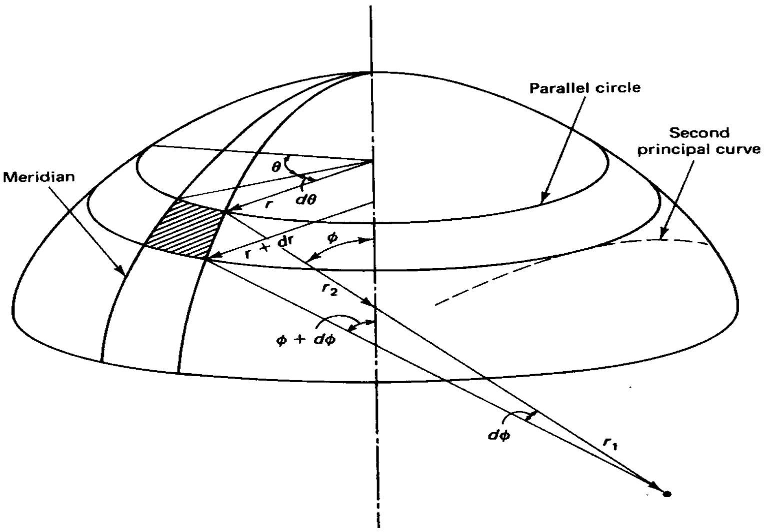


Fig. 2.4 Doubly curved shell, showing the parallel circle, principal curves, and shell element (Kelkar and *Sewell1987*)

The main equation governing the forces in a doubly curved shell is;

$$\frac{N_f}{r_1} + \frac{N_q}{r_2} = p_r \qquad (7)$$

Where p_r is the pressure at a particular point.

Equation (7) is rearranged to find the hoop force N_q;

$$N_q = r_1\, p_r - \frac{r_1}{r_2} N_f \qquad (8)$$

After some mathematical manipulation the general solution for the meridional force N_f can be found as follows;

$$N_f = \frac{1}{r_2 \sin^2 f}\left[\int \left(p_r \cos f - p_f \sin f\right) r_1 r_2 \sin f \, d f + k\right] \qquad (9)$$

where k is a constant of integration to be obtained from an appropriate boundary condition.

The next step is to find the geometric parameters for the shell. The shell's profile can be described as a curve, where $y = f(x)$. The principal radius of curvature of the surface in the meridional plane, r_1, and the second principal radius of curvature, r_2, are given by;

$$r_1 = \frac{\left\{1+\left[f^{'}(x)\right]^2\right\}^{3/2}}{f''(x)} \tag{10}$$

$$r_2 = \frac{x}{\sin f} \tag{11}$$

where $f'(x)$ and $f''(x)$ denote first and second derivatives of $f(x)$ with respect to x. By sketching a right-angled triangle it can be seen that;

$$r_2 = \frac{x\left\{1+\left[f^{'}(x)\right]^2\right\}^{1/2}}{f'(x)} \tag{12}$$

and angle f is;

$$\sin f = \frac{f'(x)}{\left\{1+\left[f^{'}(x)\right]^2\right\}^{1/2}} \tag{13}$$

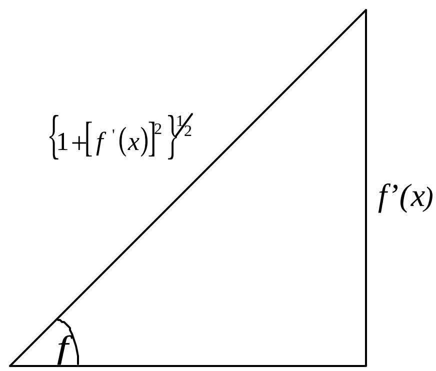


Fig. 2.5 Trigonometric interpretation of the equations

To calculate the stress resultants, figure 2.6 shows a shell of revolution generated by a rotation of a curve $y = f(x)$ about the y axis, filled with liquid of density r, to a depth $h = f\left(x_H\right)$. The pressure p_r acting in direction normal to the shell at height $g = f(x)$ above the bottom of the tank is given by;

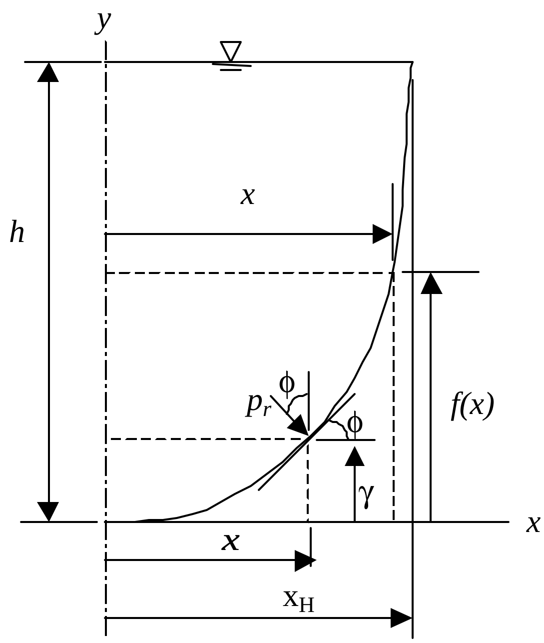


Fig 2.6 Liquid shell of revolution (Zingoni 1997)

$$p_r = \rho g(h-\gamma) = \rho g\{f(x_H) - f(\xi)\} \tag{14}$$

The net vertical resultant *W(x)* of the pressure acting on the shell whose edges are defined by *(x,(f(x))* is obtained by integration, as follows;

$$W(x) = 2\pi\rho g\left\{\frac{x^2}{2} f(x_H) - \int_0^x \xi f(\xi)\,d\xi\right\} \tag{15}$$

The meridional stress N_ϕ can be found by resolving forces in the vertical direction;

$$2\pi x N_\phi \sin\phi = W(x) \tag{16}$$

Form (16) and using the expression for *W(x)* from (15), and the expression for $\sin\phi$ given in (13), N_ϕ is found as follows;

$$N_\phi = \rho g \frac{\{1+[f'(x)]^2\}^{1/2}}{x f'(x)}\left\{\frac{x^2}{2} f(x_H) - \int_0^x \xi f(\xi)\,d\xi\right\} \tag{17}$$

As seen previously the hoop stress N_θ is given by (2).

The total volume *V* of the tank is obtained from (15), by dividing the total weight of the liquid, this is equal to $(W(x_H))$, by the unit weight ρg, thus;

$$V = 2\pi\left\{\frac{x^2}{2} f(x_H) - \int_0^{x_H} \xi f(\xi)\,d\xi\right\} \tag{18}$$

The total surface area S of the (constant thickness) tank is obtained by rotating the curve $y = f(x)$ about the y axis[2].

$$S = 2\pi \int_0^{x_H} \left\{1 + \left[f'(x)\right]^2\right\}^{1/2} dx \qquad (19)$$

To find out how much the tank walls deflect under load the strain is required. Since both N_θ and N_ϕ have already been found using membrane theory, the following relationship can be used to find the strains;

$$\varepsilon_\phi = \frac{1}{Et}\left(N_\phi - \nu N_\theta\right) \quad \& \quad \varepsilon_\theta = \frac{1}{Et}\left(N_\theta - \nu N_\phi\right) \qquad (20)\ \&\ (21)$$

Generally only the horizontal deflection Δ_H is required (Kelkar and Sewell 1987), i.e. the increase in size around the diameter of the tank;

$$\Delta_H = r\varepsilon_\theta = \frac{r}{Et}\left(N_\theta - \nu N_\phi\right) \qquad (22)$$

In the following section the formulae will be have been substituted into two Excel spreadsheets, one for singly curved shells and the other for doubly curved shells.

Due to the complexity of bending theory for shells of general revolution, it will not be incorporated. The solution given by the membrane theory gives good results for many practical problems, (Kelkar and Sewell 1987).

There are a number of good books that cover the full derivation of all these formulae, such as, ‘Shell Structures’ by Zingoni (1997) or ‘Analysis and Design of Shell Structures’ by Kelkar and Sewell (1987).

2.3 The Excel spreadsheets

The most common shape is the cylindrical tank, this style of tank resists all its applied loads in one plane. These loads vary linearly from a minimum at the top to a maximum at the base. Tanks are usually constructed with a constant wall thickness, which has to resist the maximum load at the bottom of the tank. This means that above this point the material is under utilised, and therefore wasted. To overcome this problem the tank shape may be varied, it could be made conical or tank thickness could be varied from top to bottom.

It is a well known fact that that certain shapes resist loads better than others, for example, if constructed of similar materials, a domed roof is stronger than a flat roof. Therefore it is important to investigate stresses in more unconventional tank shapes, such as the 'pumpkin' tank shown in figure 2.7(a) or the jar shaped in figure 2.7(b).

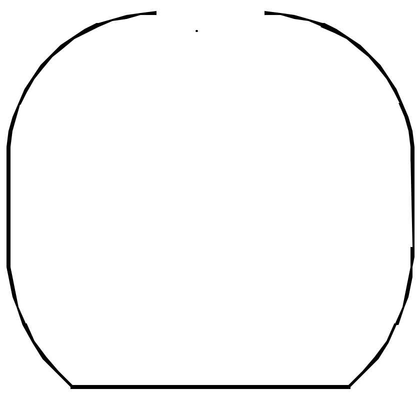


Fig 2.7(a) Pumpkin tank

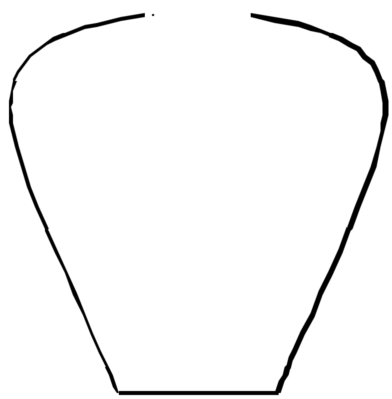


Fig. 2.7(b) 'Thai' Jar tank

The Excel spreadsheet lets the designer experiment with various shapes of tank and calculate where the maximum forces occur. These forces include shear, hoop and meridional force intestines, and bending moments, as well as tank wall deflection. The spreadsheet used to analyse the forces in the singly curved shells takes into account additional forces caused by the tank wall/base intersection (bending theory). The spreadsheet can be used to help with material optimisation as it can work out the ratio of construction material to water storage capacity.

The spreadsheet used to calculate forces in singly curved shells has already been used by the Development Technology Unit to aid in the design of a rammed earth water tank.

There are two spreadsheets, the first one analyses singly curved tanks and the second doubly curved tanks.

Singly Curved tanks

To start the operation a number of INPUTS are entered including material yield tensile strength, tank volume, base and lid thickness; and a suitable safety factor, see figure 2.8.

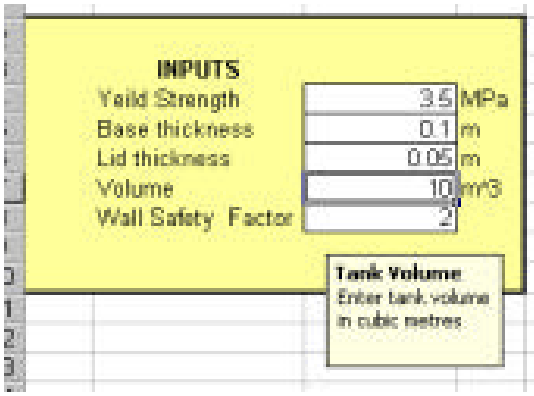


Fig. 2.8 Input parameters

The yield strength of the material can be found from literature but if unconventional materials are being used, i.e. ferrocement, it may be necessary to carry out some form of mechanical strength testing. The material yield strength used in this example was found using experimental methods (see section 5.0). The results given by the numerical model are only as accurate as the material property data therefore it is vital at this stage to enter data that is as close as possible to realistic figures.

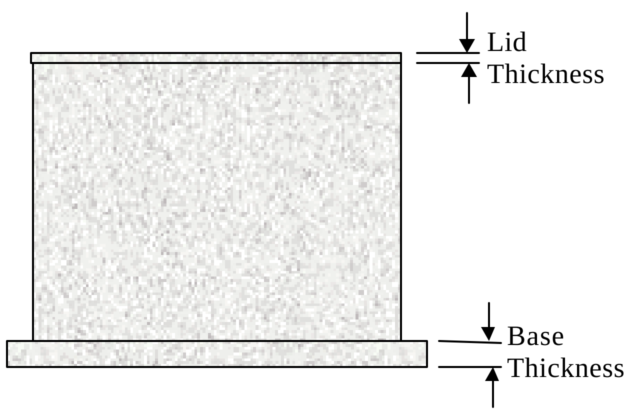


Fig. 2.9 Tank base and top geometry

The thickness of the base and lid (see figure 2.9) are used when calculating the amount of tank construction material needed to make the tank. The base thickness is also used in calculating the base rigidity (to be discussed later). The safety factor can be chosen by the designer, this may depend on a number of factors. Examples include uncertainty on the quality of raw materials, climatic and geological conditions, as well as unusual loading such as wind.

For this example, the tank volume is $10m^3$ with the data given in figure 2.8, a range of tank sizes and amount of raw material required for construction is calculated, see figure 2.10.

Diameter	height	Wall thickness		Material	
0	#DIV/0!	#DIV/0!	m	#DIV/0!	m^3
0.2	318.31	0.178	m	67.53	m^3
0.4	79.58	0.089	m	10.93	m^3
0.6	35.37	0.059	m	4.40	m^3
0.8	19.89	0.045	m	2.43	m^3
1	12.73	0.036	m	1.60	m^3
1.2	8.84	0.030	m	1.19	m^3
1.4	6.50	0.025	m	0.97	m^3
1.6	4.97	0.022	m	0.87	m^3
1.8	3.93	0.020	m	0.83	m^3
2	3.18	0.018	m	0.83	m^3
2.2	2.63	0.016	m	0.87	m^3
2.4	2.21	0.015	m	0.93	m^3
2.6	1.88	0.014	m	1.01	m^3
2.8	1.62	0.013	m	1.11	m^3
3	1.41	0.012	m	1.22	m^3
3.2	1.24	0.011	m	1.36	m^3
3.4	1.10	0.010	m	1.49	m^3
3.6	0.98	0.010	m	1.64	m^3
3.8	0.88	0.009	m	1.80	m^3
4	0.80	0.009	m	1.97	m^3

Paste suitable figues into this table from above

Diameter	height	Wall thickness		Material	
2.8	1.62	0.01275	m	1.107	m^3

Fig. 2.10 Range of tank sizes

The designer may choose any one of the tanks. The data for that particular tank including, the required diameter, height and wall thickness is entered into the next phase of the spreadsheet, see figure 2.11.

1		Constants			Constants		
2	total vol	Base			Wall		
3	#DIV/0!	E base matrix	35000	MPa	E wall matrix	35000	MPa
4	25.80806	E base fibre	210000	MPa	E wall fibre	210000	MPa
5	4.977223	volume fraction	0.06		volume fraction	0.01	
6	2.123297	E base total	45500	MPa	E wall total	36750	MPa
7	1.221713						
8	0.844287	PR base matrix	0.1		PR wall matrix	0.1	
9	0.671442	PR base fibre	0.3		PR wall fibre	0.3	
10	0.598376	volume fraction	0.06		volume fraction	0.02	
11	0.582342	PR base total	0.112		PR wall total	0.104	
12	0.603207						
13	0.65047				thickness	0.01275	m
14	0.71821				diameter	2.80	m
15	0.802881				height	1.62	m
16	0.902256				volume	10.00	m^3
17	1.014674				Material	1.107	m^3
18	1.139749						

Fig. 2.11 Inputs used to calculate tank wall and base rigidity

For the shell theory discussed in section 2.1 to be valid, it is assumed that the tank base is totally solid with no deflection. To calculate the base and wall flexibility the Youngs modulus (*E*), Poisson's ratio (*PR*) and volume fraction are required. Ferrocement is a composite of two materials, in the spreadsheet the cement mortar is called the matrix and reinforcement is called the fibre. The volume fraction is the volume of reinforcement per unit volume of ferrocement.

The spreadsheet is also capable of finding the minimum base thickness that is required to make a totally rigid base. It can be seen in figure 2.12 that for this particular tank, it can be seen that the base is totally rigid if its thickness is greater than 15*cm*, making the base any thicker than this will just be a waste of material.

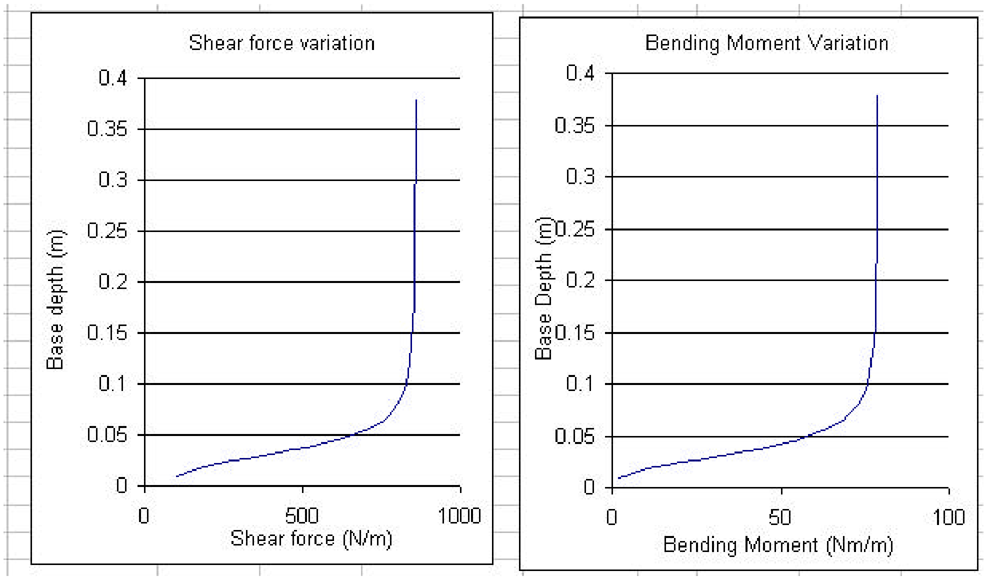


Fig. 2.12 Variation in shear force and bending moment

The spreadsheet calculates the maximum stresses, deflection and bending moment and at what point they are greatest, see figure 2.13. The ratio of construction material to water storage capacity is another deliverable. A full print out of the spreadsheet can be found in Appendix V

3.6E-10 17

OUTPUT DATA

Tank with fixed base

				Distance from base	
Max deflection	0.055	mm	at	0.24	m
Max Shear Stress	0.63	MPa	at	0.24	m
Max hoop stres	1.50	MPa	at	0.24	m
Max BM	77.34	Nm/m	at	base	
Max tensile Stress	2.86	MPa	at	base	
Safety factor at base	0.23				

Tank with floating base

Max hoop Stress	1.75	MPa	at	base	
Max deflection	0.064	mm	at	base	

Tank size		**Raw Materials**	
1.62	m	1.11	m3
2.80	m		
10.00	m3		

ater storeage to amount of raw material 9.04

Fig. 2.13 Output data

Figures 2.14(a-b) show the variation in deflection, hoop stress, bending moment and shear force up the tank wall, all acting uniformly over the stressed area.

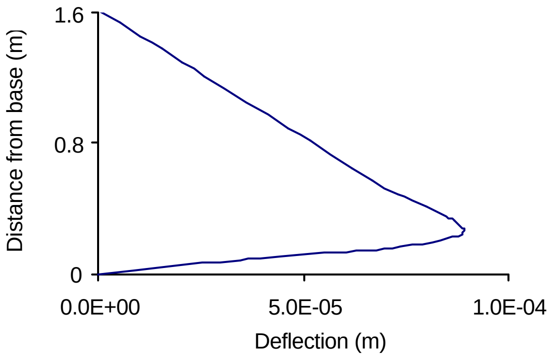


Figure 2.14(a) Variation in Deflection

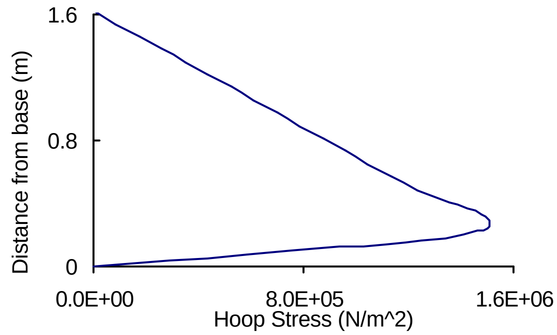


Figure 2.14(b) Variation in hoop stress

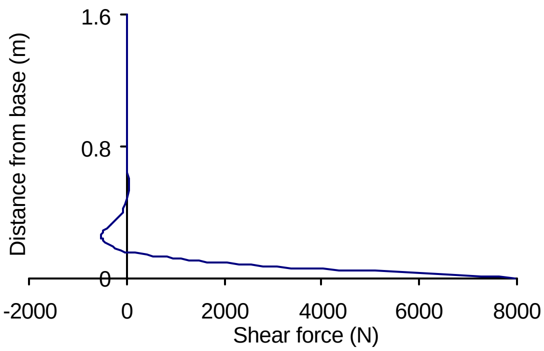


Figure 2.14(c) Variation in shear force

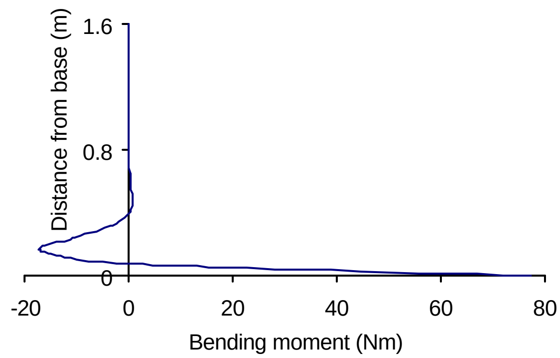


Figure 2.14(d) Variation in bending moment

The spreadsheet also allows the user to calculate the hoop stress in the tank wall if the wall is free to move, i.e. not connected to the base as shown in figure 2.1(a).

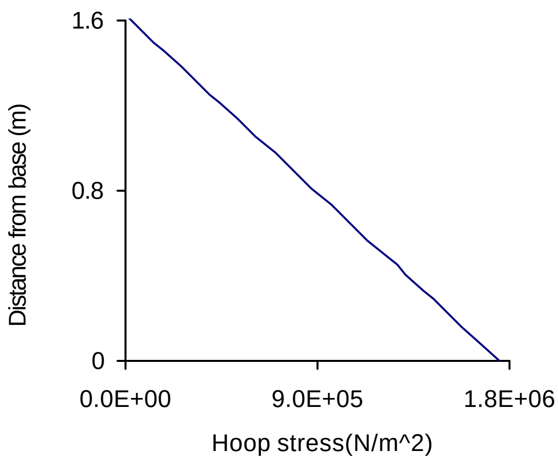


Fig. 2.15 Variation in hoop stress with floating base

Doubly curved shells

This spreadsheet is used to calculate the forces in the doubly curved shells. It is not as user friendly as the spreadsheet used for the singly curved format. As before, the material characteristics including: Young's modulus, Poisson's ratio, volume fraction and wall thickness, are entered.

For the tank profile shown in figure 2.17, the following data has been entered.

1						
2			Input			
3						
4	E wall matrix	35	Gpa	PR wall matrix	0.1	
5	E wall fibre	210	Gpa	PR wall fibre	0.3	
6	fraction volume	0.01		fraction vol	0.01	
7	E wall	36.75	Gpa	PR wall	0.102	
8						
9	thickness	100	mm			
10	tensile strength	3.4	MPa			
11						
12						
13						

Fig. 2.16 Inputs for doubly curved tank

The tank profile is entered into the Excel format using x and y co-ordinates. Using the *trendline* function the formula for the curve can be found and hence the first and second derivatives. For this example the following profile will be used:

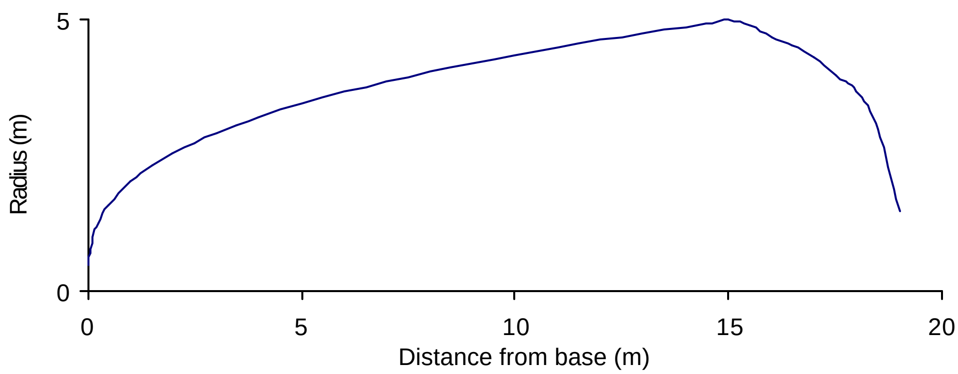
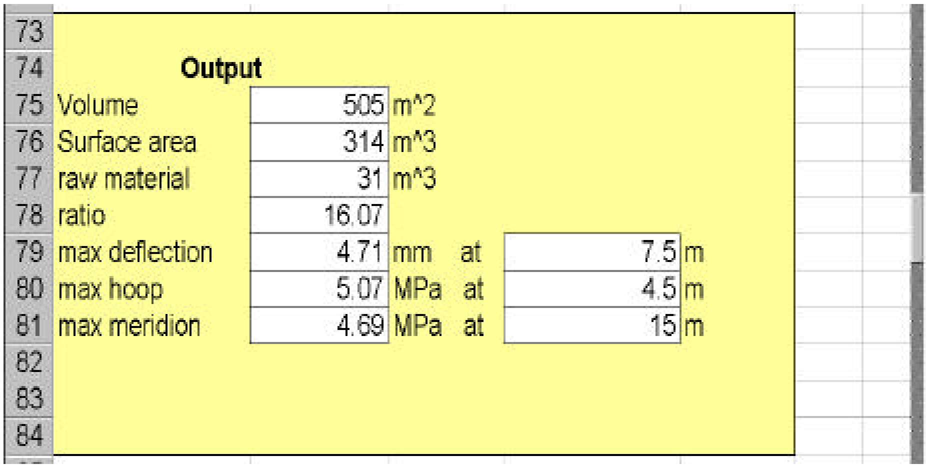


Fig. 2.17 Tank profile

The spreadsheet calculates the meridional and hoop stress, as well as the volume, surface area and ratio of construction material to volume. A full print out of the spreadsheet can be found in Appendix VI.



73						
74	**Output**					
75	Volume	505	m^2			
76	Surface area	314	m^3			
77	raw material	31	m^3			
78	ratio	16.07				
79	max deflection	4.71	mm	at	7.5	m
80	max hoop	5.07	MPa	at	4.5	m
81	max meridion	4.69	MPa	at	15	m
82						
83						
84						

Fig 2.18 Results for doubly curved tank

The spreadsheet calculates the maximum deflection, hoop stress, and meridional stress and at which point from the base of the tank they are the greatest. It also calculates the volume, surface area, the amount of raw materials required in construction and the ratio of raw materials to water storage capacity.

The variation in meridional and hoop stress is displayed in figure 2.19.

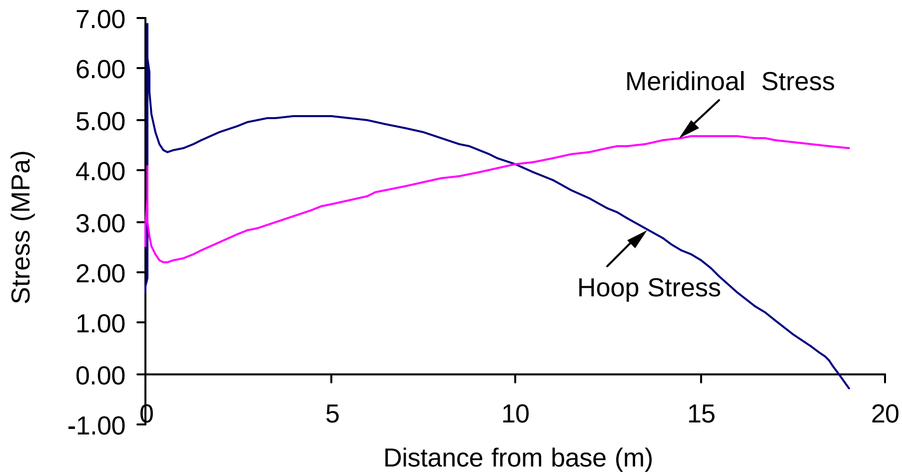


Fig 2.19 Variation in membrane stress

Variations in stress near the base of the tank can be ignored, this is due to the breakdown of the membrane hypothesis. The tank wall deflection, i.e. the increase in tank radius, can be seen in figure 2.20.

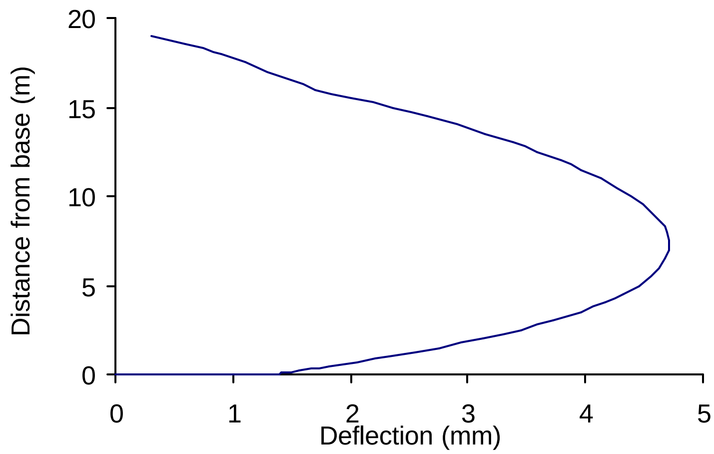


Fig. 2.19 Variation in Deflection

Discussion

The spreadsheets are to be used as tools to aid in the designing and construction of water storage tanks. They give the designer an opportunity to experiment with many shapes of tank, as well as material properties. The designer can enter any required tank specification, the spreadsheet is then run and the results are given. Changes to the tank specification can be made immediately. This process allows iterations to be made quickly and easily. The program results can be plotted out, this gives the designer a graphical representation of the forces. The visualisation of these forces will help with the design process.

From the results for the singly curved tank, figures 2.14a-c, it can be seen that the greatest load is near the base of the tank. In figure 2.15, for the tank whose wall is not attached to the base, the maximum force is at the bottom of the tank. For the doubly curved tank, see figure 2.19, the forces are more uniformly distributed over the whole tank wall. These results show the designer that if they require a tank of uniform wall thickness and hence a uniform wall strength, the doubly curved tank is the most suitable, as the forces are distributed evenly over the whole tank. It can also be seen that if a uniform wall is used to construct a singly curved tank, the construction material used in the upper part of the tank is not fully utilised and therefore wasted. Other factors have to be taken into consideration such as construction difficulties, a greater degree of skill is required to construct a doubly curved tank compared to a singly curved tank.

It is assumed that the designer is using a homogenous construction material which can withstand forces equally well in both a vertical and horizontal direction, a characteristic of ferrocement. The results and theory show that for singly curved tanks the majority of force is on the horizontal plane but for the doubly curved tanks, the force is on both vertical and horizontal planes. This means that the construction material is used to its full potential for the doubly curved tank.

It can be seen from figure 2.15, which if the wall is free to move relative to the base, the maximum stress and therefore wall deflection, is at the bottom of the tank wall. When the bottom of the wall is restricted from moving, as in figure 2.14(a-c), the maximum hoop force and wall deflection are a small distance from the base. This distance depends on the size of the tank and the characteristics of the construction materials used. Using these results the designer can decide where to position any extra reinforcing.

To overcome the problem of additional forces that are generated by the base wall interface it is possible to construct a tank so that the wall is free to expand. The Development Technology Unit is currently examining this construction technique at Warwick University. It is achieved by positioning two polythene strips between the wall and base, see figure 2.21. This reduces the sliding friction between the two surfaces, and therefore the wall is relatively free to expand, eliminating the additional forces.

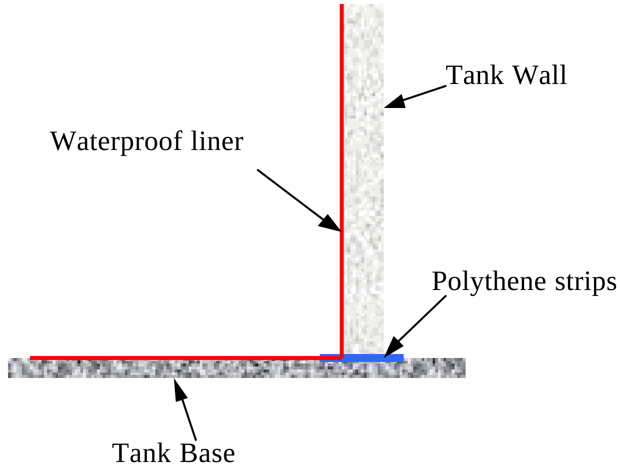


Fig 2.21 Experimental tank with sliding wall

The designer is also free to experiment with safety factors, these will vary according to raw material, climatic and ground conditions.

3.0 SHRINKAGE

This section of the project will concentrate of the effects shrinkage has on water tanks constructed from ferrocement. The mechanics of shrinkage in cement is a complicated subject and will not be fully explored in this report. Further information on the mechanics of shrinkage can be found in a number of books, including 'Proprieties of Concrete' by Neville (1995) and 'Concrete Technology' by White (1991).

3.1 Classes of Shrinkage

Shrinkage can be divided into three main categories. They are,

- Plastic,
- Autogenous,
- Drying.

Plastic Shrinkage

This takes place when the concrete is still in the plastic state. The volume change is small, in the order of 1% of the absolute volume of dry cement (American Concrete Institute). Although the mechanics of plastic shrinkage are not fully understood, it is known that it can lead to surface cracking. The amount of shrinkage is related to the rate of evaporation. One practical conclusion is that thin sections, such as tank walls, should not be cast under hot and dry conditions.

Autogenous Shrinkage

Autogenous shrinkage results in volume change without the loss of moisture. The magnitude of the strain induced is in the order of $40x10^{-6}$ at the age of one month and $100x10^{-6}$ after 5 years (Davis 1940). The contraction is relatively small and of significance only in large mass structures. This project only deals with thin walled structures therefore this type of shrinkage can be ignored as it very low compared to drying shrinkage.

Drying Shrinkage

As the cement sets it looses moisture, this loss of moisture causes shrinkage. This type of shrinkage has the greatest effect on the water tank. Shrinkage in the order of 2,000 x 10^{-6} has been observed (Singh 1989). The finer the grain size in the mix the greater the shrinkage, cement for example will have a greater degree of shrinkage compared to concrete. The rate at which the shrinkage occurs depends on the speed of water loss, therefore in hot, dry and windy climates rates of shrinkage will be high.

3.2 Differential shrinkage

The problems that shrinkage creates are worsened by the effects of differential shrinkage. Differential shrinkage occurs if water loss from one area is greater than another, this is especially so in the early stages of curing. This type of shrinkage has a tendency to induce internal stresses in the structure and can lead to cracking. Differential shrinkage occurs in all cement-based products but to what extent depends greatly on the size and shape. It can be seen from figure 3.0 that the thinner the section the greater the shrinkage, this is because the moisture loss is faster. Therefore structures that have large surface areas compared to volume, a characteristic of water tanks, will have a tendency to display a high degree of shrinkage. For practical purposes shrinkage cannot be considered as purely an inherent property of the cement without reference to the geometry of the structure.

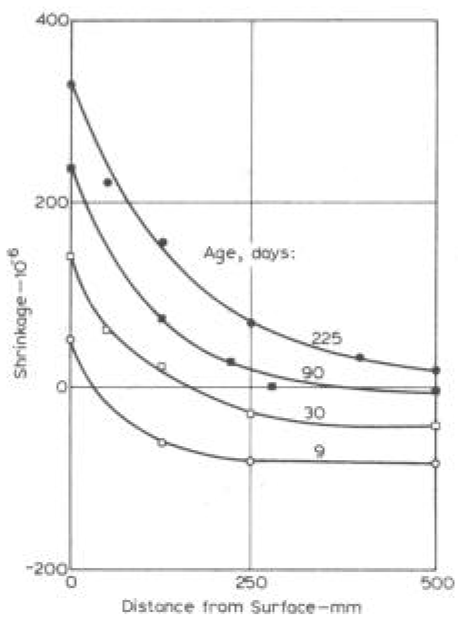


Fig.3.0 Effect of dimensions on shrinkage (Neville 1995)

There are a number of ways to help reduce the problems caused by differential shrinkage when building tanks in a hot climate;

- The wall thickness should be kept constant throughout the tank this will reduce the build up of internal stress.
- Ideally, the tank should be constructed in a shaded area, this will reduce water loss.
- The tank should be constructed in the coolest part of the day, ideally in the morning so the construction materials have had the opportunity to cool down overnight.
- The tank should not be constructed so it is half shaded, again this will cause internal stress to be built up.
- The application of cement mortar should be as continuous as possible. Rendering wet mortar onto dry should be avoided, as stresses will build up between the layers.
- A good curing regime is required, such as the use of wet hessian and plastic covers.

3.3 Restrained Shrinkage

The effects of shrinkage are only really a problem if the stresses they induce are not able to release themselves. If strains due to shrinkage cannot develop freely, large internal stresses are developed. If these tensile stresses are greater than the tensile strength of the material, cracking and sometimes total failure may occur. Shrinkage in any form weakens the structure as it acts as a preload this may lead to unexpected failure even under low loads. The effect of restrained shrinkage is shown in figure 3.1, the ends of the cement mortar specimen are held and as the cement shrinks, cracks start to form, usually at the weakest point.

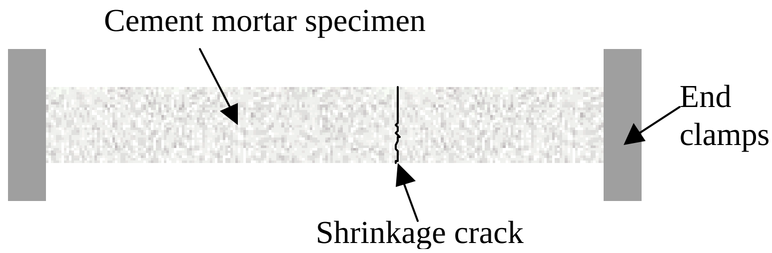


Fig. 3.1 Cracking caused by shrinkage

In the case of the water tank in figure 3.2, the bottom of the structure is usually cast earlier than the walls, consequently the shrinkage strains at the bottom ε_{base} and the walls ε_{wall} may be very different. Since the base of the wall cannot move freely additional bending moments and forces develop at the base/wall interface.

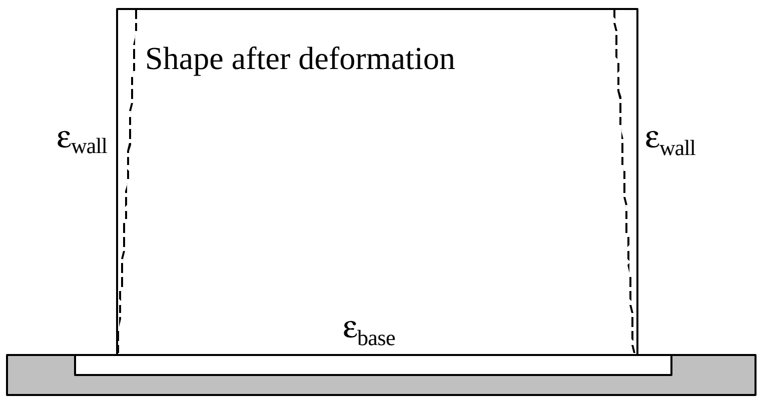


Fig. 3.2 Effect of shrinkage on the tank wall

3.4 Further factors that influence shrinkage

There are number of other factors that effect the degree and rate of shrinkage. They include,

- Water/cement ratio,
- Curing and storage conditions,
- Aggregate.

Water/cement ratio

In general, the higher the water/cement ratio the higher the degree of shrinkage. It has been demonstrated that shrinkage is directly proportional to the water/cement ratio between values of about 0.2 to 0.6 (Neville 1995). At higher water/cement ratios, the additional water is removed upon drying without resulting in shrinkage. A minimum water/cement ratio of 0.4 is required for the cement to reach its full strength, as the water/cement ratio increases the strength is adversely effected. In practice, this value of 0.4 is rarely achieved because of difficulties in working of with such a dry mix.

Curing and storage conditions

Curing is the name given to the method used for promoting the hydration of cement. It consists of the control of temperature and of moisture movement from and into the cement mortar. The main object of curing is to keep the cement saturated or as near to saturation as possible. Prolonged moist curing delays the advent of shrinkage.

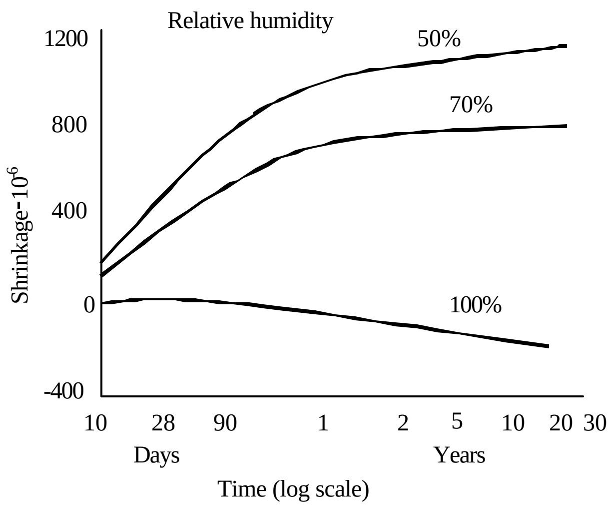


Fig. 3.3 Relationship between shrinkage and humidity (American Concrete Institute)

The ambient temperature has an important effect on the degree of shrinkage. Working with cement in hot weather, greater than 30^0C (Neville 1995), can cause problems. Because of the high evaporation rate there is a loss of workability and increased shrinkage. The increased shrinkage can cause high internal stresses that may lead to cracking. It is therefore advantageous to cast cement in cooler conditions, less than 20^0C, as the rate of water evaporation is less. This will reduce shrinkage rates as well as increasing strength.

The relative humidity of the surroundings also effects the degree of shrinkage. It can be seen in figure 3.3 that the drier the environment, the larger the degree of shrinkage. When the cement mortar is stored in 100% humidity (in water) it swells. Once removed from the water it will start to shrink. The above data is based on a mortar mix of 1:5, with a water cement ratio 0.59. The results are taken after 28 days of wet curing.

Aggregate

The size and grading of aggregate influence the magnitude of shrinkage. In general, coarser aggregate leads to lower rates of shrinkage because leaner mixes can be made. Ferrocement contains very fine aggregate in the form of sand so to make it workable relatively high water/cement ratios are required, typically in the range of 0.5 to 0.6. This makes ferrocement tanks very susceptible to shrinkage.

4.0 CRACKING

The development of cracks in any form of water retaining structure can have serious repercussions. This section looks at the how cracks develop in ferrocement and how their propagation can be reduced.

Cracks in ferrocement are mainly caused by:

- thermal and moisture movement incompatibilities between the phases of cement paste, sand and reinforcement,
- fatigue caused by repeated loading,
- induced stress caused by shrinkage.

This report will only deal with the later, stress cracks caused by shrinkage. Concerns regarding cracks in ferrocement, are expressed as follows:

- Aesthetic, from the aesthetic point of view a 0.3mm crack seen at a close distance is quite concerning, but viewed from a distance of 2 to 3 metres it is quite acceptable.
- Leakage, Nedwall and Swamy (1994) recommend an upper limit of 0.05mm for water-retaining structures.
- Durability, from the materials durability aspect, the stronger the construction material, the less likely it is to crack and the longer the tank life (assuming there is no mechanical damage).

In the case of the water tank one of the main concerns is leakage. This has two effects, loss of water and corrosion of the reinforcing material. Corrosion of the reinforcing mesh leads to spalling. Finely distributed reinforcement (wire mesh) combined with large mortar cover increases the resistance to corrosion. The permeability of the ferrocement depends greatly on the care taken in applying the mortar. The recommended minimum cover is only 2mm (Skinner 1995), assuming if the cement is applied well and cured in an appropriate manner. Because of variations of material and construction quality in less developed countries, Gould (1999) recommends a cover of 30mm, giving a wall thickness of 60mm. There are number of practical problems when it comes to applying very thin layers of cement mortar, the main one being the difficulties in ensuring an even cover over the wire mesh and that all the mesh is covered to the minimum thickness.

Development of cracks

Cracks with an applied tensile force in ferrocement develop in three stages. It can be seen in figure 4.0(a) that in the first stage of development both materials in the composite respond elastically, if the load is removed they will return to their unloaded state.

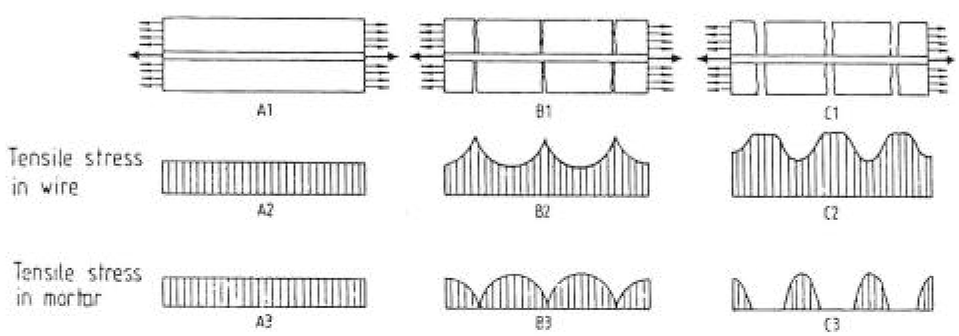


Fig 4.0(a) Stage one Fig. 4.0(b) Stage two Fig 4.0(c) Stage three

The ferrocement enters its second stage when micro cracks start to appear. During this stage, shown in figure 4.0(b), the number of cracks keeps increasing with the load though the width. Figure 4.0(c) show that the increase is only marginal. In the third stage, the number of cracks remain almost constant but their width increases with load. The mortar in the ferrocement no longer has any effect on its strength. Assuming that the steel reinforcement has not passed its yield point, if the load is removed the sample will contract leaving open cracks in the mortar.

The cracks develop in this way because the cement mortar is not as ductile as the steel reinforcement. It can seen that as the cracks develop, the load on the reinforcing increases and finally when the cracks are fully developed (stage three) the reinforcing carries all the load. The practical result of this is that, if quality of the mortar is not known and catastrophic failure is to be avoided, then the tank needs to be designed so that the reinforcing is capable of carrying the entire water load.

5.0 MATERIAL TESTING

This project looks into the practical effects of shrinkage on ferrocement water tanks. Shrinkage plays an important role in tank design as it induces forces that can cause tank failure. It is important to know the magnitude of these forces, and ways to reduce their detrimental effects need to be investigated. There is a limited amount of literature on the mechanical properties of ferrocement therefore three tests have been devised to help investigate these properties and their effect on shrinkage. They are:

- Tensile strength,
- Unrestricted shrinkage,
- Restricted shrinkage.

It is necessary to know the tensile strength of the material, as it is needed to produce realistic results in the spreadsheets. It is used in conjunction with the unrestricted shrinkage findings to calculate the theoretical crack propagation in the cement sample. The tensile strength is found by applying a direct tensile load, discussed further in section 6.2.

The unrestrained shrinkage test is needed to investigate the magnitude shrinkage. If the specimen's movement is not restricted, these results are used to help calculate the theoretical crack development of each sample. To measure the unrestrained shrinkage, blocks of each specimen are to be cast and then are allowed to set over a 28-day period, daily readings are taken and the shrinkage is calculated, discussed further in section 6.3.

The restrained shrinkage test is used to investigate the crack development in four different specimens. The test is aimed at mimicking the effect of cracking in the tank wall caused by the restriction in movement due to the base/wall junction (see figure 2.1). Two tests are used, the first looks at cracking in a reinforced specimen block and the second investigates cracking in a ring sample, both tests are discussed further in section 6.4.

There are numerous types of reinforcing materials used to enhance the mechanical properties of cement mortar. This project is aimed at tank construction in developing countries therefore only reinforcing that is available in these countries is to be used. Three types of reinforcing material that are commonly available are:

- Chicken wire (volume fraction 1%),
- Square mesh (volume fraction 2%),
- Polypropylene fibres (volume fraction 1.5%).

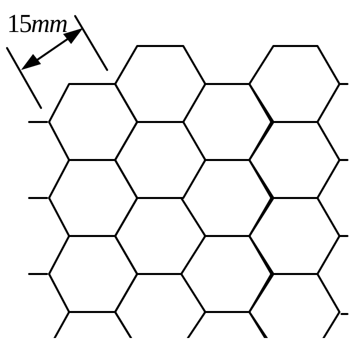
As little is known about the effects of reinforcing is the experimental volume fraction for each reinforcing material (shown above) are to be used.

To ensure uniformity of results, it is important to use an appropriate research methodology.

5.1 Procedure

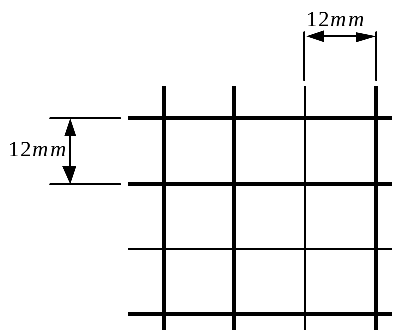
Materials

The chicken wire (figure 5.0(a)) and square mesh (figure 5.0(b)) are manufactured from galvanised mild steel. The polypropylene fibres are 'home-made' (figure 5.0(c)) and are approximately 10mm stands cut from a length of rope. The material's mechanical properties are given in Appendix I.



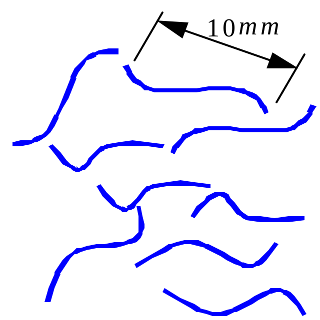
Mesh diameter = 0.5mm

Fig 5.0(a) Chicken wire



mesh diameter = 1.0mm

Fig 5.0(b) Square mesh



fibre diameter ≅ 0.5mm

Fig 5.0(c) Polypropylene fibres

The cement used in all the mixes is Ordinary Portland Cement (OPC). This is mixed with standard building sand to a ratio of three parts sand to one part cement by weight, with a water cement ratio 0.6.

Mixing and making

When cement based products are made in developing countries they are usually hand mixed. Due to the problems caused by the high ambient temperature, such as rapid setting due to water loss and in turn reduced workability there is a tendency to make a relatively wet mix. There is also sometimes a lack of knowledge about the correct consistency of cement. As stated previously, for the optimum strength of the cement, a water cement ratio of 0.4 is required. To add authenticity to the results all the samples are made using a relatively high water cement ratio of 0.6 and they are all mixed by hand. With increased mixing time, the strength of the cement paste slightly increases and variations in strength decrease therefore for the sake of uniformity all the dry materials are mixed for approximately three minutes and then wet mixed for a further three minutes.

Curing and storage

It is usual practice to cure samples of this size in a water pond for 7 days but to try to get a level of authenticity to the results, a curing regime is used that is more likely to be found in most developing countries. All of the samples are cured under wet/damp hessian for 7 days, the hessian is moistened on a daily basis. It is acknowledged that the weather patterns of a tropical country cannot be easily modelled in the workshop. Whilst the experiments were carried out the samples were stored at 25^{o}C, with a relative humidity of approximately 40%, this may be equivalent to an area such as Northern India in spring.

5.2 Tensile Strength

To help get a feel for the magnitude of tank wall thickness for various sizes of water tank. It is necessary to find out the tensile strength of the construction material. There is a great deal of documentation for the strength of concrete but little is known of the strength of ferrocement.

All the structural analysis is performed using thin shell theory. The assumption in this theory is that all the force is carried in tension through the centre of the material. There are three types of test for strength in tension: Flexural test, Splitting tension test, and Direct tension test. The first two tests are not suitable for the strength analysis used in the thin wall theory. In the Flexural test the reinforcing is at the outer edge of the sample.

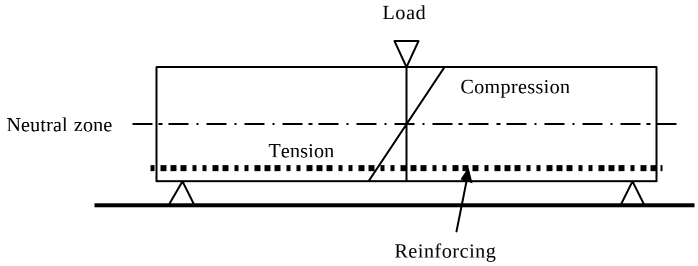


Fig 5.1 Modulus of rupture test

The indirect tensile test is used for testing the tensile strength of materials with a uniform constituency, such as concrete. This test is not suitable because the samples contain metal reinforcing.

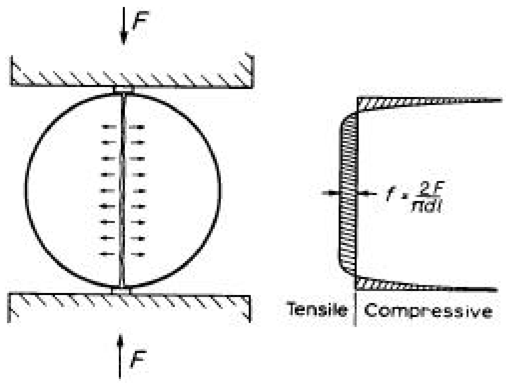


Fig. 5.2 Indirect tensile strength test (Kong and Evans 1985)

Therefore a more unconventional direct tension test has to be used to find the tensile strength. The direct tension test raises a number of problems, a direct application of a pure tension force, free from eccentricity, is very difficult. The test equipment used consisted of a square plate, into this a length of M10 threaded bar was screwed. On the other side of the plate four lengths of M6 threaded bar were attached.

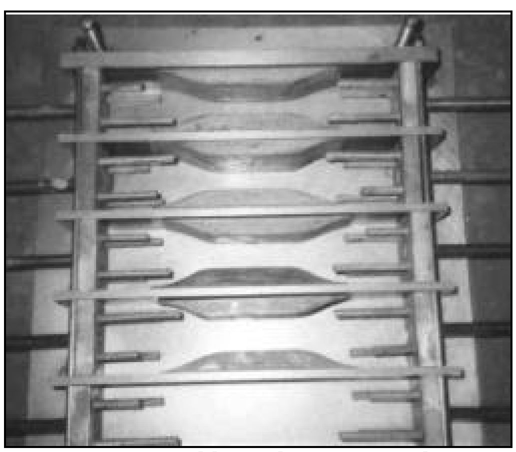


Fig. 5.3 moulds used to cast samples

The cement sample was cast onto the 6mm bar. The samples were manufactured in batches of five, in a mould (see figure 5.3). They were mechanically vibrated, this was carried out to aid compaction as there was difficulty manually compacting due to restricted access. To avoid creating a weak point at the end of the M6 retaining bars, the bar lengths were varied from 35 to 50*mm* long, see figures 5.4 and 5.5.

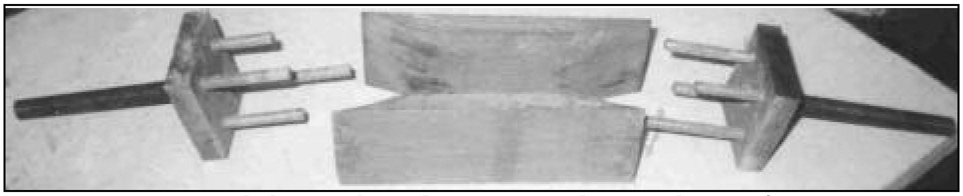


Fig 5.4 End plates and wooden inserts used in sample manufacture

The tensile load is applied through the 10mm bar by the tensile testing machine.

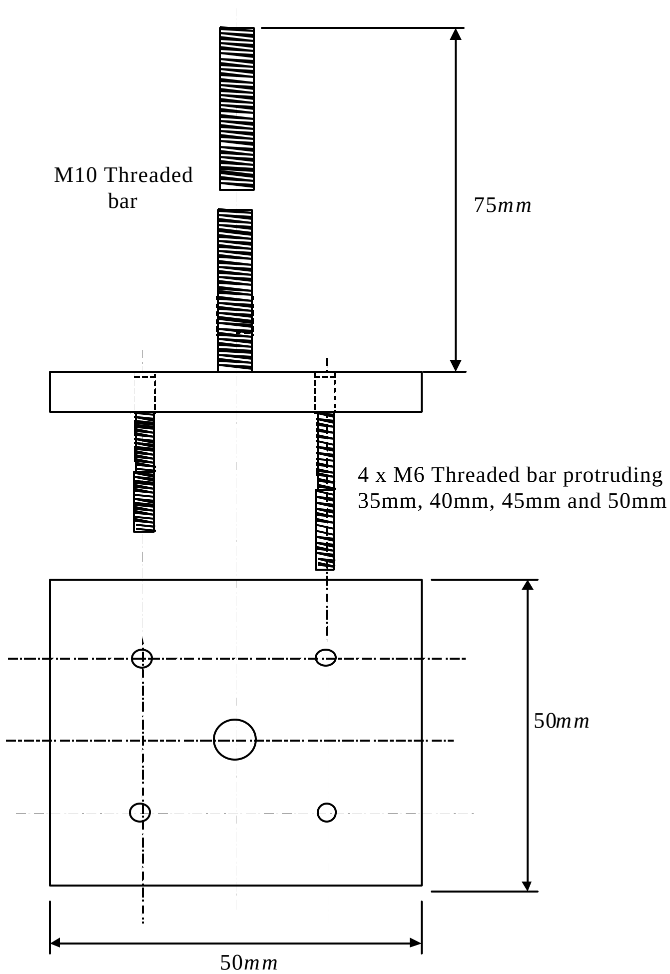


Fig. 5.5 Test plate assembly

The overall dimension of the specimen is shown in figure 5.6

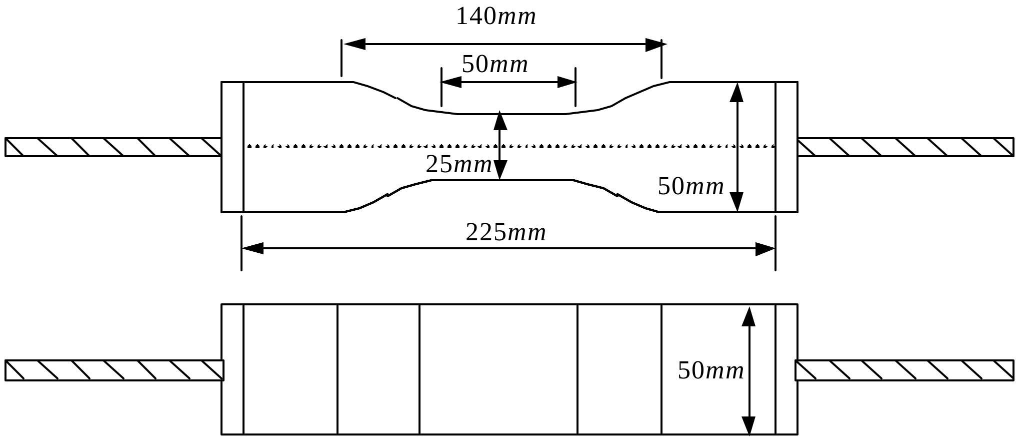


Fig 5.6 Specimen dimensions

To ensure the sample fails in the correct region the sample is necked using two wooden inserts, see figure 5.7.

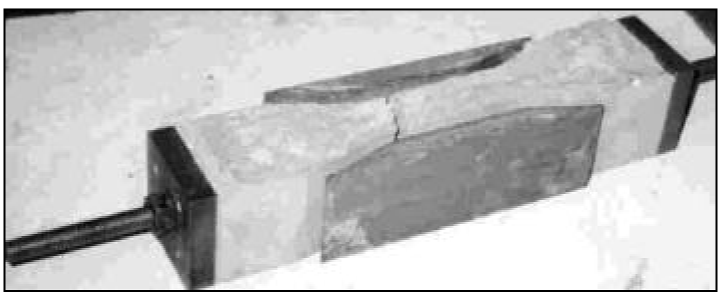


Fig 5.7 Wooden inserts used to neck samples

A full set of drawings of the test equipment can be seen in appendix II.

Results

The samples were tested using a Testometric 100kN tensile testing machine. The results only show breaking loads. During the test, loads were applied at a strain rate of 0.07*mm/min* until the cement failed, at this point the load was removed. In the case of the chicken wire reinforced samples the mortar and mesh failed at the same time. With the square mesh the mortar failed first. To increase the accuracy of the results five of each sample were made and the mean strength was found. A full set of results is given in Appendix III.

Results for samples reinforced with one sheet of 13mm 19 gauge (1mm diameter) galvanised mild steel square mesh (4 strands) and one layer chicken wire also galvanised.

Sample	Average Strength (MPa)	Standard Deviation	Coefficient of variance
Plain mortar	1.60	0.57	0.24
Reinforced with chicken wire	1.90	0.45	0.15
Reinforced with square mesh	2.40	0.41	0.13

Table 5 tensile test results

Discussion

The results show that the reinforced samples are stronger. It can also be seen that there is less variation in strength in the reinforced samples compared to the non-reinforced. The reinforcing mesh acts in two ways, it provides tensile strength, but more importantly it restricts crack growth. It can be seen in figure 5.8 that if there is a flaw in the sample it acts as a stress concentrator where cracks can start to grow. In the plain mortar samples there is nothing to stop the propagation of the crack. In the reinforced sample crack propagation is stopped or restricted when it meets the mesh. The effect the mesh has on the cement is analogous to rip-stop nylon.

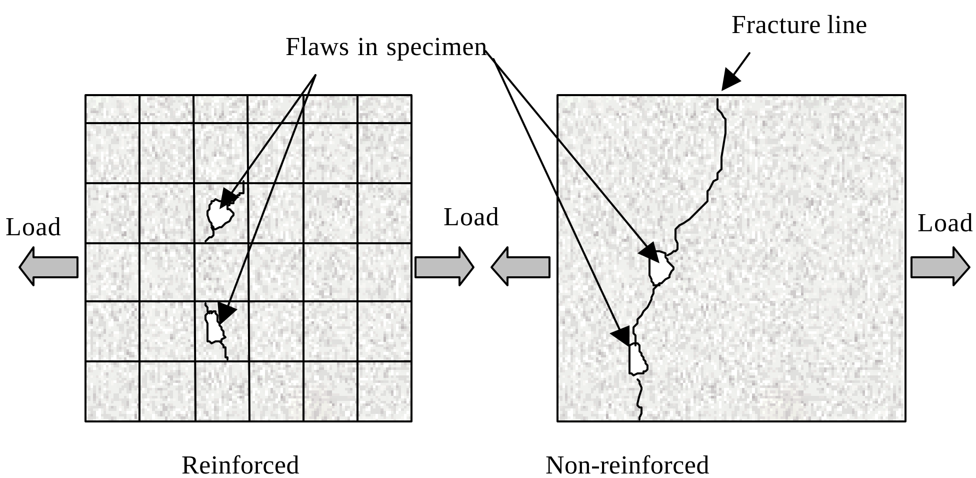


Fig. 5.8 Crack propagation in cement

Figure 5.8 shows one of the square mesh reinforced sample in the third stage of crack development (discussed in section 4.0). The sample was subjected to a tensile load, this load was increased until the cement mortar broke, then the load was removed. It can be seen that although the sample has failed it still retains its integrity, this implies that if the tank fails due to overloading, the failure will and will not be catastrophic but initially only lead to water leakage. If the cracks are small, e.g. less than 0.05mm, leakage will not occur. Cracks larger than 0.05*mm* will leak, this will cause water loss and corrosion on the reinforcement, which will also lead to spalling.

5.3 Unrestrained (free) shrinkage

The unrestrained shrinkage test is used to find the degree to which the specimen contracts if it is free from external clamping. These results are used to calculate the theoretical crack development of each sample. The unrestrained shrinkage tests were carried out on three samples, plain cement, chicken wire reinforced and square mesh reinforced. The reinforcement volume fraction and positioning was similar to that for the tensile testing (see figure 5.9). All of the samples are 225*mm* long and have a 50*mm* square cross section, see figure 5.9. The length of the sample over which the measurements were taken was 200*mm*. The unrestrained shrinkage test was not performed on any fibre reinforced samples as work in this area has already been carried out by Karagular & Shah (1990), see Appendix IV, and showed that the addition of fibres does not substantially alter the degree of free shrinkage.

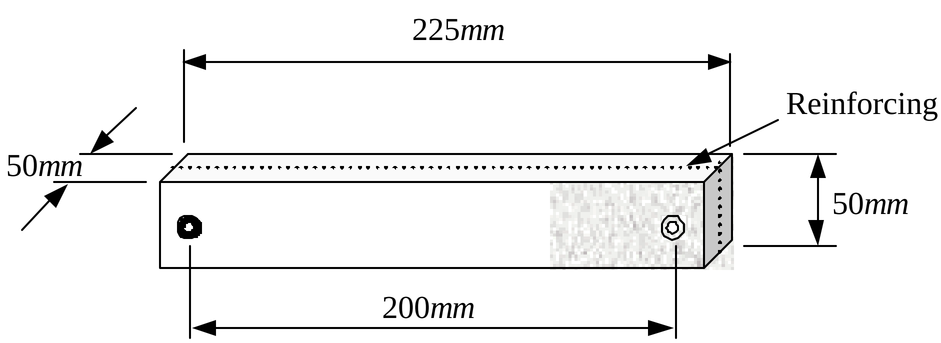


Fig. 5.9 Dimensions of unrestrained shrinkage sample

Daily readings were taken over a 28-day period using a demountable mechanical strain gauge (see Appendix II). To investigate any possible warping caused by differences in drying rates, measurements of shrinkage were taken two on sides of the sample, see figure 5.10.

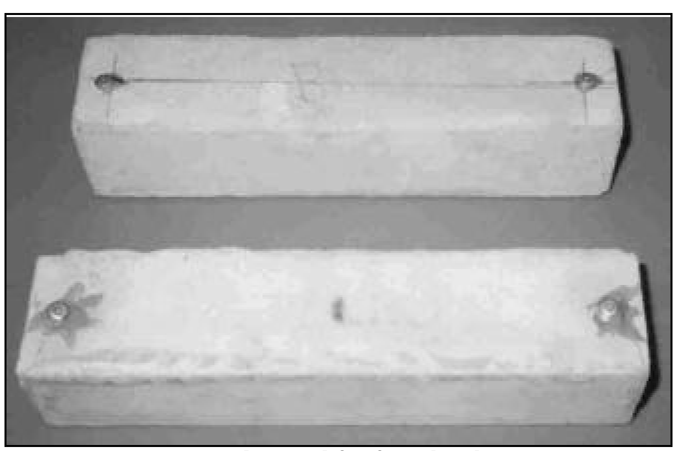


Fig 5.10 samples used for free shrinkage rests

Results

The results from this test compare favourably with the theory that was discussed in section 3.0 and Neville (1995). It can be seen in figure 5.11 that rates of shrinkage are relatively high, this is due to the high water/cement ratio (0.6), dry storage conditions (40% humidity), and an average temperature of 25^{0}C. The initial swelling is due to curing in a damp environment (under hessian), once this is removed the samples began to shrink. All data points are points are given in Appendix III.

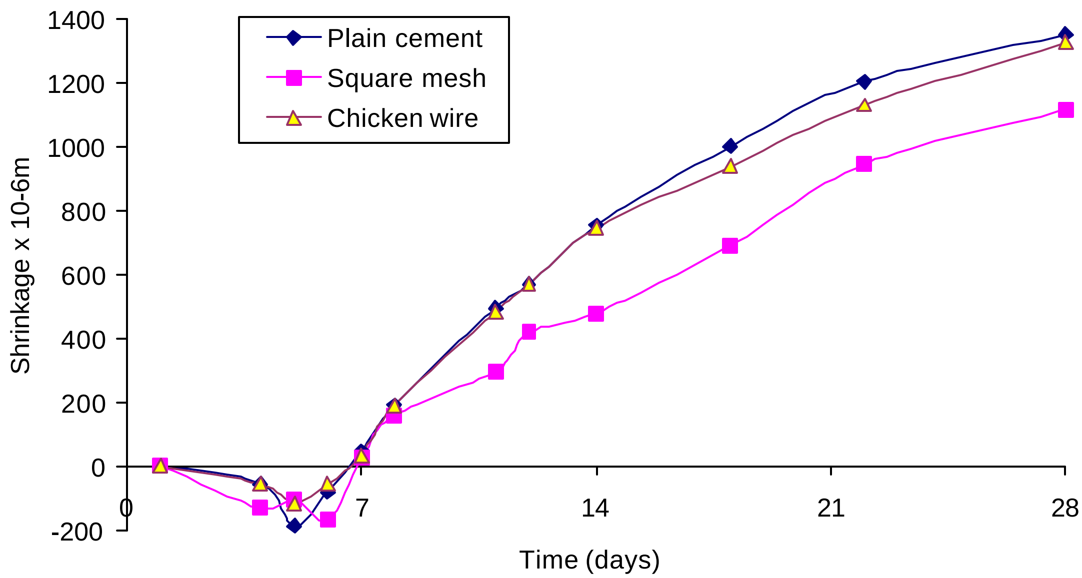


Fig. 5.11 Unrestrained Shrinkage results

Discussion

Cement swells when kept in moist conditions and shrinks in dry, the hotter and drier it is the faster the rate of shrinkage. As stated earlier, ferrocement tanks built in developing countries are very susceptible to rapid shrinkage because they have high wall area to volume, high water/cement ratios and they tend to be constructed in hot (and sometimes dry) climates. The results from the free shrinkage test show that there is initial swelling of the cement and as it dries it shrinks.

If the shrinkage is free to develop without any restriction, and it does not generate any extra stresses it should have little effect on the tanks performance. It can be seen that the use of chicken wire has little effect on the degree of shrinkage when compared to the plain sample. The ferrocement sample that contains square mesh has the greatest effect on the degree of shrinkage, this will induce additional internal stresses in the material.

The chicken wire does not perform as well as the square mesh when it comes to restricting shrinkage because it has a tendency to collapse under load, whereas the square mesh carries the load in same plane, see figure 5.1(a). There are two other reasons why the chicken wire may not perform as well:

- they are the wire is a smaller gauge,
- there is a lower volume fraction.

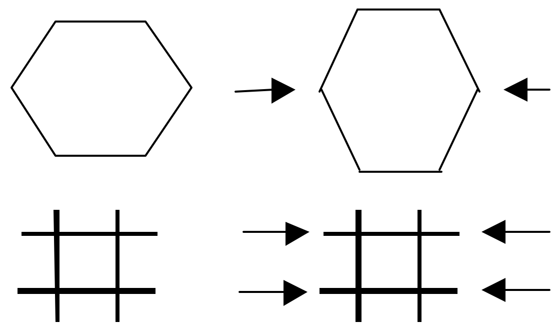


Fig. 5.11(a) Effects of compressive load on wire mesh

If the samples were restrained so that the strain caused by the shrinkage was not free to relieve itself, then the cracks would start to develop when the stress caused by the shrinkage exceeds the tensile strength of the material. Using the values for Youngs modulus in appendix I, i.e. 3.0GPa for cement and 210GPa steel, and knowing the volume fraction it is possible to calculate the overall Youngs modulus for each specimen. Using the results from the tensile testing (table 5), the theoretical time after which cracking will start to occur can be calculated, these are shown in table 5.1.

Sample	Tensile strength (s) MPa	E (GPa)	$e=\frac{s}{E}$ (me)	No. of days before cracking starts (see figure 6.7)
Plain	1.6	3.50	373	9.5
Chicken wire	1.9	3.67	518	10.5
Square mesh	2.4	3.85	623	17

Table 5.1 Breaking strains

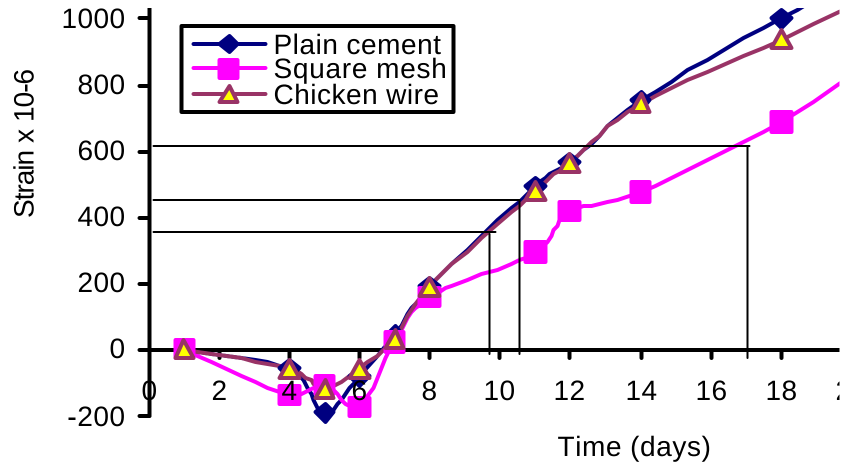


Fig 5.12 Breaking strain

It can be seen from figure 5.12 that the higher the tensile strength, the longer it will be before cracks start to develop. The practical implications are that to stop the tank cracking one or more of the following must be done:

- the amount of reinforcement needs to be increased, Skinner (1995) recommends a volume fraction of at least 5.1 to 6.3%,
- the tank needs to be kept in moist conditions so shrinkage is minimised (see section 3, figure 3.3),
- or the wall thickness needs to be increased so that the tensile strength is higher.

5.4 Restrained Shrinkage

The restrained shrinkage test is used to investigate how the cracks develop in a cement specimen. The test is aimed at mimicking the effect of cracking in the tank wall caused by the restriction in movement due to the base/wall junction (see figure 2.1). There is no British Standard test to assess cracking caused due to restrained shrinkage so a test had to be devised. The first test investigated cracking a reinforced specimen block, this test proves to be unsuccessful so a second test was used. The second test investigated cracking in a ring sample. The main failure in the first test was the effectiveness of the retaining pins, using a ring specimen this problem was overcome.

Test Method 1

An initial test was devised to examine when and where the specimen would crack if the shrinkage were restrained. To ensure that the test gave reliable results it was important to ensure that the two ends of the cement specimen were not free to move. A test rig was set up using a section of steel 'U' channel and two of the end plates from the tensile testing experiment, see figure 5.13. The 'U' channel had two purposes, it was a mould for the sample to be cast in to and it also ensured that the end plates did not move.

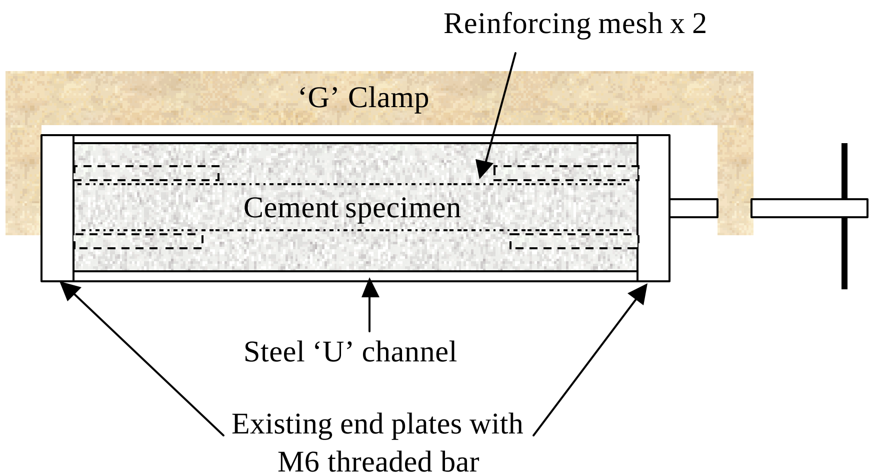


Fig. 5.13 Initial crack test

Results

This test gave inconclusive results, there was initial cracking, but these did not develop beyond micro cracks (less than 0.1*mm*), see figure 5.14.

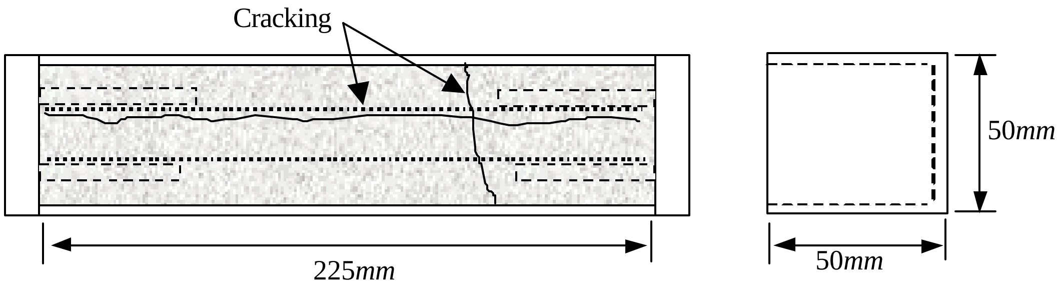


Fig. 5.14 Cracking caused by shrinkage

It can be seen from the specimen in figure 5.15 that even after 28-days the cracks were difficult to see without very close examination.

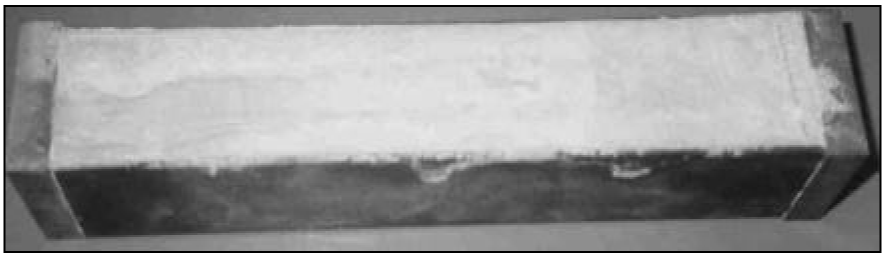


Figure 5.15 Micro crack development in restrained sample

The results from the unrestrained shrinkage tests showed that the amounts of shrinkage should have been much greater. These inconclusive results could have been due to a number of reasons such as that the specimen was cast in a steel ‘U’ channel therefore the cracks could only be viewed from surface also the moisture could only escape through one face. Another factor affecting the poor results was that the cement mortar might have pulled off the threaded retaining pins therefore leaving the mortar free to contract. All these factors lead to the test being discarded and a more reliable test being developed.

Test Method 2

Test method two overcomes the problem of holding the sample. A ring mould is used (see figure 5.16) and the cement is then rendered onto the outside of it. This test is likely to give more reliable results compared to the first test as it more resembles a ‘real life’ tank. In similar way to an actual tank, the ring mould gives greater surface area where moisture can escape therefore modelling real life conditions.

Due to time restrictions and uncertainties in the testing technique only one of each sample was made, one of each of the following:

- Reinforced with one layer of square mesh (see figure 5.17),

– Reinforced with one layer of chicken wire (see figure 5.17),

– Reinforced with 'home-made' polypropylene fibres,

– Plain mortar.

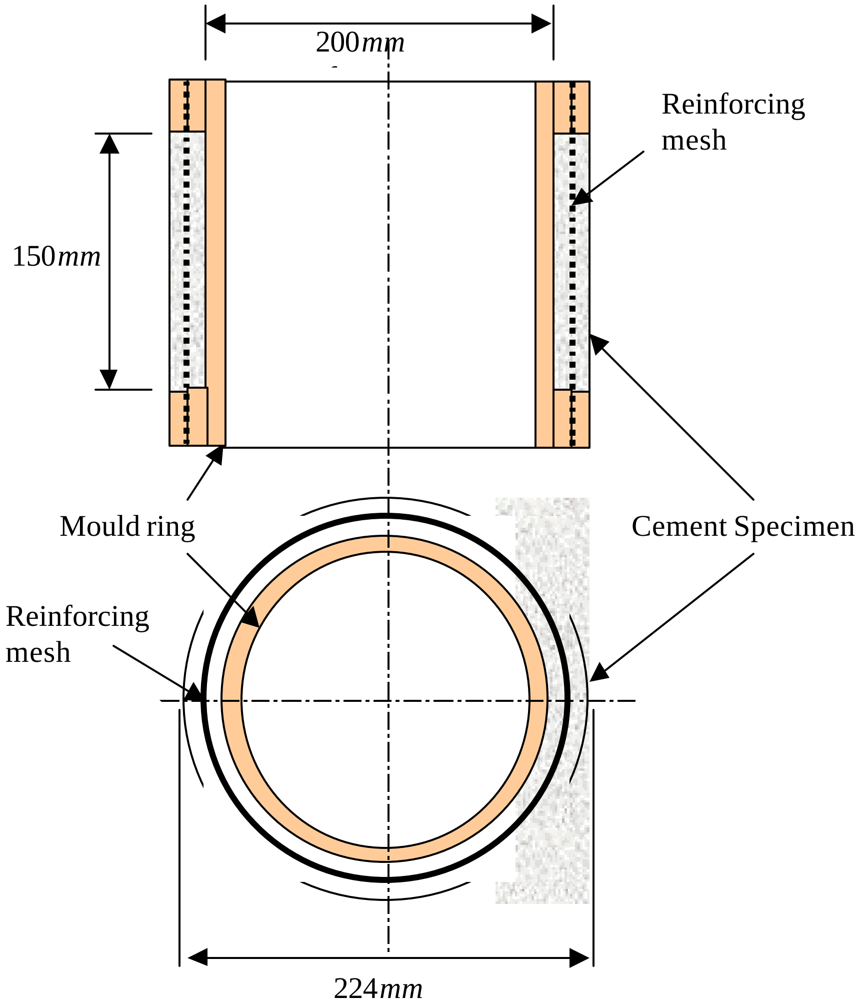


Fig. 5.16 Ring test used to investigate cracking

As with all the other samples, these were manufactured using the procedure described in 6.1, i.e. cured under wet/damp hessian for 7 days. Whilst the experiments were carried out the samples were stored at 25°C, with a relative humidity of approximately 40%. The cement used is OPC, which is mixed with standard building sand to a ratio of three parts sand to one part cement by weight, with a water/cement ratio of 0.6.

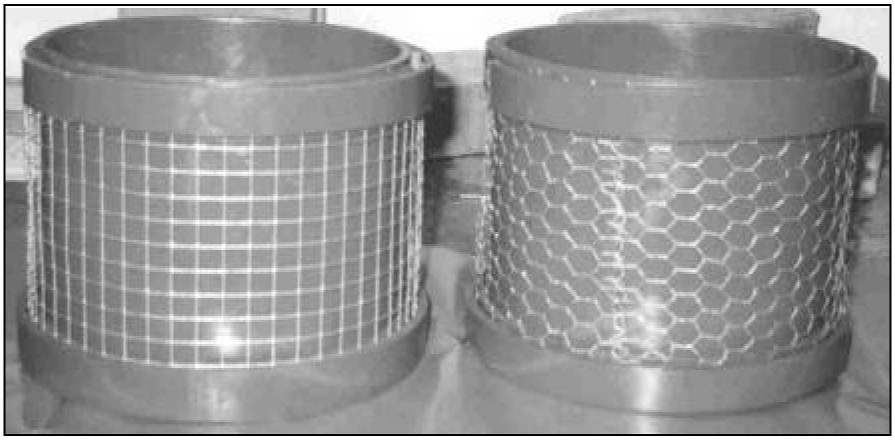


Fig 5.17 Reinforcing on test rings before mortar is rendered on.

As with both the tensile and unrestrained shrinkage, the restrained shrinkage tests were also carried out over a 28-day period.

Results

Plain mortar

The plain mortar specimen (figure 5.18) showed signs of cracking after five days. A single crack developed at a relatively linear rate, approximately 0.09mm/day. After 28 days the crack width was approximately 1.75mm.

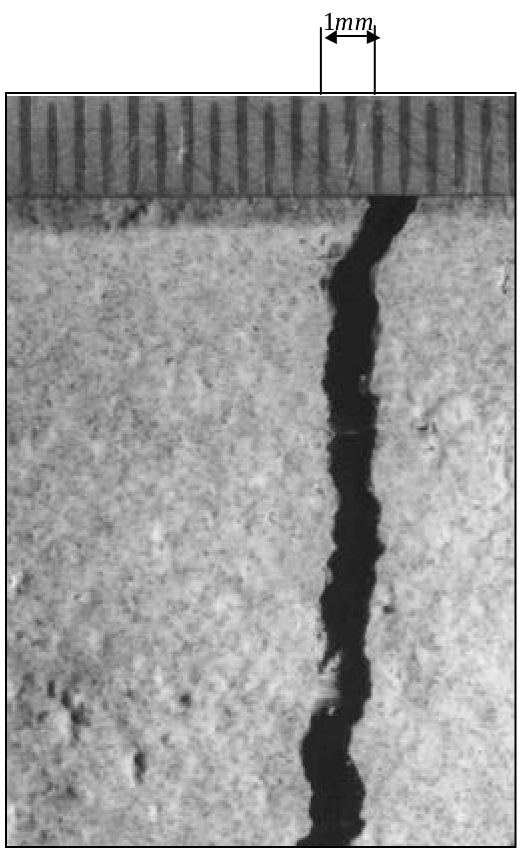


Fig. 5.18 Crack in plain mortar sample

Fibre reinforced

In this case two vertical cracks developed over the complete length of the specimen, they were approximately 180^{o} apart. After 28 days the larger crack was slightly less than 0.5mm (figure 6.19(b)) and the smaller one approximately 0.1mm (figure 5.19(a)). The cracks started to develop after 6 days and developed at a relatively linear rate, the larger crack at approximately 0.02mm/day and the smaller at 0.005mm/day.

Fig. 5.19(a) Small crack in fibre reinforced sample

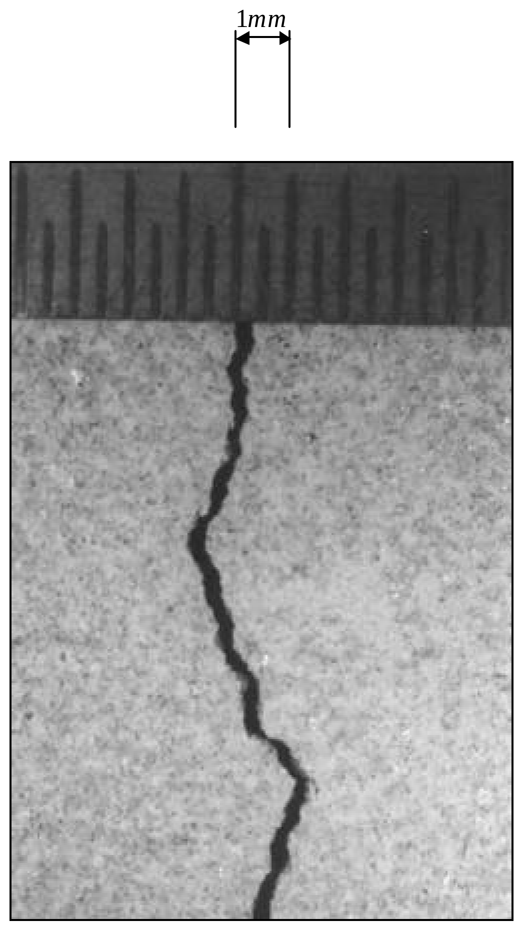


Fig. 5.19(b) Large crack in fibre reinforced specimen

It cannot be clearly seen in the photographs but the fibre strands in the mix bridged the gaps between the cracks helping hold the sample together and therefore reducing shrinkage. This is illustrated in figure 5.20. The fibre also added to the tensile strength of the specimen, this also helped to reduce the shrinkage.

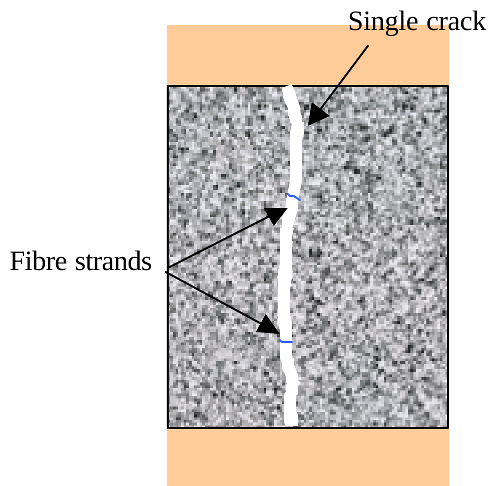
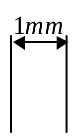


Fig. 5.20 Fibre reinforcing

Chicken wire reinforced

In the specimen containing chicken wire reinforcing the crack size was greatly reduced compared with the two previous samples. It can be seen from figure 5.21(a) that chicken wire reinforcing increased the number of cracks but reduced the size of individual cracks. Figure 5.21(b) shows a single larger crack with a width of approximately 0.2mm, due to the effects of the reinforcing. Towards the top of the specimen, the crack divides into two smaller cracks. Again in this case the cracks run vertically and are approximately 180^{o} apart, the cracks started to develop after 7 days.



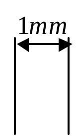


Fig. 5.21(a) Development of small cracks in chicken wire reinforced specimen

Fig. 5.21(b) Large cracks in chicken wire reinforced specimen

Square mesh reinforced

The specimen containing square mesh reinforcing showed the least amount of cracking. There were a number of minor hairline cracks propagating from the edge (figure 5.23). The specimen only had one major crack, see figure 5.22(a). Of all the minor cracks the greatest can be seen in figure 5.22(b). The cracks started to develop after 8 days, the rate of growth was relatively slow compared to the previous three and the rate decreased over time.

Fig. 5.22(a) Single cracks in square mesh reinforced specimen

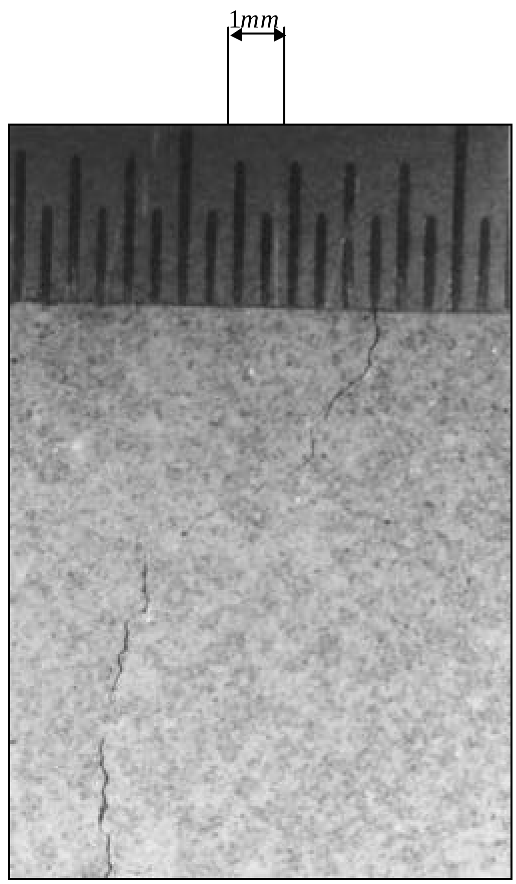


Fig. 5.22(b) Distributed cracks in square mesh reinforced specimen

Discussion

In all cases, the cracks started to develop at the edge of the specimens and propagated vertically towards the centre. In all the cases apart from the square mesh reinforced sample, the cracks were approximately 180^0 apart. The cracks started at the edge due to stress raisers caused by the rough surface finish of the cement mortar. This can be seen in figure 5.23. If the surface between the ring mould and the sample had a zero friction factor then only one crack would develop, if the friction factor is greater than the force needed to start a crack, a new crack will develop. This is what happened in this case.

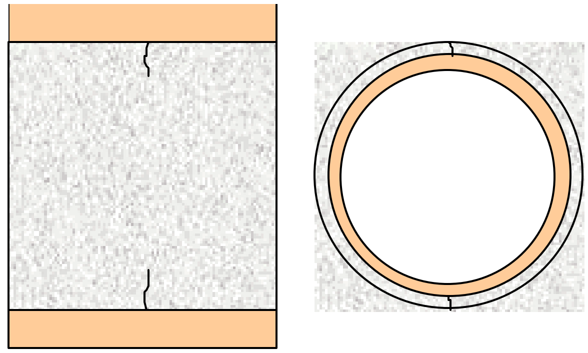


Figure 5.23 Crack development

One of the factors that affects the rate of shrinkage is the environment. In the early stages where rates of shrinkage are high, this has a very important role. The experimental results show that ferrocement cured and stored in a dry environment (40% relative humidity) is very susceptible to cracking (all samples started to crack after 5 to 7 days).

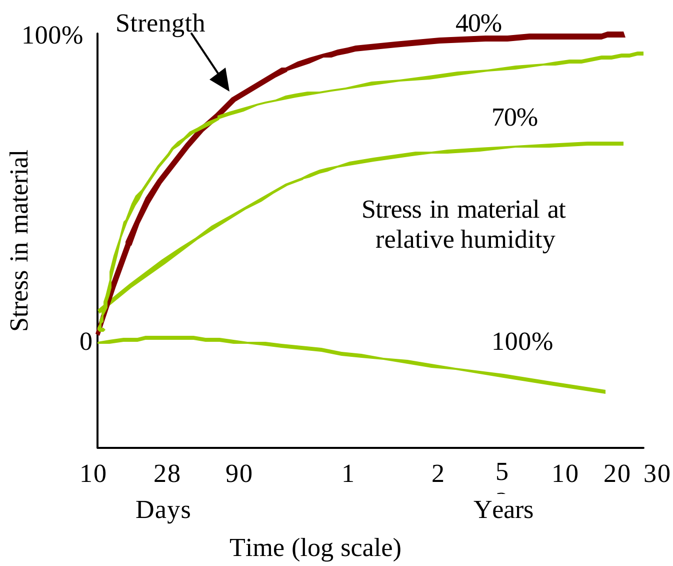


Fig. 5.24 Stress in material caused by shrinkage

To take full advantage of the composite strength it is important not to allow the cement to crack. This can be achieved by making the walls thicker as this slows down moisture loss and also increases overall strength. Another way of achieving this is to keep the cement in such an environment that shrinkage will not occur preferably in moist conditions.

Figure 5.24 shows the theoretical development of stress caused by restrained shrinkage against the development of stress in the cement as discussed in section 3.0. It can be seen that at a relative humidity of 40% the stress developed is greater than the strength of the material, causing the material to crack or fail. If the cement is in an environment with a higher humidity the cracking will not occur. From the figure 5.24 it is seen that after about 40 days the composite has gained enough strength to withstand the internal stresses caused by shrinkage. These results have a practical application, if the water tank is constructed in humid conditions it will be less likely to crack due to shrinkage.

Fig 5.25(a) Polypropylene fibre reinforced

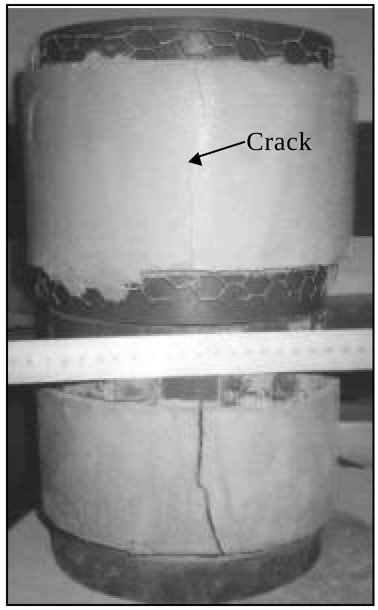


Fig 5.25(b) Chicken wire reinforced

Fig 5.25(c) Square mesh reinforced

Figures 6.25(a-c) show the relative crack size when the plain cement specimen (on the bottom) is compared to the three reinforced specimens. Compared to the crack development in the plain specimen, the specimen containing square mesh restricts the shrinkage cracking by approximately 95%, the specimen containing chicken wire by 80% and the fibre reinforced specimen by 52%. The mesh reinforcing disperses the cracks into a series of micro-cracks. In the specimen containing square mesh the majority of these micro-cracks are less than 0.05mm, according to Nedwall and Swany (1994) cracks of this size should not leak.

One of the reasons the fibre reinforcement does not restrict crack size when compared to the other two is because it has a lower tensile strength 29-38MPa compared to 215 MPa for steel mesh as well as having a higher ductility. Another reason is that there may be a tendency for the reinforcing to pull out of the mortar matrix.

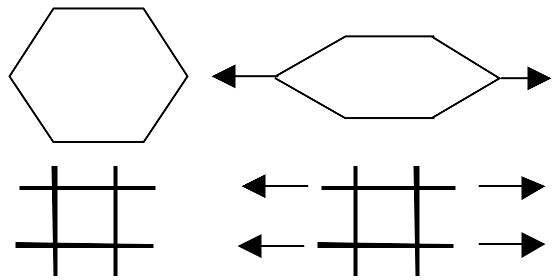


Fig. 5.26 Effects of tensile loads on wire mesh

The main reason why the specimen containing the square mesh restricted the cracking to a greater degree compared to the chicken wire can be seen in figure 5.26. When chicken wire is loaded there is a tendency for the mesh to ‘flatten’ out and expand in the plane of the load. In the case of the square mesh, the load is carried in the same plane therefore the only expansion is due to the wire mesh stretching. There are also a number of other factors which effect the degree to which a specimen shrinks such as volume fraction and the wire gauge. Generally, the larger the volume of steel reinforcing and the larger its gauge the lower the shrinkage will be.

6.0 DISCUSSION

Spreadsheet

In the current spreadsheets the designer is free to enter any safety factor they wish. It would be useful if the designer had some feel for what size of safety factor is required. For example, if the designer knows that the tank is going to be constructed by skilled masons, with high quality materials in a cool wet environment, a safety factor of 2 to 3 could be used. If a designer is not sure of the quality of the raw materials or skills of the masons but knows it will be constructed in a cool wet environment a safety factor of 4 to 5 could be used. If the designer has little knowledge of the environment or the quality of the materials a higher safety factor of 6 to 8 may be used.

Materials

For a typical ferrocement tank constructed in Kenya costs vary from 26$US\$/m^3$ for a 46m^3 tank to 50$US\$/m^3$ for a 11m^3 tank (Gould 1999). Approximately 80% of the cost is materials. If the tanks are going to be financed by the individual householder or community this cost will normally be out of their range. Any saving in the cost of raw materials can be of great benefit, for example, the cost of cement in Uganda is equal to its cost in the UK, but the average income in Uganda is approximately 1/30th to that in the UK. Therefore the real cost of cement is equivalent to 30 times the cost in the UK, (£100-£150 per 50kg). One of the best ways of lowering these costs is to reduce the amount of expensive raw materials. This can achieved be through in a number, this report investigates tank design and improved construction methods.

The spreadsheet allows for experimentation with tank design. Results from the mechanical testing show that reinforcing square mesh inhibits shrinkage better than chicken wire. Reinforcing is used not only for strength but also to add uniformity to the structure. It also gives a base for the cement to be rendered onto. The reinforced samples have greater strength uniformity, this gives the designer greater confidence in predicting the strength of the materials thus allowing for reduced safety factors and therefore thinner walls which in turn reduce material inputs.

Construction techniques

One of the main problem areas in tank design is the wall/base interface. It was shown using the spreadsheet that the interface causes additional stresses, but more importantly the interface restricts the free shrinkage of the tank walls therefore generating additional forces at the tank base of the walls. There are two ways to overcome this problem, either strengthen the base/wall interface or construct the tank so the wall is not connected to the base and it is free to move. The first method increases material costs and second method adds to construction complexity. The final decision on what option to take is up to designer/builder, factors such as cost of additional materials and levels of available construction skills have to be taken into account.

7.0 GUIDE TO THE DESIGNER AND MANUFACTURER

The project has looked into many aspects of tank design and construction. This section summarises the findings into a set of useful guide rules that will help the designer and/or manufacturer. Some of these rules are already in use in the construction industry. Using theory and experimental results these rules have been made specific to overcome problems that are particular to constructing ferrocement water tanks in hot, dry climates.

One of the main problem areas is differential shrinkage; this can be reduced in a number of ways,

- The wall thickness should be kept to a minimum and where possible constant throughout the tank, this will reduce the build up of internal stress. For example, a wall thickness of 75*mm* internal stresses caused by shrinkage can vary by 20%.

- The tank should not be constructed so it is half shaded, this will cause internal stress to be built up. Ideally, the tank should be constructed in a shaded area, this will reduce rapid water loss.

- If possible the application of cement mortar should be continuous. Rendering wet mortar onto dry should be avoided, as stresses will build up between the layers. The maximum recommended time between layers is 4 hours.

Hot dry weather has an adverse effect on tank strength.

- Where possible tanks should be made in the wet season or in the coolest part of the day, ideally in the morning so the construction materials have had the opportunity to cool down overnight.

It is important to incorporate good building practices and have an understanding of the properties of the construction materials.

- The water/cement ratio should be kept as low as possible as a low water/cement ratio will maximise strength and minimise shrinkage. Cement mortar having a water/cement ratio of 0.6 is 40% weaker than that with a water/cement ratio of 0.4.

- There is less variation in strength in reinforced cement compared to non-reinforced. In the tensile strength experiment, compared to plain mortar, variations were reduced by 40% and 45% for chicken wire and square mesh respectively. The lower the variation the lower safety factors that can be used.

- Increasing the amount of reinforcement can reduce cracking. Skinner (1995) recommends a volume fraction of at least 5 to 6%.

- Ensure that not only fine gauge mesh (chicken wire) is used, from experimental observations it was seen that when failure did occurred it was catastrophic.
- The finer the mesh size, the slower and more disperse the crack propagation. Using experimental results, compared to plain mortar the chicken wire reduced cracking by 80% and the square mesh by 95%.
- A good curing regime is required, i.e. the use of wet hessian and plastic covers. The tensile strength of a tank cured in water is approximately 300% greater than that cured in dry air. Appropriate early curing is very important as cement acquires 75% of its final strength within 14 days of casting.
- Shrinkage and cracking can be reduced if the tank is partly filled with water. This needs to be carried out as soon after construction as possible, the depth is not critical as long as the important wall/base junction is covered, e.g. 75 to $100mm$.
- Use of safety factors, for example, 2-3 for a tank constructed by skilled labour, with high quality materials in a cool wet environment, 4-5 if the materials are of dubious quality but there are skilled labour, and 6-8 if there is little knowledge of environment, quality of the materials and labour.

For tanks of similar storage volume the material input for tanks with doubly curved walls are lower than that for tanks with singly curved walls. For example, a Thai jar uses approximately 30% less construction materials than a conventional cylindrical tank of similar volume.

8.0 FUTURE WORK

Spreadsheet

The spreadsheet that is used to calculate the force in doubly curved tanks needs to be made more user friendly. The effects of edge loads also need to be incorporated. The spreadsheets do not take into account the effect of shrinkage and cracking. At the moment their effects are covered by the use of safety factors. Ideally, it should possible to enter a numerical correction factor, which has a similar effect as the safety factor, this may vary depending on construction materials and environmental conditions.

The current spreadsheets do not cater for tanks with non-uniform wall thickness, further work needs to be carried so that this facility is possible. Once this facility has been added it will be posted on the DTU's rainwater harvesting web site and available to the general public.

Construction materials

Future RWH work may concentrate on developing lower cost construction materials. Ideally these should be locally sourced. Work needs to carried out on a replacement for OPC using rice ash husks, rice straw ash and peanut shell ash and future study needs to be carried out on how they reduce shrinkage and therefore cracking. The cost elements need to also be examined. Tank construction materials that could be investigated include bamboo-mud composites, especially in remote rural areas where access to conventional materials such as cement and steel reinforcing bar may be limited. In tropical areas one option is latex, this could be used in conjunction with jute or coir (coconut shell) sacking, as a waterproofing material. Flexible tanks could be made from this composite.

Construction techniques

The merits of shotcreting or the spraying of cement could be studied. Shotcreting is the spraying of concrete at high velocity, conventionally it is projected through a hose usually to a thickness of 50mm. The advantage of spraying concrete/cement is that the mix can be much drier (water/cement ratio 0.30 to 0.50), which in turn will reduce shrinkage and cracking. Spraying cement will give thin uniform cover over reinforcing mesh, as well as reducing application time.

9.0 CONCLUSIONS

Thin wall ferrocement tanks have many advantages and their use should be promoted. One of the main factors that reduce their widespread use is the cost of construction materials, in developing countries it is approximately 80% of the overall cost. The project has looked at the problems that can occur when material inputs are reduced, these are mainly the problems caused by shrinkage induced cracking.

Due to the restricted sample size the accuracy of the results from the material testing may be limited. The purpose of testing was to get a ball park figure for tensile strength and investigate the mechanics of shrinkage of ferrocement. Three reinforcing regimes have been examined and their effects of tensile strength and shrinkage have been reported.

To help optimise material inputs two spreadsheets have been produced. These spreadsheets give the designer an opportunity to experiment with many shapes of tank, as well as material properties. The spreadsheets also have the added advantage of speed, allowing the designer to make changes to the tank specification and immediately seeing the consequences of their actions. This allows iterations to be made quickly and easily. For example comparing a Thai jar (figure 2.7(b)) style water tank to standard cylindrical tank (figure 2.7(a)) of a similar volume material inputs can be reduced by approximately 30% (assuming the material is homogeneous).

One of the main problems that can arise when material inputs are reduced is cracking that is caused by shrinkage. Various ways of reducing this and cracking has been investigated. They include,

- Reducing the effects of potentially damaging differential shrinkage,
- Incorporating good curing regimes,
- Studying the role reinforcing plays in reducing shrinkage and cracking.

The project has also sought to improve construction practices that will help with the manufacture of thin walled ferrocement tanks. A guide for the designer/manufacturer has been produced, this gives a list of some simple, practical and easy to follow rules that can be used in the ‘field’ to help overcome the problems that can occur when material inputs are reduced.

10.0 BIBLIOGRAPHY

Calladine C.R. (1983), *Theory of Shells Structures*, Cambridge University Press.

Cartwright A.J. ed. (1996), *Engineering Data Book*, University of Warwick, Coventry.

Childers L.F. (1985), *Materials testing and mixing*, UNICEF, Nairobi, Kenya.

Davis H. E. (1940), *Autogenous volume changes of concrete*, ASTM, USA.

Flugge W. (1967), *Stresses in Shells*, Springer-Verlag, New York.

Gould J & Nissen-Petersen E (1999), *Rainwater Catchment Systems for domestic supply*, Intermediate Technology Publications, UK.

Gray W.S. (revised by Manning G.P.) (1960), *Reinforced Concrete Reservoirs and Tanks*, Concrete Publications Ltd., USA.

Ichikawa R. (1995), *Rainwater and you: 100 ways to use rainwater,* Tokyo International Rainwater Utilisation Conference, Japan.

Karagular M.E. & Shah S.P. (1990) *A test method to evaluate shrinkage cracking of concrete*, Proceedings of the material Engineering congress, American Society of Civil Engineers, USA.

Kelkar.S. & Sewell R.T. (1987), *Fundamentals of the Analysis and Design of Shell Structures*, Prentice-Hall, New Jersey.

Kong F.K. & Evans R.H. (1980), *Reinforced and Prestressed Concrete*, Pitman, Bath.

Nedwall P.J. & Swamy R.N. ed. (1994), *Ferrocement*, Proceedings of the 5th International Symposium on Ferrocement (1994), Chapman & Hall, London.

Neville A. M. (1971), *Hardened Concrete: Physical and Mechanical Aspects*, American Concrete Institute, Detroit, USA.

Neville A.M. (1995), *Properties of Concrete*, Longman, England.

Pacey A & Cullis A (1985), *Rainwater Harvesting*, Intermediate Technology publications, UK.

Shigley J.E. (1983), *Mechanical Engineering Design*, McGraw-Hill, Singapore.

Singh G. (1989), *Control of Cracking in Concrete Structures*, ACI Journal, Proceedings V69 No. 12, 1972, pp 717-753.

Skinner B. (1995), *Ferrocement water storage tanks*, Sustainability of water and sanitation systems, 21st WEDC Conference, Kampala, Uganda.

Smerda Z. & Kristek V. (1988), *Creep and shrinkage of concrete elements and structures*, Developments in civil engineering: 21, Elsevier, Amsterdam

Stulz R. (1998), *Appropriate Building Materials*, Intermediate Technology Publishing, UK.

Turner S. (1999), *Design of storage tanks for rainwater harvesting*, Indian Institute of Technology, Delhi, India.

White G.R. (1991), *Concrete Technology*, Delmar Publishers Inc, New York.

Zingoni A. (1997), *Shell Structures in Civil and Mechanical Engineering*, Thomas Telford, London.

APPENDIX I
Construction Materials

Construction Materials

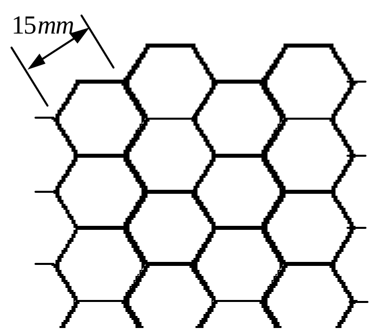
Figures from the Engineering data book (Cartwright 1996)

	Material	Youngs modulus (GPa)	Ultimate Tensile Strength (MPa)	Yield Tensile Strength (MPa)
Chicken wire	Mild steel	210	430	280
Square mesh	Mild steel	210	430	280
Polypropylene fibres	Polypropylene	0.9-1.38	29-38	-
Cement mortar	OPC:Sand 3:1	3.5	1.60*	

*From experimental readings

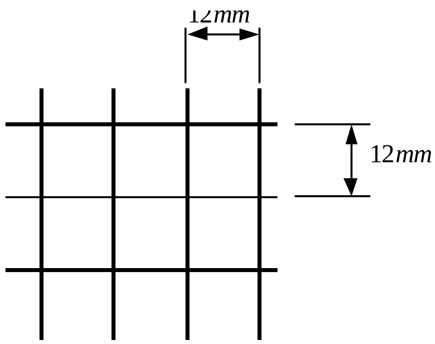
Chicken wire

Chicken wire is hexagonal mild steel galvanised mesh approximately 0.5mm in diameter

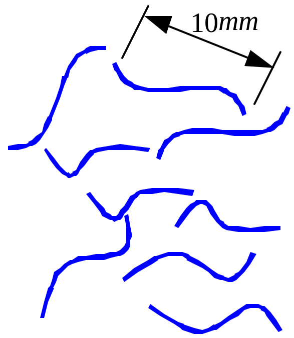


Square mesh

The 12mm square mesh wire is made from mild steel galvanised welded wire approximately 1mm in diameter



Polypropylene fibres



Home-made fibres made from polypropylene rope, individual fibres having a diameter of approximately 0.5mm.

APPENDIX II

Material testing equipment

Tensile testing equipment

See drawings of plate, plate assembly and insert overleaf.

Free shrinkage

The free shrinkage was measured using a *Demec gauge* as shown below

Demountable Mechanical Strain Gauge

APPENDIX III
Material Test Results

Tensile strength test results

Results for non-reinforced samples

	Load		Strength	
Sample 1	2.68	kN	2.14	MPa
Sample 2	2.44	kN	1.95	MPa
Sample 3	1.13	kN	0.90	MPa
Sample 4	1.67	kN	1.34	MPa
Average	**1.38**	**kN**	**1.58**	**MPa**
SD	**0.71**		**0.57**	
COV	**0.38**		**0.24**	

Results for chicken mesh reinforced samples

	Load		Strength	
Sample 1	2.78	kN	2.22	MPa
Sample 2	1.55	kN	1.24	MPa
Sample 3	2.65	kN	2.12	MPa
Sample 4	2.53	kN	2.03	MPa
Average	**2.38**	**kN**	**1.90**	**MPa**
SD	**0.56**		**0.45**	
COV	**0.24**		**0.15**	

Results for square mesh reinforced samples

	Load		Strength	
Sample 1	3.27	kN	2.62	MPa
Sample 2	2.34	kN	1.88	MPa
Sample 3	2.32	kN	2.34	MPa
Sample 4	3.54	kN	2.83	MPa
Average	**3.02**	**kN**	**2.42**	**MPa**
SD	**0.51**		**0.41**	
COV	**0.20**		**0.13**	

COV = Coefficient of Variance, **SD**= Standard deviation

Unrestrained shrinkage results

Square mesh

Day	Dummy reading	Side A	Side B	Strain A	Strain B	Averag e strain
1	648	648	648	0	0	0
2	648	663	666	-121.5	-145.8	-133.65
5	648	658	664	-81	-129.6	-105.3
6	648	667	671	-153.9	-186.3	-170.1
7	648	644	646	32.4	16.2	24.3
8	648	627	630	170.1	145.8	157.95
9	648	611	612	299.7	291.6	295.65
12	648	619	621	424.9	408.7	416.8
13	648	611	614	489.7	465.4	477.55
15	648	590	594	659.8	627.4	643.6
19	752	663	660	910.9	935.2	923.05
23	745	649	648	967.6	975.7	971.65
28	745	632	630	1105.3	1121.5	1113.4

Plain cement

Day	Dummy reading	Side A	Side B	Strain A	Strain B	Averag e strain
1	648	648	648	0	0	0
4	648	660	650	-97.2	-16.2	-56.7
5	648	670	673	-178.2	-202.5	-190.35
6	648	658	658	-81	-81	-81
7	648	643	642	40.5	48.6	44.55
8	648	624	625	194.4	186.3	190.35
11	648	586	588	502.2	486	494.1
12	648	576	580	583.2	550.8	567
14	648	555	555	753.3	753.3	753.3
18	648	625	632	1028.7	972	1000.35
22	752	595	598	1215	1190.7	1202.85
28	745	577	579	1360.8	1344.6	1352.70

Chicken wire

Day	Dummy reading	Side A	Side B	Strain A	Strain B	Average strain
1	648	648	648	0	0	0
4	648	658	652	-81	-32.4	-56.7
5	648	660	665	-97.2	-137.7	-117.45
6	648	658	652	-81	-32.4	-56.7
7	648	643	645	40.5	24.3	32.4
8	648	624	625	194.4	186.3	190.35
11	648	586	588	502.2	486	480
12	648	576	580	583.2	550.8	567
14	648	555	555	753.3	753.3	745
18	648	530	535	955.8	915.3	935.55
22	752	610	615	1150.2	1109.7	1129.95
28	745	582	580	1320.3	1336.5	1328.4

APPENDIX IV

Unrestrained shrinkage

Unrestrained Shrinkage

Base on free shrinkage test carried out by Karagular and Shah (1990) on concrete samples, cured in water for four hours and then after demoulding dried at 40% humidity.

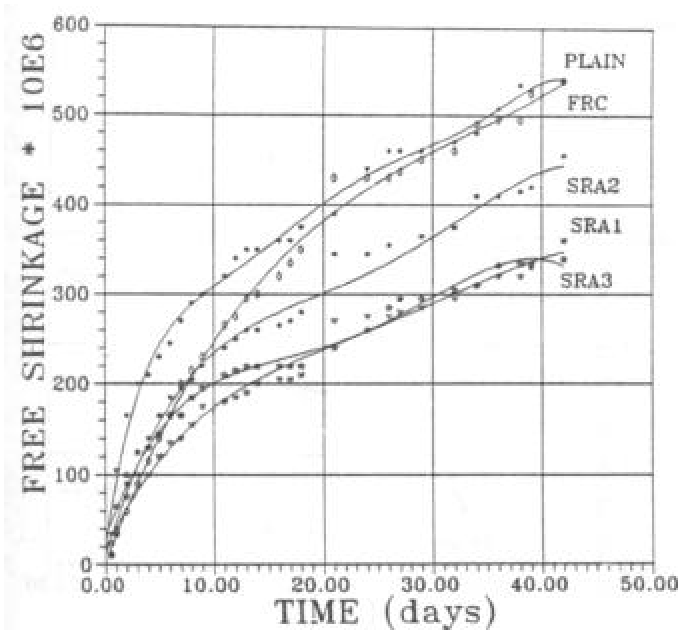


Fig A 4 free shrinkage test results (Karagular and Shah 1990)

Where;

- FRC is fibre reinforced
- SRA is shrinkage reducing agent

The plain concrete was reinforced with steel fibres, the amount being 0.5. Work was also carried out on the effects of shrinkage reducing agents, these agents were a commercially available material containing alkoxylated alcohol. The results for these agents can also be seen above.

Summary

This working paper carries out an investigation into cracking in cementitous renders used to waterproof cheap hand-built water tanks in the developing world. A study of the theory behind cracking in mortar is followed by a review of readily available admixtures that affect the properties of mortar. Extensive experimentation has been carried out on these different mixes of mortar, with the result that the investigation suggests the use of a superplasticiser will reduce the cracking and hence the leakage in a mortar rendered tank. A further recommendation is to add silica fume to the mortar to increase its strength and help reduce cracking. Further investigation into the subject is also recommended.

Contents

Chapter 1: Introduction

1.1 Overview

In the developing world, many communities don't have access to a reliable water source, and as such have to travel long distances to find water. The Developing Technology Unit (DTU) in the University of Warwick is working on providing a source of water to such communities. This project is involved with the production of low cost water tanks to provide people with water.

Uganda is an example of where the tanks will be used, because it has high rainfall during some parts of the year and periods of drought at other times. This sort of climate is ideal to implement rainwater harvesting (RWH). Runoff water from rooftops can be collected and stored in a large tank next to the house. The tank itself can be either above-ground or below-ground and is constructed from local materials such as rammed earth. The defining factor in the production of these tanks is that they have to be cheap and therefore made from readily available materials from local sources. However, such materials are permeable and hence not suitable for storing water.

Waterproof renders consisting of a thin layer (10mm) of mortar, are applied to the walls of the tanks to allow them to store water. These cementitous renders are prone to shrinkage induced cracking, which causes leakage reducing the effectiveness of the tanks. This project is concerned with investigating and developing methods of reducing any cracking, and hence allowing more reliable water tanks to be constructed.

1.2 Project Aims

As stated above, the purpose of this project is to conduct a study of cementitious renders used to waterproof tanks used for RWH. This study will centre on investigating the amount of, and seriousness of cracks in various types of mortar based renders. The experimental side of the study will report on various measures of crack reduction available, and combined with an investigation into the theory behind crack development in cementitious materials, lead to conclusions on which mix of mortar is most suitable for the use described above.

Steve Turner, a graduate of engineering from the University of Warwick in 2000, had begun a similar study into leakage from waterproof renders in the summer before the commencement of this project. He had cast some mortar samples to experiment on, but was unable to carry on, and these samples were inherited and experimented on as an extension to this investigation. These samples provided an introductory look into cracking in cementitous renders, and Steve's notes are provided in the appendix, followed by results taken from the samples he prepared. These results are later used in the analysis and to draw conclusions. Chapter 9 illustrates problems with the procedure and highlights any alterations made to the design of test equipment.

Chapter 2: RWH Tanks

2.1 Description of tanks

Much research has been carried out by the DTU on the forms of water tanks to be used for RWH, so only a brief summary of the types of tanks is given in this report to familiarise the reader. There are 3 types of water tank that can be used for RWH:

- Above ground
- Below ground
- Overhead (roof of building)

The cheapest being below ground tanks as the surrounding ground provides support for the walls and therefore less emphasis must go into designing the tanks for strength. This project was undertaken with below ground tanks in mind although the principles developed can be applied to all types of tank.

The soil walls of the below ground tank are normally reinforced with rammed earth which can then be rendered. In readiness for the waterproofing mortar the tanks walls are scored to provide a good gripping surface on which to plaster. The mortar must be of a suitable consistency to allow plastering, not too thick, and not too runny. A capacity of 10,000 litres is average size for one of these tanks. Figures 2A below, taken from the DTU web site, show the excavation and completion of a partially below ground tank, PBG, combining the benefits of both designs.

Fig 2A

2.2 Pros and Cons

The pros and cons of the 3 types of tank.

	Pros	**Cons**
Above Ground	Helps prevent contamination from water run off. Easy to identify and repair cracks and leaks. Water can be extracted using a simple tap. Can be used in any environment regardless of soil types.	Expensive. Needs lots of free space. Weaker than below ground tanks. Must be designed to be strong enough to hold enough water. Easily damaged.
Below Ground	Cheap. Economical on space. Earth provides sidewalls so are very strong. Not easily damaged.	Hard to spot any cracks or leakage. Pump needed to extract water. Contaminated water could drain into tank. Dangerous to children and animals (should they fall in). Need stable soil conditions to prevent failure of sidewalls
Overhead	Increased water pressure due to head created from elevation. Economical on space. Easy to identify and repair cracks and leaks. Can be used in any environment.	Weaker than below ground tanks. Expensive. Must be designed to be strong enough to hold enough water. Failure of tank can potentially cause serious injury.

Chapter 3: Cement Theory

3.1 How Does Cement Harden?

Water is the key ingredient that causes cement to harden. The process by which cement powder combines with water to harden is called hydration. This process is when the major compounds in the cement react with the water to form hydrates. The water used is vital in determining the strength and end properties of the mortar. Cement is vulnerable to imperfections in additives and impure water can cause weak mortar. The water cement ratio is also important when mixing mortar. Too much water will result in weak mortar whereas too little will make it unworkable and not appropriate to use for many of the tasks in which it is employed. This will be discussed further in sections that follow.

3.2 Hydration

Hydration only occurs when the cement has access to moisture. Moist cement will hydrate and cure, but this process stops once the sample has dried out. This means that the strongest mortars are left to cure for a long period of time. This process can last months and even years. Amounts of water added to mortar, and the length of time it is wet for before drying out, are vital factors when considering the strength and usefulness of mortar. Portland cement has five major constituents. These are listed in the table below.

Cement Compound	% weight	Chemical formula	Alternative chemical formula
Tricalcium silicate	50	Ca_3SiO_5	$3CaO{\cdot}SiO_2$
Dicalcium silicate	25	Ca_2SiO_4	$2CaO{\cdot}SiO_2$
Tricalcium aluminate	10	$Ca_3Al_2O_6$	$3CaO\ {\cdot}Al_2O_3$
Tetracalcium aluminoferrite	10	$Ca_4Al_2Fe_2O_{10}$	$4CaO{\cdot}Al_2O_3{\cdot}Fe_2O_3$
Gypsum	5	$CaSO_4{\cdot}2H_2O$	N/A

All of these compounds undergo hydration when exposed to water, but only the calcium silicates contribute to the overall strength of the mortar. Tricalcium silicate reacts more quickly than dicalcium silicate, and so is responsible for most of the strength of the mortar after the first 7days of hydration. The manner in which each of the calcium silicates affects strength of mortar will be discussed individually.

3.2.1 Tricalcium silicate

Tricalcium silicate reacts rapidly with water to release calcium ions and hydroxide ions. The reaction is exothermic and therefore a lot of heat is produced. The chemical equation is given below.

Tricalcium silicate + water ⟶ Calcium silicate hydrate + Calcium hydroxide + Heat

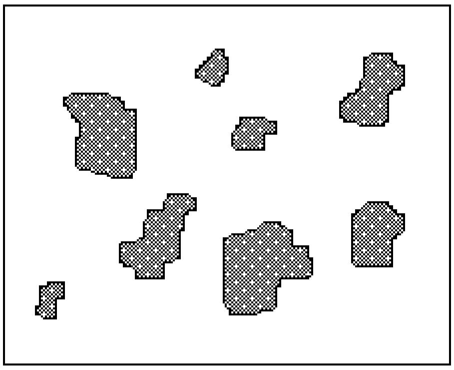
$$2\,Ca_3SiO_5 + 7\,H_2O \longrightarrow 3\,CaO{\cdot}2SiO_2{\cdot}4H_2O + 3\,Ca(OH)_2 + 173.6kJ$$

The Ph rises to over 12 due to the presence of alkaline hydroxide ions. The reaction continues over time, gradually producing more calcium and hydroxide ions until the effect is a saturation of these ions. Crystallisation of the calcium hydroxide now begins to occur, while at the same time calcium silicate hydrate crystals forms. The evolution of heat from the reaction increases due to *Le Chatlier's principle.* This is where ions precipitate out of solution, accelerating the reaction of tricalcium silicate to calcium and hydroxide ions.

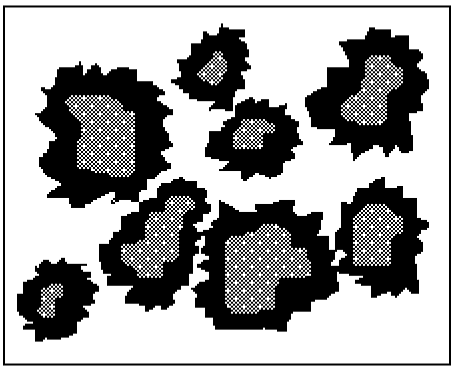
The formation of calcium hydroxide and calcium silicate hydrate crystals increases. The crystals act as a starting point for more calcium silicate hydrate to grow upon, and so they get bigger as further hydration takes place. This makes it harder for water to reach the unhydrated tricalcium silicate, and hence the reaction slows down. As further crystal growth continues the speed of the hydration reaction is constrained by the rate at which water can penetrate through to the unhydrated tricalcium silicate, so over time the production of calcium silicate hydrate becomes slower and slower. The diagram below (figure 3A) illustrates the process.

Figure 3A

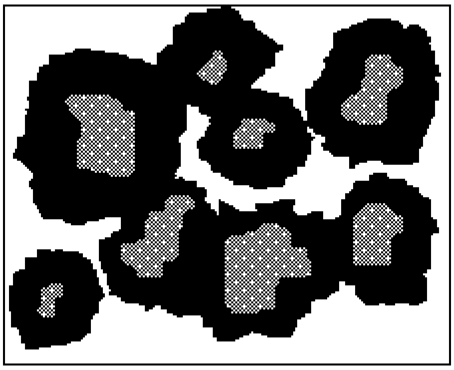
Hydration not yet occurred. Pores filled with water.



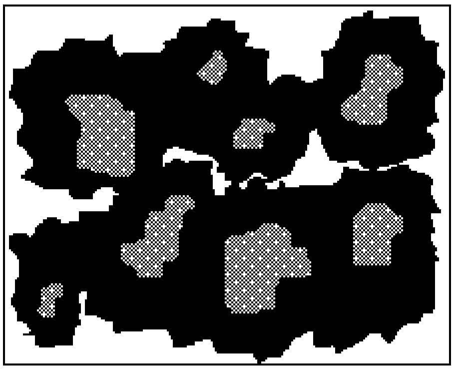
Beginning of hydration. Calcium silicate hydrate builds up.



Hydration continues. Spaces filled with water and calcium hydroxide



Nearly hardened concrete. Most space filled with calcium silicate hydrate. Remaining gaps mainly calcium hydroxide solution



3.2.2 Dicalcium silicate

Dicalcium silicate affects the strength of mortar much more slower than tricalcium silicate. It reacts with water in a similar way but is much less reactive and so less heat is evolved. The products of the hydration of dicalcium silicate are the same as those for tricalcium silicate, and are shown below.

Dicalcium silicate + water ⟶ Calcium silicate hydrate + Calcium hydroxide + Heat

$$\mathbf{2\ Ca_2SiO_4 + 5\ H_2O \longrightarrow 3\ CaO{\cdot}2SiO_2{\cdot}4H_2O + Ca(OH)_2 + 58.6\ kJ}$$

The production of calcium silicate hydrate and calcium hydrate occurs in a similar way as shown in the above diagram, but over a longer period of time.

3.2.3 Tricalcium Aluminate

The amount of tricalcium aluminate is relatively small but can have a significant affect on the properties of the hardening cement paste. The hydration of tricalcium aluminate can occur extremely fast which can lead to a phenomenon known as flash setting. This can occur because the reaction between tricalcium aluminate is very violent and can result in advanced hydration over a very small period of time (a few seconds). This is undesirable as it would cause premature setting of the cement mixture and makes it very difficult to work with. When the clinker first forms in the kiln there is nothing to stop flash setting of the material should it come into mortar with a small amount of water. Because of this gypsum is added as it suppresses flash setting. Gypsum is added to the clinker before the whole mixture is ground down to make cement paste.

3.2.4 Gypsum

When gypsum is added to the clinker, it reacts with the tricalcium aluminate to form calcium sulphoaluminate.

Tricalcium aluminate + gypsum $\longrightarrow$ Calcium sulphoaluminate

$$3CaO \cdot Al_2O_3 + CaSO_4 \cdot 2H_2O \longrightarrow 3CaO \cdot Al_2O_3 . 3CaSO_4 \cdot 31H_2O$$

A lot of heat is produced in the hydration of tricalcium aluminate and a rapid rise in the temperature of cement paste within five minutes of water being added hints that not all of the tricalcium aluminate becomes calcium sulphoaluminate, resulting in limited rapid hydration which explains the rapid rise in heat.

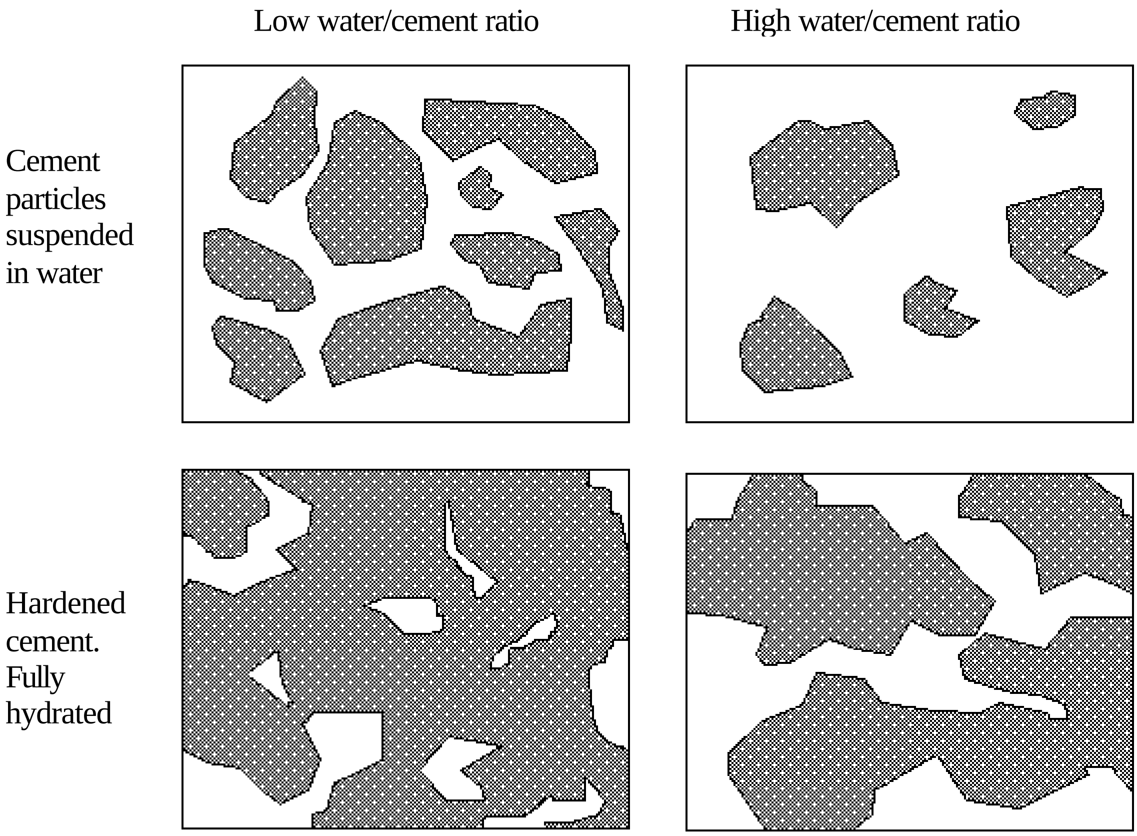
Chapter 4: Strength of Mortar

The strength of mortar is determined by 2 main factors.

1. The amount of water used (water / cement ratio).
2. The length of time for which the mortar is left to cure.

Total hydration requires an exact amount of water, much less than what is used in practice to add to cement. An excess of water is provided to increase the workability of the mixture and allow it to be worked into the desired position. Any excess of water not used up by hydration will simply remain in the mixture and reside in pores in the microstructure. Once the mortar dries, the water will evaporate out of the mixture leaving the pores empty. The more excess water used, the more will be left over after hydration has occurred and therefore the larger the pore volume will be. It can therefore be seen that the strength of mortar reduces as more water is used. If the amount of water used is much greater than that needed by hydration, the space taken up by pores in the microstructure will be relatively large and the porosity of the mortar will increase. This can be illustrated in the diagrams below (figure 4A).

Figure 4A

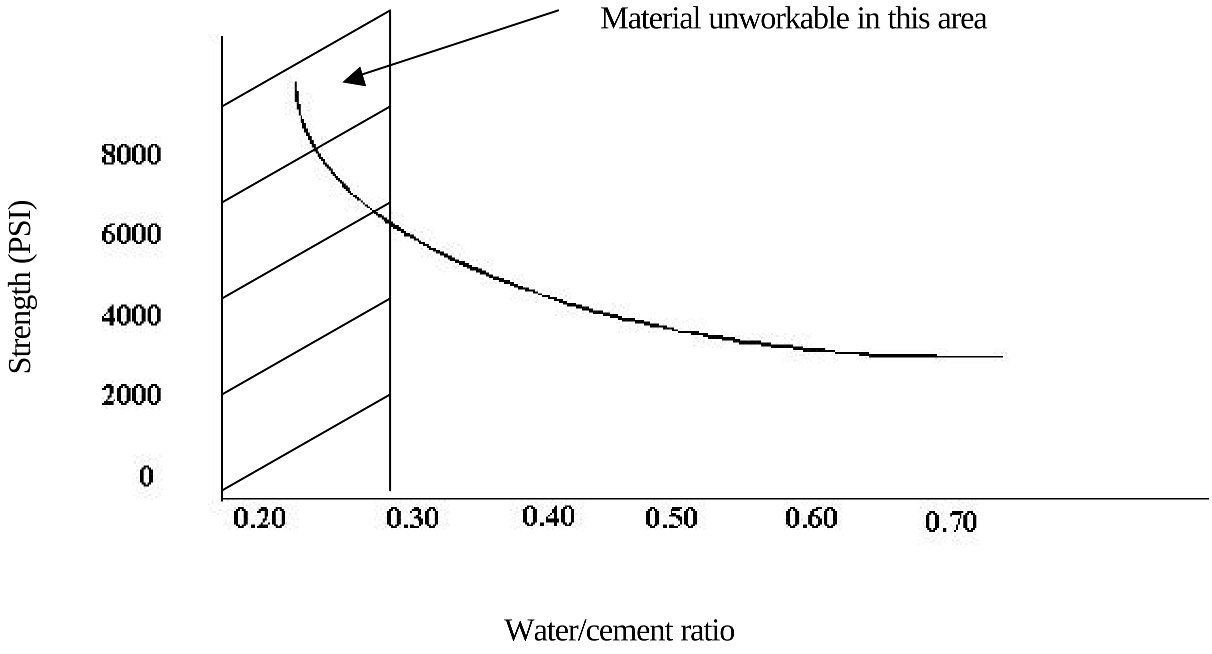


It can be seen from the diagrams above that the mixture with the lower water cement ratio has fewer pores than the high ratio mixture. Porosity of the cement/mortar is directly linked to its strength and the lower the porosity the higher the strength.

Achieving the theoretical maximum strength by using the exact amount of water for complete hydration is not achievable in reality as there will always be some pores present, even if the cement has been highly compacted. The trade off between strength of mortar and the workability desired to use the material depends on what task the mortar needs to perform. For casting in moulds it must be very liquid to allow pouring, but while this does increase workability it will result in weak mortar. For applications such as plastering, less water is used and the paste is much more viscous and will be stronger and less porous once set.

The graph below (figure 4B) shows how the water cement ratio affects the strength of the mortar produced. The graph highlights the limit of workability of concretes and mortars, below which it would be impossible to use the mix for any practical purpose.

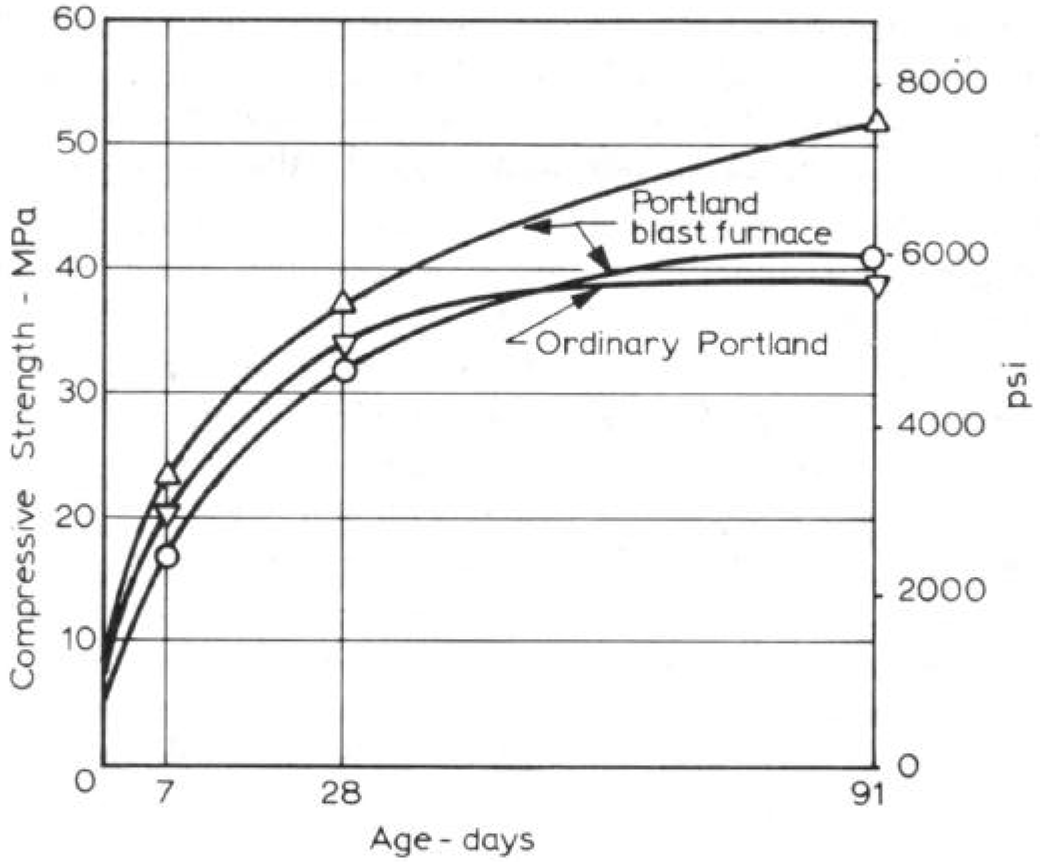
Figure 4B



The other factor that decides the strength of mortar is the length of time it is allowed to cure. Curing is the continuing process of hydration. It can take years for all of the calcium silicates to become hydrates and so the longer the mortar is left to cure the stronger it will be once dry. The graph on the following page (figure 4C) taken from “Properties of Concrete” by A.M.Neville, illustrates the manner in which different types of cement harden over time. The 7 and 28 day points have been plotted because they are commonly used indications of the strength of concretes and mortars. It can seen from the graph that even after 90 days the different types of cement are still increasing in strength, and many will do so for months afterwards, although at a very slow rate.

Figure 4C

0.6 water/cement ratio concrete



Chapter 5: Cracking in Mortar

5.1 Why does Mortar crack?

It has already been shown that mortar is a very complex material and many reactions take place within the cement to allow it to harden. A very common problem with the material is that over time cracks appear on the surface. Internal cracks are also common and in structures it is possible that cracks propagate unseen through the material for a long distance before finally breaching the surface. As the mortar is being used as a waterproofing agent in this investigation it is important to keep cracking to a minimum.

When cement powder is mixed with sand and water to form mortar it can be of various viscosities, but there is always a volume of water present which will be lost at some stage during the curing and drying processes. This loss of water changes the volume of the mortar and therefore the material shrinks. If the mortar is unconstrained then this change in volume is not a problem because the material will simply shrink with no damage to its properties. If the mortar is constrained during curing and drying, then it cannot change its volume as easily. Tension builds internally, and if this force exceeds the materials yield stress then it will crack. Mortar has a very low strength in tension compared to its strength in compression and so cracking occurs very easily in a constrained sample.

Theoretically, the strength of cement paste is much higher than those values actually achieved. The theoretical strength has been estimated to be up to 10.5Gpa, but this theory is based on perfect surface texture and internal structure. In reality, the material is not homogenous and there are many stress concentrations that are set up in the material. These concentrations allow very high local stresses to accumulate resulting in micro-cracking. Thousands of micro cracks are present in every meter squared of mortar but these do not cause any significant structural problems. Larger cracks of the order of 1mm or more, while not as common, represent a more significant reduction in the yield strength. These cracks can initiate from places such as the suspended aggregate, and any small imperfections in the material.

Cracking is caused by restraints acting against the shrinkage of the mortar, the tank wall in this case. Another form of restraint is non-uniform shrinkage within the mortar itself. When the paste dries, moisture is lost first from the surface and only later do the internal sections dry. This sets up a moisture gradient, which is part of what is called differential shrinkage. If a specimen dries in a symmetrical way this is not much of a problem, but in extreme cases warping can occur if the specimen dries in a non-symmetrical way. Because of the fact that the surface dries much quicker than the interior, size and shape of a specimen are extremely important factors in how much the mortar shrinks and to what extent it cracks.

Differential shrinkage should not be a factor considering the thickness of mortar used in this project. In theory the layer of mortar should be as thin as possible to counteract differential shrinkage, but considering the project is looking at renders with a thickness of the order of 10mm, the amount of shrinkage caused in this way will be negligible. The main consideration for this will be when the tank walls are left to dry. They should be left covered, with even heat distribution throughout so as not to allow one area of the tank to dry before the rest, which could cause cracking at the boundary between the two differently dried areas. Also, the walls should not be exposed to sunlight while drying, as this is likely to cause uneven heat distribution resulting in uneven rates of drying, and in turn a moisture gradient which could lead to cracking.

Chapter 6: Mathematical Model

Cracking in cementitious materials is progressive and as such can occur over long periods of time, although the majority of cracks initiate and propagate to nearly full length in the first month after drying begins. The effect a crack has on the material properties depends on its length, depth and width. When considering a cement based material for strength, it is found that the wider the crack the greater the resultant reduction in strength. This section will attempt to model how crack width affects the permeability of mortar. When considering a crack in concrete it is useful to make assumptions to make analysis easier. Two methods of analysis are shown in this section for different assumptions about how a crack can be modelled. They are shown below.

6.1 First Method

From fluid theory, flow through a crack can be approximated to flow through a tube (see figure 6A). Assume the pressure gradient, G, is parallel to the axis of the tube. For such an arrangement the forces can be derived as follows.

$$\text{Axial force} = \text{Gäë . 2ðr är} \qquad \textbf{1.}$$

$$\text{Viscous Shear force} = \text{2ðr äë . í .}\left(\frac{\text{dõ}}{\text{dr}}\right) \qquad \textbf{2.}$$

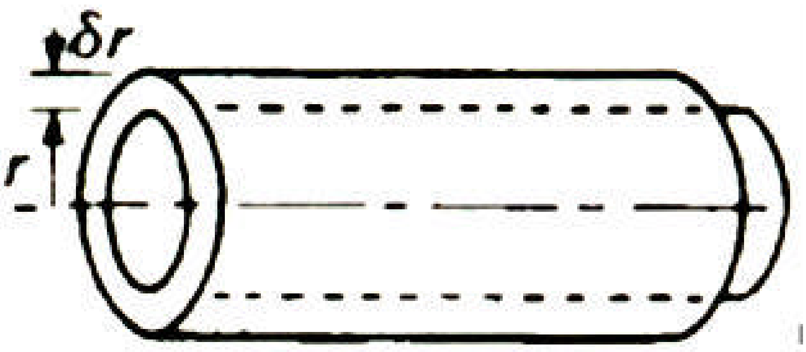
Equating 1 and 2 above produces the following expression for Gär:

$$\text{Gär} = \text{í}\left(\frac{\text{dõ}}{\text{dr}}\cdot \text{är}\right) \qquad \textbf{3.}$$

Substituting 3 for Gär into the equation for the viscous shear force yields the following:

$$\text{Viscous Shear Force} = \text{2ðr äë . í}\ \frac{\text{dõ}}{\text{dr}}\left(\text{är} + \frac{\text{d}^2\text{õ}}{\text{dr}^2}\text{är}\right) \qquad \textbf{4.}$$

Fig 6A



The laminar flow stresses $ô_{inner}$ and $ô_{outer}$ can be derived as follows and substituted into the force expressions in 1 and 2 on the previous page:

$$ô_{inner} = í\ \frac{dõ}{dr} \qquad \textbf{5A.}$$

$$\therefore\ F = äë \cdot 2ð r \cdot ô_{inner} \qquad \textbf{5B}$$

$$ô_{outer} = í\left(\frac{dõ}{dr} + \frac{d^2õ}{dr^2}\,dr\right) \qquad \textbf{6A}$$

$$\therefore\ F = äë \cdot 2ð(r + är)ô_{outer} \qquad \textbf{6B}$$

From 5B and 6B above the net force can be derived:

$$\text{Net Force} = 2ð d ë \cdot í + \frac{dõ}{dr} \cdot är + r\frac{d^2õ}{dr^2} \cdot är \qquad \textbf{7.}$$

To solve this differential equation set F to zero and reduce as shown below:

$$r\frac{G}{í} - \frac{dõ}{dr} - r\frac{d^2õ}{dr_2} = 0 \qquad \textbf{8.}$$

Solving this second order differential equation for the velocity distribution across the top of the tube of radius R yields the following:

Knowing that: $\frac{dõ}{dr} = 0$ at r = 0

õ = 0 at r = R

$$õ = \frac{K}{8}\left(R^2 - Rr^2\right) \qquad \textbf{9.} \qquad \text{Where } K = \frac{G}{í}$$

The mean velocity, $\bar{õ}$, can be found from 9 above by substituting the expression for õ in the equation shown below:

$$\bar{õ} = \frac{\int_0^R 2ð\, r\, õ\, dr}{ðr^2} \qquad \textbf{10.}$$

The result is shown below:

$$\boxed{\bar{õ} = \frac{K}{16}R^2} \qquad \textbf{11.}$$

The variable R, which represents the size of the crack, is of interest and equation 11 above shows that if the crack size R, has a square relationship with flow rate. An example is if a crack were twice the size, the flow would increase by 4.

6.2 Second Method

An alternative assumption of the form of a crack can be used to verify the above result. A laminar crack between two plates of width t, length b and thickness L. In this example the pressure gradient G = pressure drop/L.

$$\therefore \text{pressure force on layer } (F) = Gbdy \cdot äë \qquad \textbf{1.}$$

Shear force on bottom of layer = $\text{bäë} \cdot \text{í}\ \frac{\text{dõ}}{\text{dy}}$ **2.**

Shear force on top of layer = $\text{bäë} \cdot \text{í} \left(\frac{\text{dõ}}{\text{dy}} + \frac{\text{d}^2\text{õ}}{\text{dy}^2} \right) \cdot \text{dy}$ **3.**

Equating 2 and 3 above yields the following differential equation.

$$\frac{\text{d}^2\text{õ}}{\text{d}^2\text{y}} = -\text{G} \qquad \textbf{4.}$$

This can be solved in the same way as equation 8 in section 5.1 above. Using õ=0 at y=t/2 gives:

$$\text{õ} = \frac{\text{G}}{2}\left(\frac{\text{t}^2}{4} - \text{y}^2 \right) \qquad \textbf{5.}$$

The mean velocity, $\bar{\text{õ}}$, can be found from 5 above by the following integral.

$$\bar{\text{õ}} = \frac{\text{G}}{2} \int_0^{\frac{t}{2}} \left(\frac{\text{t}^2}{4} - \text{y}^2 \right) \text{dy} \qquad \textbf{6.}$$

Therefore:

$$\boxed{\bar{\text{õ}} = \frac{\text{G}}{2} \frac{\text{t}^2}{4} \left(\frac{2}{3} \right)} \qquad \textbf{7.}$$

6.3 Summary

Both the model for a tubular crack and the model for a laminar crack come to the same conclusion.

Mean velocity (flow per unit area of crack) $\propto t^2$

In the context of this investigation the result of the analysis is that: -

$$\textbf{Flow} \propto \quad \textbf{(crack length x crack width}^2\textbf{)}$$

IF the length of the crack is much bigger than the width (as should be the case)

The consequence of this result is that mortars should be designed to spread any shrinkage between many small cracks rather than few wide ones. The width is of great importance due to the fact it affects flow rate by a power law. Any small increase in crack width would increase flow rates dramatically.

Chapter 7: Admixtures

7.1 Overview

There are many different admixtures available on the market able to change many different properties of mortar, yet the most popular two types improve the material properties in the following areas:

1. Strength
2. Watertightness

These two factors are of interest to this project as they both can affect the mortars suitability as a render under the conditions set out in section 2. As highlighted, cracking is the major cause of leakage and cracking only occurs if the internal stresses in the mortar exceed its yield strength. Increasing this strength will reduce the amount of cracking for a given internal stress, thereby reducing the leakage. The usefulness of the second point is rather more apparent, although the manner in which admixtures that claim to improve watertightness do so is likely to be closely related to the strength of the material.

7.2 Tested Admixtures

The admixtures used in this investigation were inherited from previous research carried out by the DTU, and all satisfy the above criteria of being cheap and readily available in the developing world. The four admixtures are listed below.

1. Silica Fume
2. Superplasticiser (complast 211)
3. Harilal Leak seal
4. Festegral

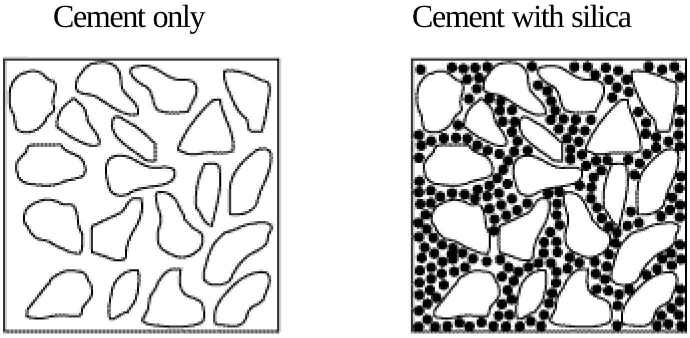
Another product was available for testing, which was a slurry-based layer that was to be sandwiched between two layers of plain mortar. This product, ferrofest, claimed to reduce shrinkage when used in this way, and hence reduces cracking and improves the water tightness of the mortar. Where available, an analysis of the theory behind each of the admixture's claims is given below.

7.2.1 Silica Fume

Silica Fume is created by heating quartz, coal, iron and wood at 1800^{o}c and collecting tiny particles from the emissions. These spherical particles have a diameter of approximately 0.1 microns (of the order of 100 times smaller than cement particles). Silica Fume increases the strength of concrete mixes and is used worldwide in all types of application, and hence is very readily available.

The silica particles, being so small, are able to fill spaces between cement grains and so displace excess water and act as nucleation sites for hydration to begin. This is known as the microfiller effect, and results in reduced porosity of concrete (or mortar as in this case) and hence it is stronger. Figure 7A below illustrates this effect.

Fig 7A



Another effect of silica fume that adds to the mortar strength is the pozzolanic effect. The amorphous silica particles have a very large surface area due to their small diameter and react with the calcium hydroxide in the cement to form calcium silica hydrates, which are the hydrate products found in hardened cement. This increase in the amount of hydrates adds to the strength of the material.

7.2.2 Superplasticiser

Plasticisers are used in concrete to reduce the amount of water needed to reach a required workability. In a normal concrete mix, cement particles tend to agglomerate, trapping mix water that would otherwise be used for lubrication. When superplasticiser is added to the mix, it is absorbed onto the cement particles causing electrostatic repulsion and dispersing the cement particles evenly throughout the concrete mix. The result of this is that water is not wasted because it is being more effectively used for hydration, and hence lower water:cement ratios can be used to achieve the same workability of mix as when the superplasticiser is not used. The reduction in water in turn increases the strength of the cement.

7.2.3 Harilal Leak seal

There is no indication on the packaging of this Indian admixture as too how it affects the permeability of cement.

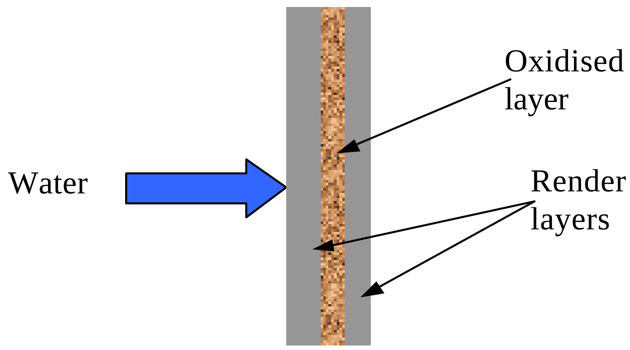
7.2.4 Festegral

There is no indication on the packaging of this Mexican admixture as too how it affects the permeability of cement.

7.2.5 Ferrofest

Ferrofest is not an admixture, and instead of being mixed in with the rest of the mortar ingredients, is sandwiched between two layers of plain mortar. It is iron based and claims to reduce the effects of shrinkage in concrete by expanding to counteract the shrinkage. During wet curing of the mortar, the iron within Ferrofest oxidises causing the layer to expand. These oxidised particles will clog up the pores in the plain layers and the associated expansion will help close any cracks formed in either layer of plain mortar, and therefore should reduce permeability. Figure 6B below illustrates the application of Ferrofest.

Fig 7B



The proportions of each admixture were provided by the manufacturers and are shown in figure 6C below.

Figure 7C

Sample	Admixture % (cement weight)
Harilal Leak Seal	2
Festegral	4
Silica fume	10
Superplasticiser (Complast211)	0.8
Ferrofest	100

Chapter 8: Variables

Three variables have been identified as appropriate for investigation in respect to waterproofing an underground water storage tank with mortar. Section 5 showed how upon curing and drying, mortar is prone to cracking. Because this is the root cause of any leakage from the tank, the variables chosen for investigation are all related to how much the mortar will crack and the resultant effects. They are as follows.

1. Shrinkage
2. Cracking
3. Leakage

Shrinkage – As the mortar dries, the associated water loss causes a change in volume that will be measured experimentally to determine the amount of shrinkage in each specimen of mortar.

Cracking – Cracking occurs in samples that undergo constrained shrinkage, and hence internal stresses build up causing crack initiation and propagation if the mortar's yield strength is surpassed.

Leakage – The purpose of the leakage experiment will be to determine what effects crack size has on water loss. Ultimately this will lead to conclusions stating whether it is better to have shrinkage accounted for by few large cracks or many small cracks.

Chapter 9: Experimental Procedure

The mortar must be of a suitable plastering viscosity, so a water cement ratio of 0.6 was used as a standard. All tests mentioned below use this ratio with the exception of the superplasticiser, which uses a ratio of 0.5 for reasons explained earlier.

9.1. Shrinkage

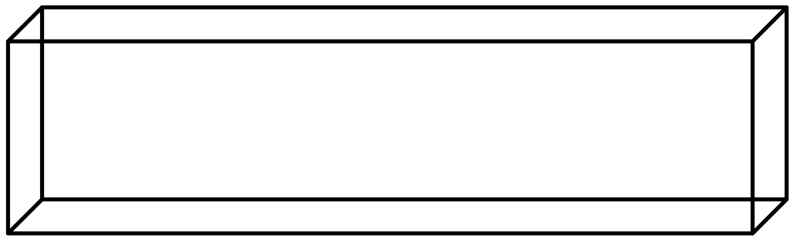
This variable proved very difficult to measure accurately as there are two possible means of shrinkage that occur at different stages. The manner in which the experiments were undertaken will be shown and then the associated problems discussed.

9.1.1. Procedure

Many methods of experimentation on concrete have been done in the past resulting in standard tests that the industry recognises and that are established as the way in which experiments with concrete are undertaken. British Standards (BS) 1881: part 5 (1970) and BS 2028 (1968), are two such standards relating to shrinkage measurement in concrete. However due to equipment and time factors, these standards could not be applied to this set of experiments, although they are of a very similar nature.

Firstly mortar was cast into blocks of dimensions 50 x 50 x 225mm and left to cure for a measured period of time. Once hardened (24 hours) small areas of the surface were dried using acetone and two metal tabs bonded to the surface of two opposing faces. These tabs were at a distance apart, which decreased as the blocks underwent shrinkage. The strain is then measured between the tabs using a dial gauge with an accuracy of 0.2 micro strain. The blocks were left to dry end-up so as to expose the greatest surface area to the air, to get even drying, thus helping to prevent warping. Two opposing sides were measured to monitor any possible warping that may occur during drying (see fig 9A).

Figure 9A



The amount of time the blocks were left to wet cure (no drying) was a variable investigated in the first round of experiments. In the first round three identical plain mortar samples were cast and left to cure in wet conditions for different lengths of time (2 days, 4 days and 14 days) to see what effect, if any, this would have on the shrinkage observed upon drying. Subsequent tests involving the admixtures would use a period of 2 days wet curing and results taken during 28 days of drying.

Each admixture used would be tested for shrinkage, and these results compared to the results for the other variables under investigation, cracking and leakage, to see if there was any correlation. See section 10.1 for breakdown of exact experiments that will take place.

9.1.2. Limitations

The main problem with doing experiments on the shrinkage of mortar is that at the time of casting the material is viscous and will flow, and only hardens to a point where it can removed from the mould after 24 hours. Any reduction in volume during this period is extremely difficult to measure and was not attempted in this investigation due to the said problem. Because of this, the results obtained for the shrinkage of the mortar are only for the period subsequent to the metal tabs being bonded to the samples, about 24 hours. Any shrinkage during the setting and early curing processes is unknown.

It would clearly be preferable to have values for the exact shrinkage or associated volume change from casting through to dryness, but this is not possible in this investigation. However, observation of the samples prepared by Steve Turner back up the opinion noted from literature on the subject, that shrinkage during wet curing accounts for only a tiny fraction of overall shrinkage when compared to the drying process. The samples were cured for three months underwater before the onset of this investigation, and upon initial inspection had no signs of any cracking on the surface. Once the samples were left to dry, extensive cracking was noted after 1-2 weeks.

Due to the observations made on the samples it is assumed that the shrinkage during the period of curing is negligible.

9.2. Cracking

9.2.1. Procedure

The purpose of investigating cracking in samples of mortar is fundamental to the overall aims of the project. It ties in with the other two variables because it is the shrinkage that causes cracking, and the cracking that causes leakage. Cracking will occur in a sample of shrinking concrete/mortar if the sample is constrained and not allowed to shrink unhindered (see section 5).

Steel rings are to be used as the constraint in this experiment. Mortar is to be applied to the rings in a layer 10mm thick between two retaining clips at the top and bottom of the cylinder (see fig9B). The rings have the following dimensions:

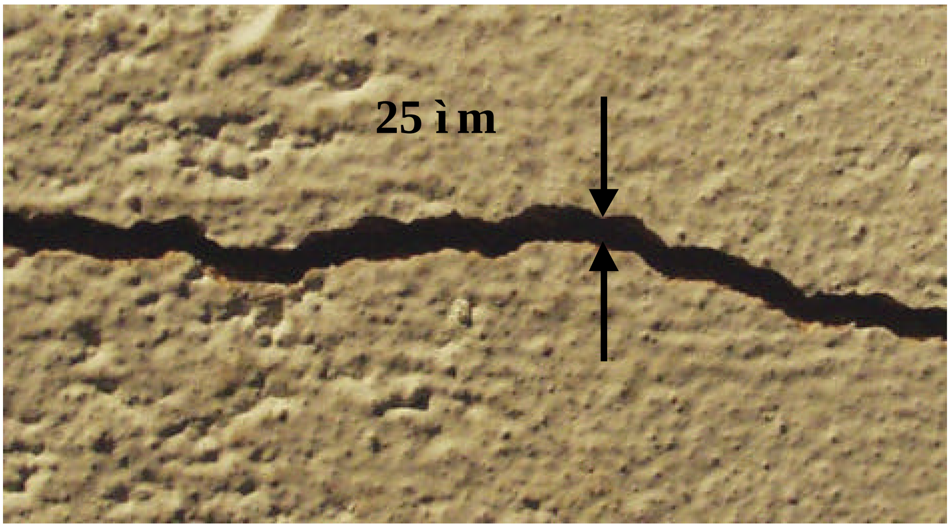
- Diameter = 170mm
- Height = 140mm
- Wall thickness = 5mm

Fig 9B

On application the mortar will adhere to the surface of the ring and harden around it. As the mortar layer dries, it will shrink and this movement will be constrained by the ring, resulting in internal stresses in the layer of mortar and eventual cracking. Cracks should begin to initiate towards the end of the first week and will have propagated significantly by weeks 1-2. The specimens will be left to dry and crack for 28 days, the same testing length as the shrinkage experiments.

Regular checks are made on the mortar covered rings to watch for signs of cracking or propagation of existing cracks. The width of any crack is vital as to determining the effect it has on the overall structure and flow rate of leakage water, so this is measured using a microscope with a lens calibrated with divisions every 50ìm allowing measurements to made in 25ìm increments. Fig 9C below illustrates a crack seen on the surface of a specimen prepared by Steve Turner prior to the commencement of this project.

Fig 9C



9.2.2. Limitations

This experiment only registers cracks that have broken the surface of the mortar and would not account for any cracking beneath the surface. However, any leakage that may take place would require a surface breaking crack to allow the water to escape. For this reason sub-surface cracking is not an important factor in this investigation as it is to do with leakage, but such cracks must be considered because over time they will eventually propagate to the surface and allow leakage.

When considering the use of mortar as a render for water underground water storage tanks, it is desirable to have the tank permanently filled or at least have some moisture present to keep up the relative humidity in the tanks, as the more time it is left empty to dry, the greater the likelihood of cracks emerging. It has been noted the specimens investigated, cracks generally appeared after 1-2 weeks so if the tank was left dry longer than this period, then extensive cracking is likely.

Another factor to consider is the rate at which the mortar dries, and therefore the rate at which it shrinks and what relationship this has to cracking. The quicker mortar dries, the greater the likelihood of cracks initiating. The reason for this is that creep plays a part in relieving the internal stresses that build up inside the material due to constrained shrinkage. If the samples dry slowly then the material will creep and result in reduced cracking compared to a sample that was dried very quickly. It is for this reason that the relative humidity of a near empty water tank must be kept high by covering it with polythene (for example), because if humidity were low, the moisture would be able to leave the surface of the render with greater ease.

9.3 Leakage

9.3.1 Procedure

The leakage experiments have been designed to run in conjunction with the cracking experiments. The steel rings as shown in fig 9B in section 9.2.1 were modified with a spiral groove run from top to bottom. This groove was `V` section and, due to warping in the rings, had a varying depth of between 2-3mm. The depth was set at a minimum of 2mm to ensure it could not be blocked with mortar, as it had to act as a channel for water to flow through. The pitch of the spiral was set to 48mm. The reason for this is that the channel is designed to be in contact with every crack in the mortar to feed each with water and allow leakage through them. The minimum crack length noted from samples prepared by Steve Turner was 50mm after one month and as these samples were to be left for the same amount of time and would generally be weaker because of the reduced curing, it was thought that no crack in any of the samples would be beneath the 48mm in length that the lathe was capable of machining.

Once any mortar was applied to the rings, there was no way of telling whether the channel was blocked and so it was decided to test various measures to stop it getting blocked.

Two methods were devised and tested prior to any of the experiments taking place. The first involved placing a length of string in the channel. This would act to stop the channel getting blocked and also acted as a wick to help draw the water down along. The second was to place thick wire in the channel which after consideration was to be crimped so as to stop it from sealing the channel and stopping the water feeding any cracks. Fig 9D below illustrates how the metal may seal the channel and prevent the water feeding the cracks.

Fig 9D

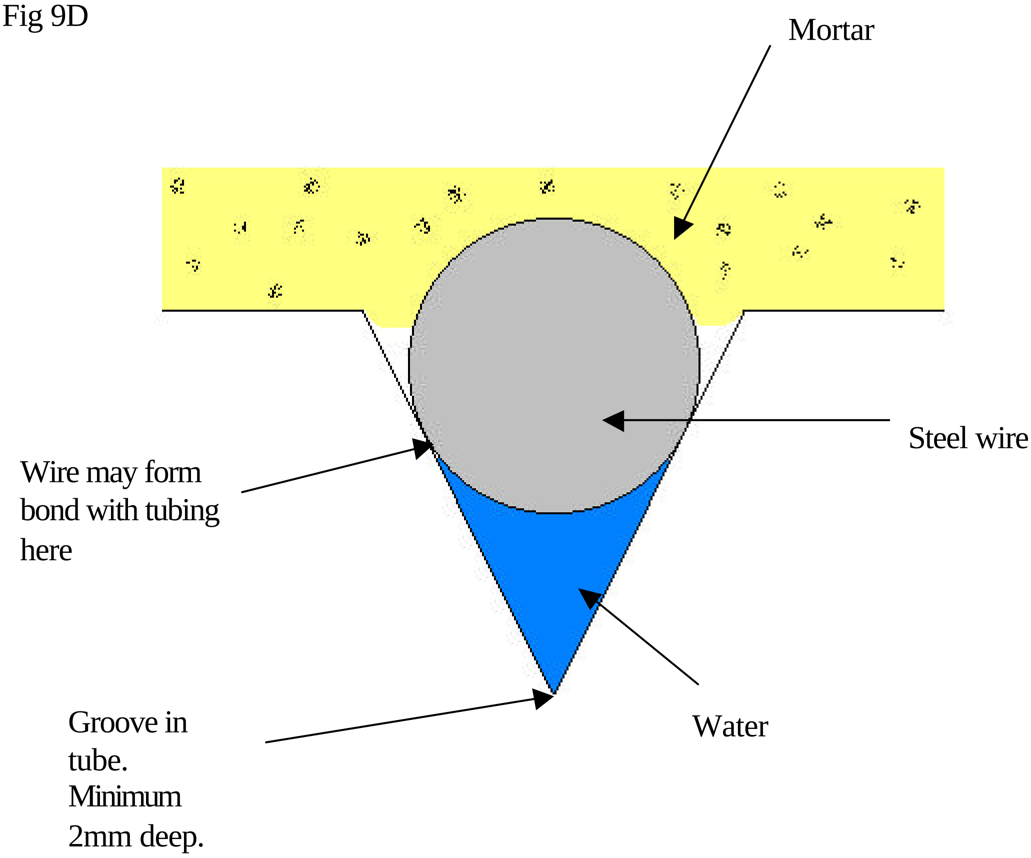
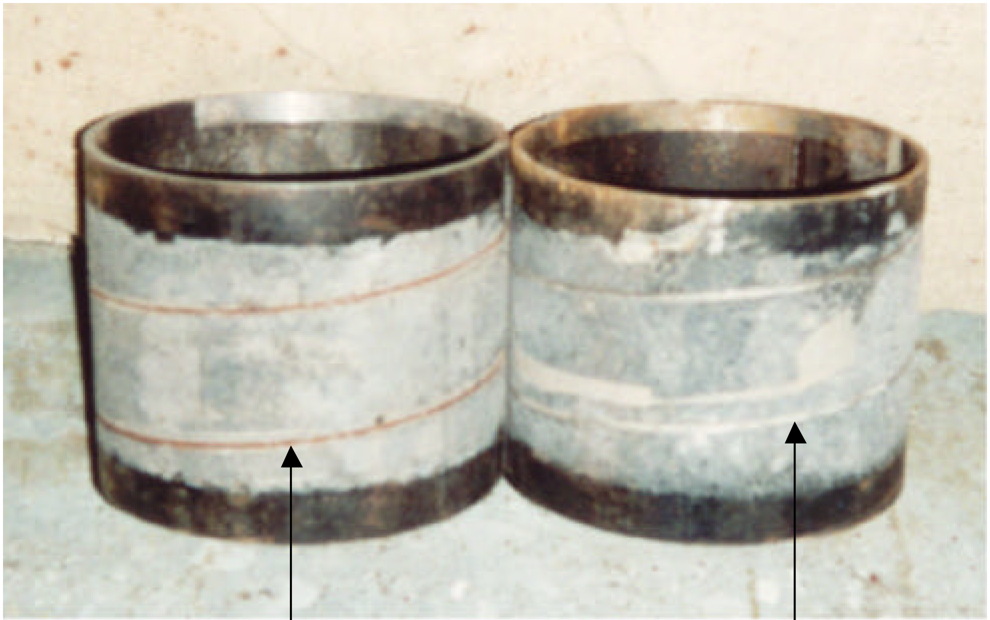


Figure 9D illustrates the way in which the two materials would prevent the channel getting blocked. Crimping the metal would prevent it sealing the channel and the string is porous and doesn't have a smooth surface finish so wouldn't seal the channel as the metal could.

Both of these methods were tested and any problems with their implementation noted. The string was exceptionally easy to lay in the crack and was simply taped at either end and posed no problem when plastering the steel in mortar. Because the steel wire was quite thick, it was very difficult to bend into shape and keep in the channel. It was carefully taped in place until the two ends could be fixed, but on removal of these tapes small sections of the channel were open. The results of two tests on each method were that the string kept the channel open and in both cases water flowed through the length of it. The wire method allowed the channel to get blocked on one occasion and so it was decided to use the string.

Once the mortar was removed and the lengths of string and wire removed, the channel was checked to see what state it was in. Both channels in which the string was used had a layer of rust along their lengths, showing that water did reach all areas of the channel, see fig 9E. The blockage in one of the wire samples was found to be caused by the wire coming out of the channel and it being blocked with mortar.

fig 9E

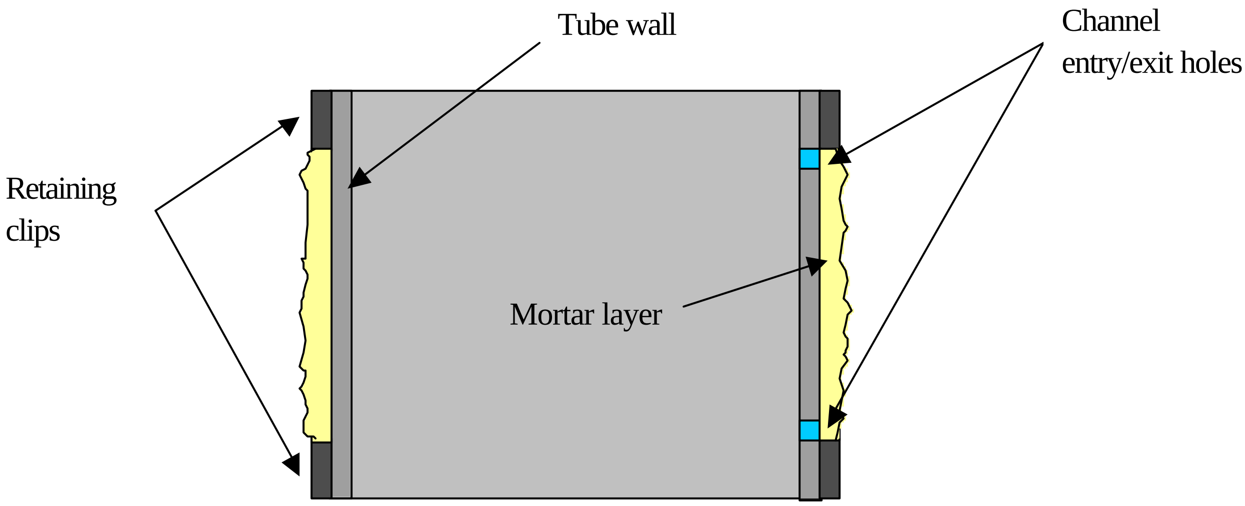


String sample
Rusted areas

Wire sample
No rust

2.5mm diameter holes were drilled 25mm from the edges of the rings in the groove to provide a means of feeding the channel water. The channel was then sealed upstream of these holes to stop water leaking out through the top of the rings. Hollow tubing was bonded over the holes on the inside of the rings to allow rubber tubing to be fixed on and connected to a supply of water which comprised of a glass tube calibrated at 1ml intervals. Once this was full and attached to the groove, the samples were observed to see how much leakage, if any, occurred. Fig 9F below shows a cross section of plastered the steel tube, with retaining clips top and bottom, and feed points to the channel.

Fig 9F



The glass tube used to supply the groove was positioned with the water level 1.5m above the sample to provide a "head" of water to help drive water through the cracks. As each sample leaked the level of water in the tube dropped and so too did the head, which would result in less water pressure on the cracks, so after every measuring interval, the water was topped up to the zero point 1.5m above the base of each sample. The amount of water lost in each time interval was approximately equivalent to only 4cm or 2.7% of the total head.

In all of the experiments on this variable, the aim was to find the steady flow rate of water through any cracks. Expected results would show that there would be an initial period of instability where the flow rate changes from an initially very high level. This can be accounted for by the time needed for the groove to be filled with water, the string to become fully saturated and for the cracks to fill with water.

The full flow rates for each sample are included in the results to illustrate this point, but it is the steady flow rates that are of most interest. From the first leakage sample it can be seen that the flow takes approximately 7 minutes to settle to a steady level. To take into account any differences in the samples, readings were initially taken every minute for the first 10 minutes while flow rates are high, and then every 5 minutes for a further 45 minutes. Flow rates are calculated by recording how much water has leaked out of the samples over the test period (1 or 5 minutes) and using this value to find the total volume of water that would leak out over an hour period.

9.3.2. Limitations

The experiments using the steel rings worked well for studying crack propagation, but problems were encountered when the leakage experiment was set up. There was inadequate sealing between the layer of mortar and the retaining clips, and due to the close proximity of the channel entrances to the boundary between the clips and the layer of mortar, water quickly leaked out. The manner in which this happened can be seen in figure 9F below, which is a photograph of the equipment and clearly shows water leaking from the top of the retaining clips.

Fig 9F

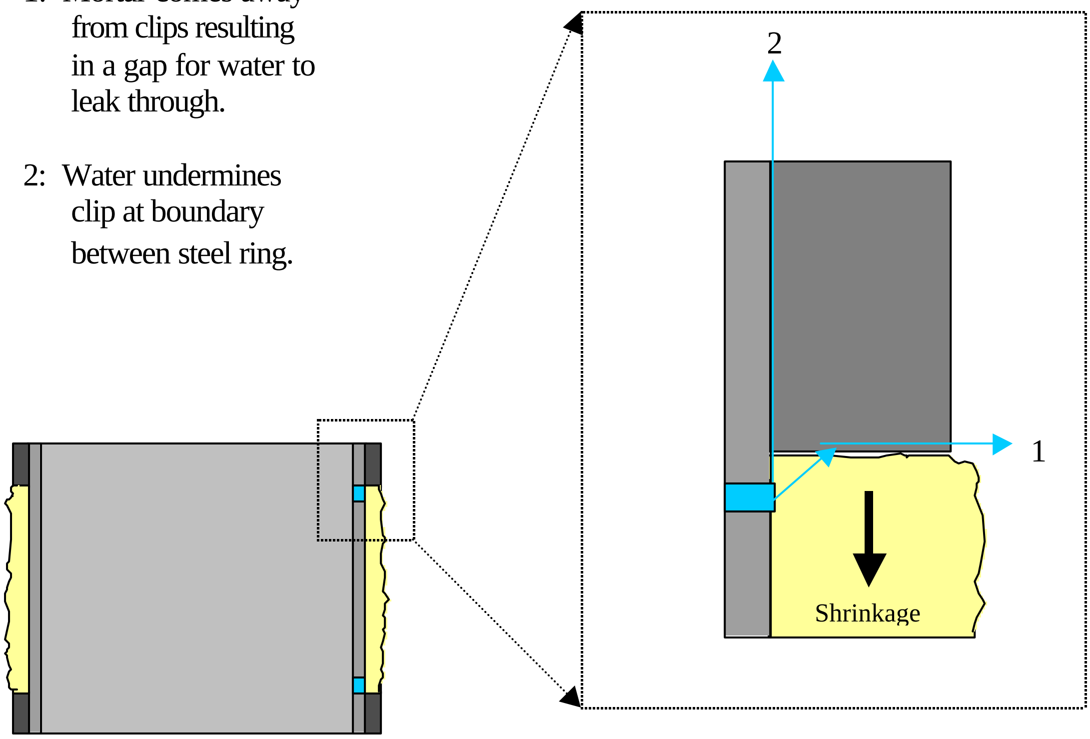
Upon manufacture of the samples, care was taken to ensure the boundary between the clips and the mortar was filled, but due to shrinkage along the axis of the cylinder, the mortar came away from the clips making it easy for water to drain out of the channel. Figure 9G on the following page shows diagrammatically how the experiment failed.

The net result of this leaking was that the samples had to be recast taking into account the means of failure of their predecessors. It was decided that to prevent the clips coming away from the mortar at the boundary between the two, that the clips would be enclosed in mortar themselves. The clips were also raised 5mm to increase the distance from the water feed holes. Once the samples dried and it was time to test for leakage, the added measure of sealing all exposed boundaries with silicone sealant was implemented. The boundary between the clips and the tubing was sealed to prevent any water undermining the clips and, the boundary between the mortar and clips was also sealed. Now any shrinkage of the mortar will result in it pulling into the clips rather than away from them, making the experiment less vulnerable to leaking.

Fig 9G

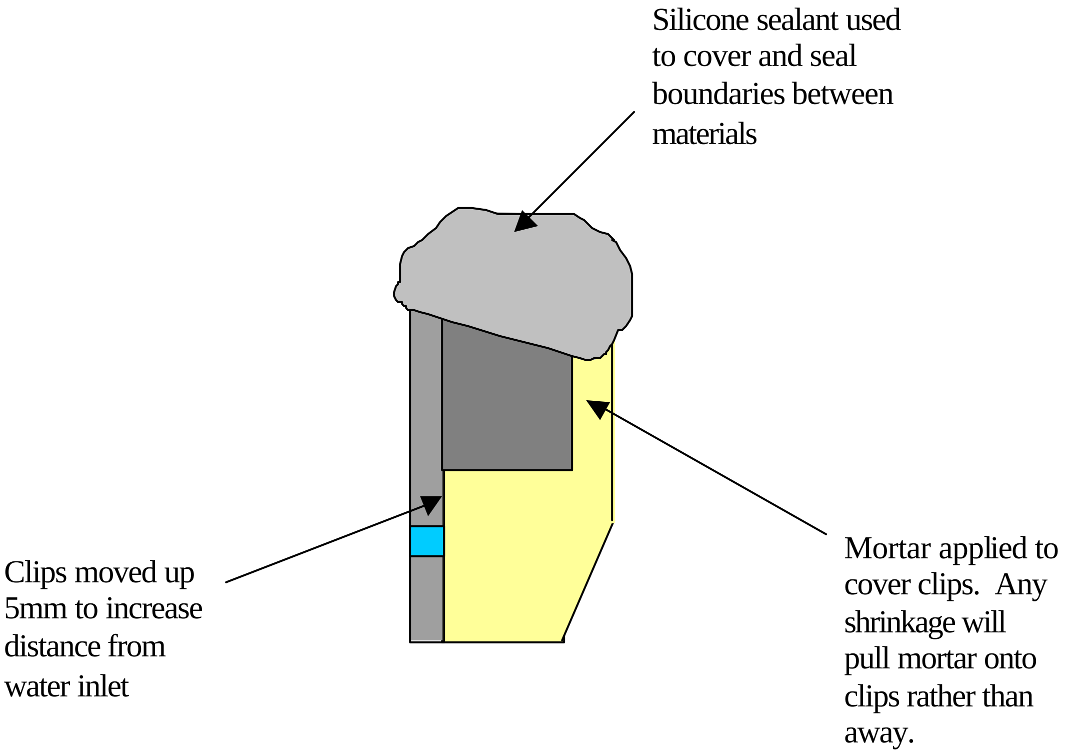
1: Mortar comes away from clips resulting in a gap for water to leak through.

2: Water undermines clip at boundary between steel ring.



The diagram at the top of the following page, figure 9H, illustrates the changes in the design of the experiment.

Fig 9H



Due to the fact that the first round of samples had to be recast, a month of experimental time was lost, and the time constraints on this investigation made it not possible to conduct a second round of experiments. This means that the only 4 samples investigated for cracking and shrinkage were plain mortar, silica fume, Harilal Leak Seal and Superplasticiser Complast 211. Had there been time for a second round of experiments, another 4 samples could have been cast consisting of a ferrofest sample, a sample with a layer of pure cement paste sandwiched between 2 layers of plain mortar, and a further 2, possibly investigating the uses of multiple admixtures in each sample.

Chapter 10: Summary of experiments

10.1 Shrinkage

1st round – 4 blocks cast. 3 plain mortar (no admixtures), 1 pure cement paste (no sand). 3 plain blocks left for various lengths of time to wet cure, 2 days, 4 days, and 14 days respectively. Pure cement left for 2 days wet curing.

2nd round – 3 blocks cast, 1 using silica fume admixture, 1 using superplasticiser admixture, 1 using Harilal leak seal admixture. All left for 2 days wet curing.

Each block had its shrinkage monitored regularly for 28 days after drying began.

10.2 Cracking

1st round – 4 rings cast. 1 plain mortar, 1 with superplasticiser admixture, 1 with silica fume admixture, 1 with Harilal leak seal admixture.

Each sample was left to cure for 2 days before drying began. Samples left to dry for 28 dries and cracks monitored.

10.3 Leakage

1st round – 4 rings cast. 1 plain mortar, 1 with superplasticiser admixture, 1 with silica fume admixture, 1 with Harilal leak seal admixture.

Each sample began testing after having been dried for 28 days

Chapter 11: Previous Investigations

As mentioned in the introduction, Steve Turner began experimentation into cracking in waterproof renders before this project began - his notes can be found in the appendix as can results taken from the samples he prepared. The samples talked about in the included documents were of similar design to the rings cast in the main investigation experiments into cracking and leakage. The main difference was that the plain, ferrofest, and nil coat samples, were cast on larger sized steel tubing, a practice dropped for the reports own experiments to keep the procedure constant.

The leakage experiment was never carried out on these samples because it was decided that the chance of all of the cracks lining up with the holes in the tube were small. It was thought necessary to be sure that all cracks were fed with water to ensure correct leakage rates were measured, which is why it was decided that the rings be modified with a spiral groove, of a pitch that was no less than the length of the smallest crack in the specimens studied prior to commencing the investigation.

Results on the cracking of these samples are included for comparative purposes, although changes in the experimental design restrict their use somewhat for this purpose. The manner in which the render shrinks makes it vital that, for experimental purposes, there is no room for an element of unconstrained shrinkage. The gauze was thought to reduce the steel ring's suitability as a constraint and may allow for some unconstrained shrinkage to be present.

To summarise, results from the samples prepared by Steve Turner, are included in the appendix and are referred to and considered in the analysis, although changes in the design of these experiments restrict exact comparisons being made.

Chapter 12: Shrinkage Results and Analysis

12.1 Shrinkage Results

The results for the set of shrinkage experiments are displayed over the coming pages, including graphical analysis. All numerical references to strain are as read from the equipment (x10^{-2} ì strain) from this point forward.

Table 12A and Graph 12A – Results for plain mortar with 2 days wet curing
Table 12B and Graph 12B – Results for plain mortar with 4 days wet curing
Table 12C and Graph 12C – Results for plain mortar with 14 days wet curing
Table 12D and Graph 12D – Results for pure cement paste with 2 days wet curing
Table 12E and Graph 12E – Results for Harilal Leak Seal with 2 days wet curing
Table 12F and Graph 12F – Results for Silica Fume 2 days wet curing
Table 12G and Graph 12G – Results for Superplasticiser Complast 211 with 2 days wet curing

Table 12A

PLAIN MORTAR 2 DAYS WET CURING		
Number of Days	**Side A**	**Side B**
-2	0	0
-1	2	2
0	1	2
1	-24	-25
2	-33	-33
3	-47	-43
4	-57	-53
5	-66	-61
8	-92	-83
9	-94	-86
10	-97	-89
11	-102	-94
12	-104	-95
19	-117	-112
25	-123	-117
26	-128	-121
28	-131	-125

Graph 12A

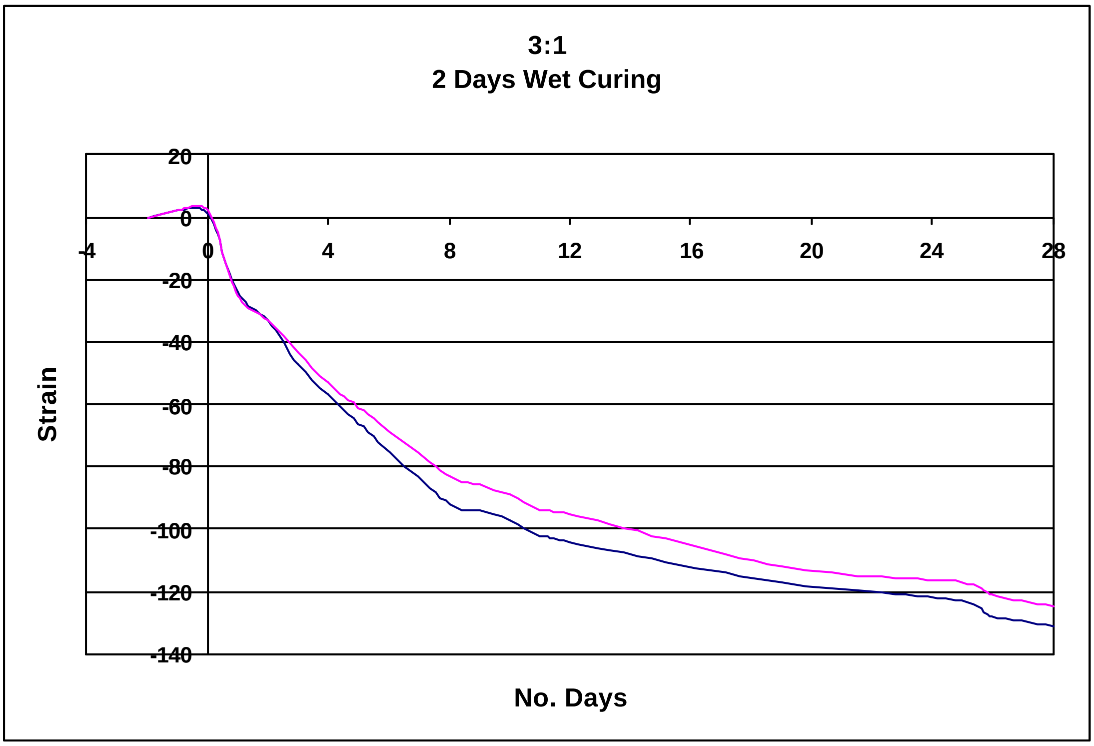


Table 12B

PLAIN MORTAR 4 DAYS WET CURING		
Number of Days	**Side A**	**Side B**
-4	0	0
-1	2	10
0	-7	-1
1	-16	-13
2	-25	-22
3	-33	-31
6	-53	-52
7	-58	-55
8	-62	-60
9	-69	-67
10	-72	-68
16	-95	-93
23	-108	-107
24	-111	-110
28	-115	-116

Graph 12B

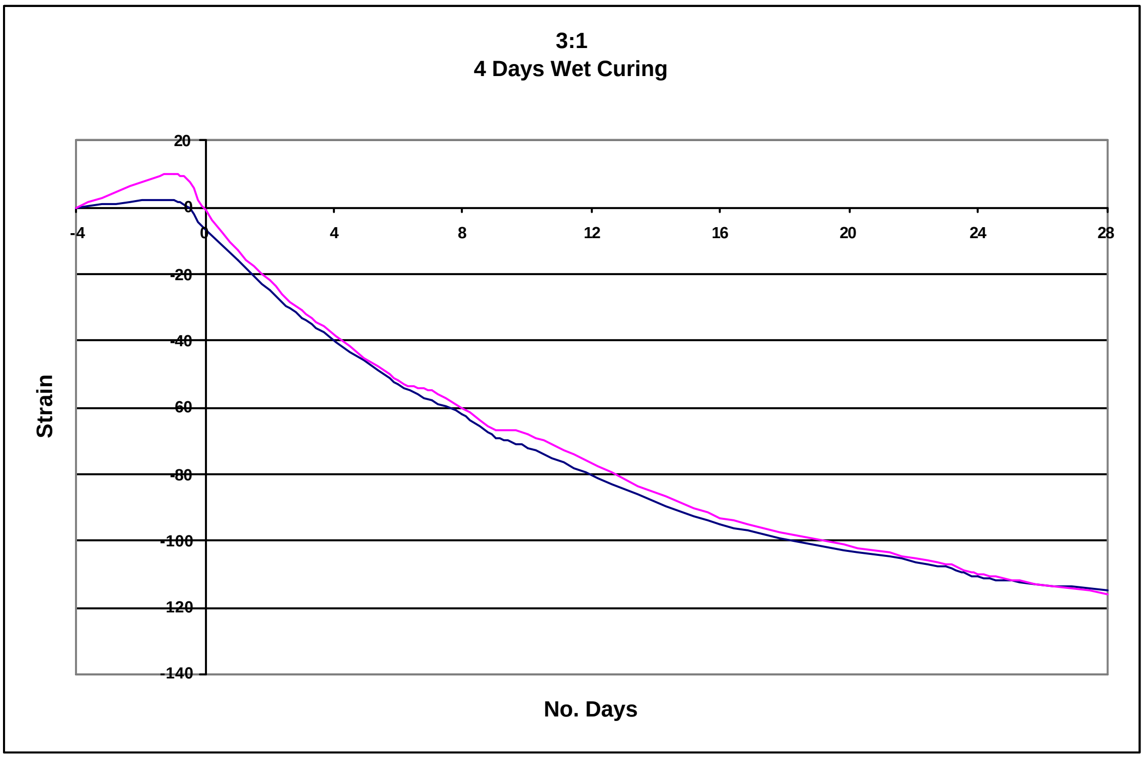


Table 12C

PLAIN MORTAR 14 DAYS WET CURING		
Number of Days	**Side A**	**Side B**
-14	0	0
-11	10	9
-10	12	10
-9	11	10
-8	12	9
-7	12	10
-4	11	9
-3	12	10
-2	12	10
-1	11	10
0	11	10
2	0	-2
5	-35	-34
6	-42	-39
8	-52	-50
13	-73	-68
14	-77	-72
17	-84	-79
20	-90	-84
22	-92	-87
26	-96	-91
28	-98	-94

Graph 12C

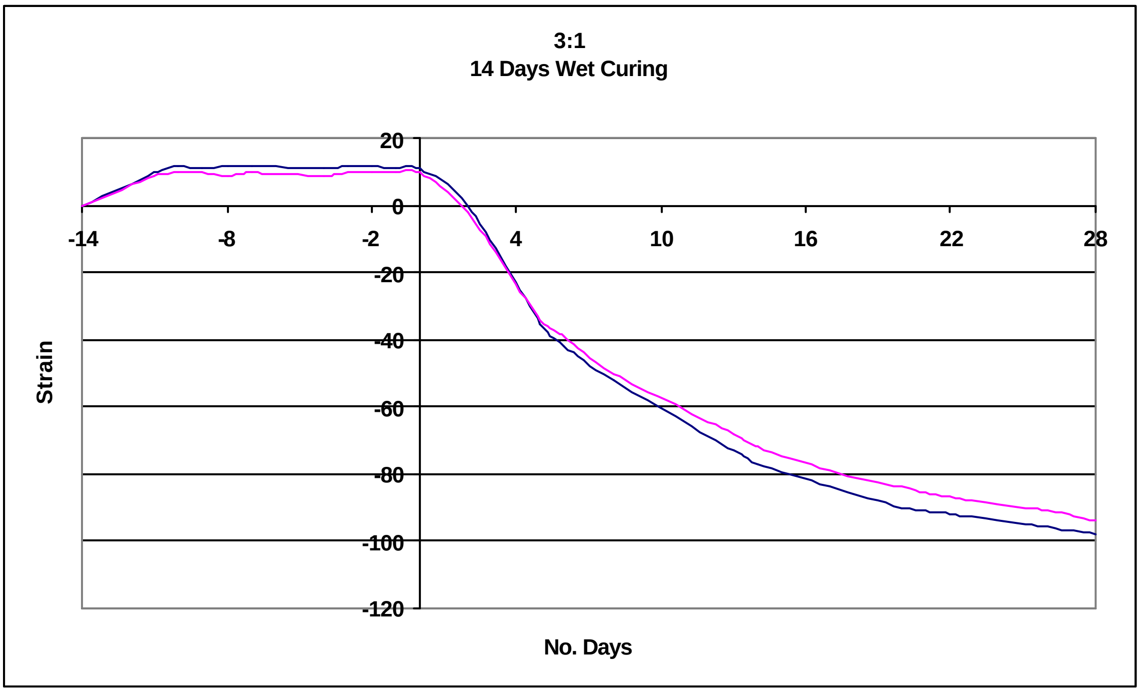


Table 12D

PURE CEMENT PASTE		
Number of Days	**Side A**	**Side B**
-2	0	0
0	20	15
1	-2	-9
2	-33	-38
3	-59	-64
4	-83	-86
7	-143	-151
8	-156	-163
9	-168	-176
10	-184	-192
11	-191	-199
24	-271	-277
25	-280	-286
28	-295	-304

Graph 12D

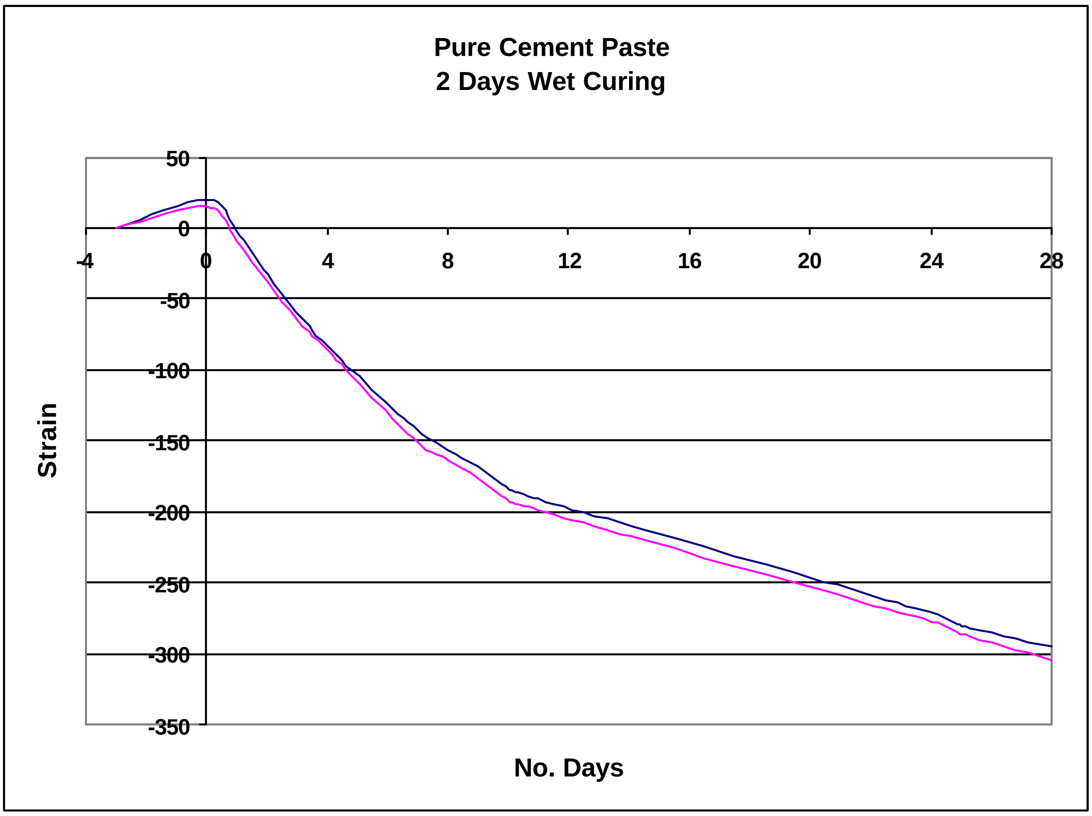


Table 12E

HARILAL LEAK SEAL		
Number of Days	**Side A**	**Side B**
0	0	0
1	-11	-9
2	-18	-16
3	-26	-21
6	-46	-44
8	-55	-54
9	-63	-66
12	-72	-71
15	-83	-81
16	-85	-83
19	-87	-86
22	-90	-88
23	-91	-90
26	-94	-91
27	-98	-94
28	-99	-96

Graph 12E

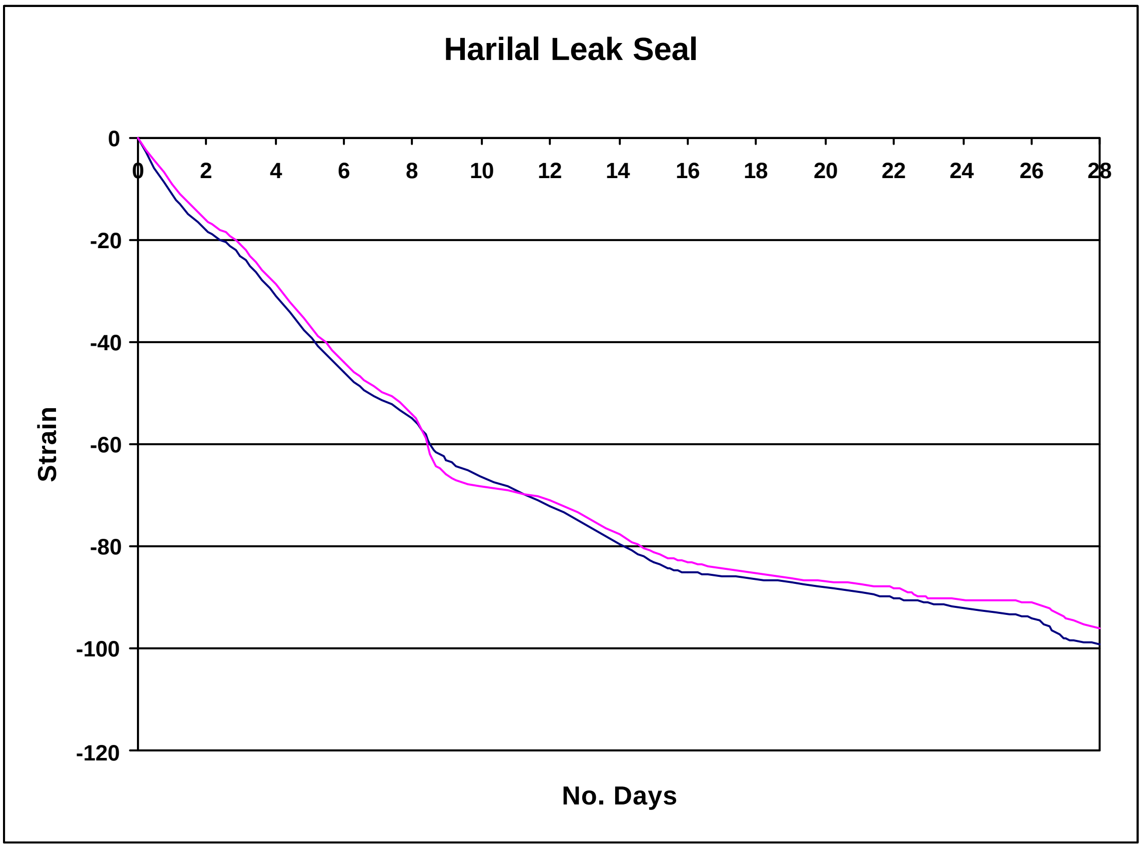


Table 12F

SILICA FUME		
Number of Days	**Side A**	**Side B**
0	0	0
1	-10	-8
2	-21	-22
3	-28	-26
6	-58	-60
8	-70	-71
9	-80	-81
12	-87	-89
15	-99	-101
16	-102	-104
19	-109	-109
22	-112	-111
23	-115	-114
26	-117	-116
27	-123	-122
28	-125	-125

Graph 12F

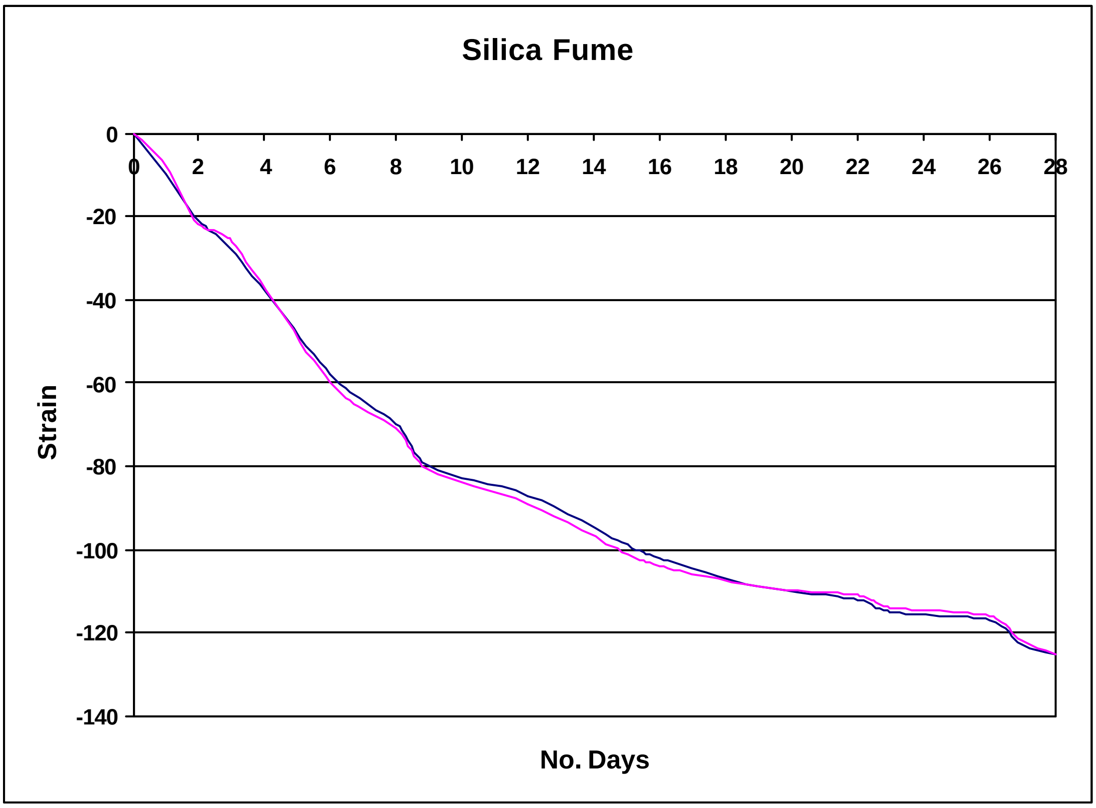
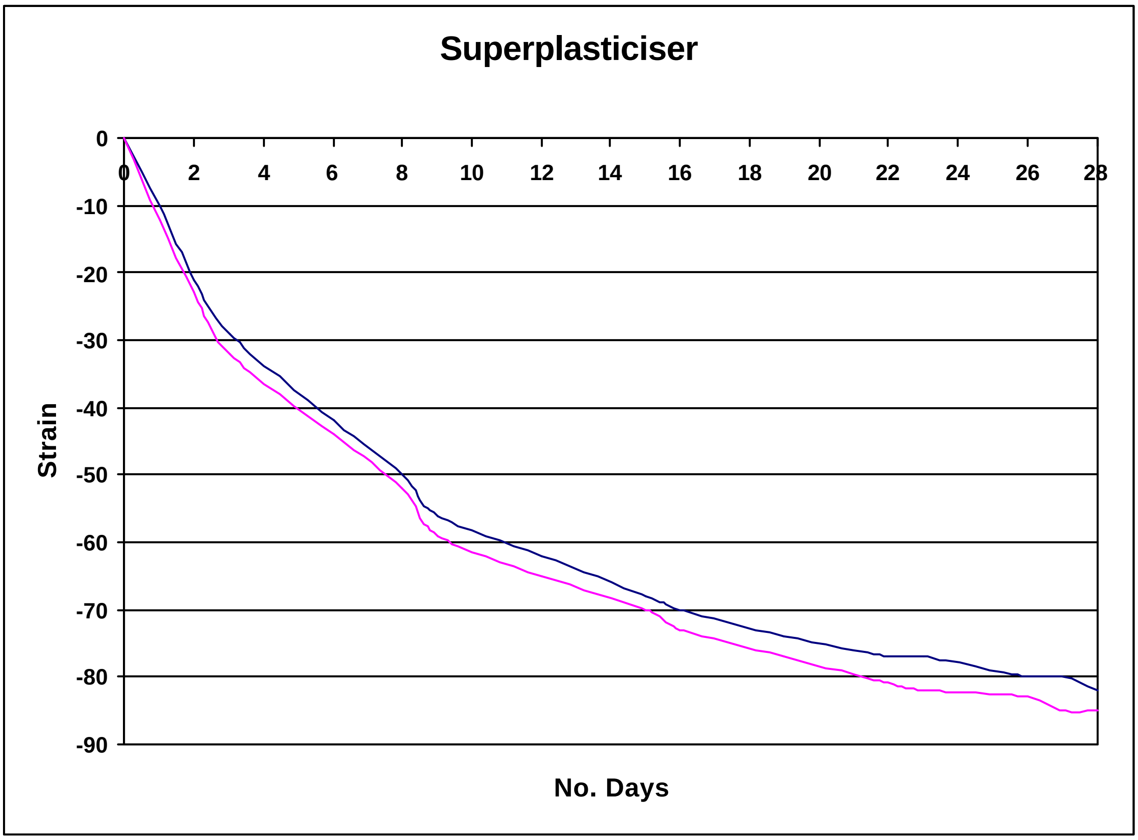


Table 12G

SUPERPLASTICISER		
Number of Days	**Side A**	**Side B**
0	0	0
1	-10	-12
2	-21	-23
3	-29	-32
6	-42	-44
8	-50	-52
9	-56	-59
12	-62	-65
15	-68	-70
16	-70	-73
19	-74	-77
22	-77	-71
23	-77	-82
26	-80	-83
27	-80	-85
28	-82	-85

Graph 12G



12.2 Shrinkage Analysis

The first round of shrinkage experiments consisted of 4 samples, 3 plain mortar and 1 pure cement paste. Graphs 12A-12G plot shrinkage against time and show well how each sample shrinks quickly at first before slowing, resulting in a relatively smooth curve of decreasing gradient. The two lines on each graph represent the two sides of each sample and it can be seen that generally warping has not been significant with any samples, including the second round of admixtures, because the lines remain close together throughout the plots. Any difference in the position of the lines would indicate a difference in the shrinkage of one side relative to the other. The sample with the most amount of warping was the plain mortar block wet cured for 2 days, with a 4.5% difference in strain between each of the two sides, but this figure remains small enough to not affect the validity of the result.

The next observation to be made is what affects the period of wet curing had on the shrinkage characteristics of each sample. Firstly, research shown earlier in this report predicted that the majority of shrinkage would take place after drying has begun. This indeed was the case and due to the fact that during wet curing the samples were at 100% humidity, no shrinkage was recorded. Instead the opposite occurred and all of the first round samples underwent an increase in volume. This increase in volume was seen in the sample wet cured for 14 days, to level off at approximately +10 micro strain. This result wasn't entirely unexpected as research showed that this it is not uncommon for cementitous materials to increase in size in 100% humidity conditions. Even after drying this affect can be seen, although was not investigated in this project.

The period of wet curing had a marked affect on the amount of shrinkage seen in each of the 3 identical plain mortar samples. The sample cured for 14 days shrank 25% less than the one cured for only 2 days. The sample that experienced 4 days wet curing had a 10% reduction in shrinkage compared to the 2 day sample. It was expected that the increased curing would increase the material's strength, but it was unexpected that it would affect the shrinkage in such a significant manner. 25% is a major reduction in shrinkage and even though the period of curing was of secondary interest in this report, the result shows that it is a field worthy of further investigation.

In the context of using the mortar as a render in a water tank it is undesirable to have a long period of wet curing, but it should be considered if it could reduce the overall shrinkage seen in the render. Even an extra two days has been seen to reduce the shrinkage by up to 10%.

A possible explanation for the reduction in the levels of shrinkage recorded can be linked to the material strength, as follows: It has been shown that cementitous materials shrink when water is lost through evaporation. As the material dries pores are left, basically cavities where excess water has been stored, which makes the material porous after drying and reduces the strength of the material. The longer the mortar cures, the stronger it will become and it would be able to resist the shrinkage forces associated with drying and water loss. It may therefore shrink less and instead have greater internal forces built up within the material when compared to a less cured and hence weaker mortar.

When looking at the results from the samples containing admixtures, it can be seen that they too have had a big effect on the shrinkage of the mortars. As mentioned in section 6, an admixture will generally make a mortar less permeable by one of the two following methods:

1. Reduce shrinkage
2. Increase strength

Silica fume is an admixture that is used to make cementitous materials stronger. This will in theory reduce permeability because the material would crack less for any given reduction in volume. This theory is backed up by this result because the addition of silica into the mortar has had a negligible effect on the shrinkage. It underwent a reduction in length of 125 micro strain compared to 128 for the average of both sides of the similarly cured plain mortar with no admixtures, less than a 3% change.

The superplasticiser had the biggest effect on the shrinkage characteristics of mortar. At 83 micro strain reduction, it experienced a 35% reduction in shrinkage compared to the plain sample. This was expected because the plasticiser allowed a reduction in the amount of water used, although the magnitude of the reduction is surprising seeing as only 17% less water was used to create the 35% reduction in volume.

It was not known at the beginning of the investigation how the Harilal Leak Seal would affect the permeability of mortar, but it can be seen that it is less like the silica fume than the plasticiser because it has resulted in a reduction in the recorded shrinkage of 24% compared to the plain sample. The next experiment on cracking will determine whether it also has an effect on the strength of the mortar.

All of the admixture samples were prepared and tested over the same period, and a small anomaly can be noted on the day 8-9 period on all of the graphs for these samples. The anomaly in question is a small kink in the graph indicating an increase in shrinkage over the one day period. It is interesting that it is present on all of the samples and is a good indication of the validity of the results because it shows all samples were tested under the same conditions. The increase in shrinkage can be explained by there being a temporary increase in the temperature of the samples over that period which would result in more moisture being evaporated and hence a greater reduction in volume.

Chapter 13: Crack Results and Analysis

13.1 Crack Results

The results for both rounds of the crack experiment are provided over the next 2 pages.

Table 13.1A – Plain mortar results (1st round)
Table 13.1B – Silica Fume results (1st round)
Table 13.1C – Harilal Leak Seal results (1st round)
Table 13.1D – Superplasticiser results (1st round)

Table 13.2A – Plain mortar results (2nd round)
Table 13.2B – Silica Fume results (2nd round)
Table 13.2C – Harilal Leak Seal results (2nd round)
Table 13.2D – Superplasticiser results (2nd round)

Table 13.1A

Plain			
Crack Number	**Width (divisions)**	**(Width ìm)**	**Length mm**
1	4	200	130
2	3	150	81
3	2.5	125	76

Table 13.1B

Silica Fume			
Crack Number	**Width (divisions)**	**(Width ìm)**	**Length mm**
1	1.75	87.5	95
2	1.5	75	130
3	1.5	75	130
4	1.25	62.5	20
5	1.25	62.5	66

Table 13.1B

Harilal Leak Seal			
Crack Number	**Width (divisions)**	**(Width ìm)**	**Length mm**
1	2.5	125	130
2	2	100	130
3	1.5	75	81

Table 13.1C

Superplasticiser			
Crack Number	**Width (divisions)**	**(Width (ìm)**	**Length mm**
1	2.5	125	98
2	2.5	125	62

Table 13.2A

Plain			
Crack Number	**Width (divisions)**	**(Width ì m)**	**Length mm**
1	3.5	175	130
2	2.5	125	130
3	2	100	28
4	1.5	75	43

Table 13.2B

Silica Fume			
Crack Number	**Width (divisions)**	**(Width ì m)**	**Length mm**
1	1.75	87.5	130
2	1.5	75	130
3	1.25	62.5	91
4	1	50	97
5	0.5	25	130
6	0.5	25	130

Table 13.2C

Harilal Leak Seal			
Crack Number	**Width (divisions)**	**(Width ì m)**	**Length mm**
1	2.5	125	130
2	2.5	125	130

Table 13.2D

Superplasticiser			
Crack Number	**Width (divisions)**	**(Width (ì m)**	**Length mm**
1	2.5	125	98
2	2.5	125	62
3	1.75	87.5	41

13.2 Crack Analysis

The first fact that can be noted from looking at the tables on the crack sizes is that there is a great range of lengths and widths of cracks across the samples, as well as the number of cracks per sample. The first round of results show that the silica fume sample was most extensively cracked in terms of quantity, but the plain sample had the widest cracks of all samples.

Multiplying the length and width of each crack and adding all values for each of the cracks in any one sample can be a simple piece of analysis. It will provide a rough idea of the total crack surface area for each sample. See table 13.3A below.

Table 13.3A

SAMPLE	TOTAL CRACK SURFACE AREA: ROUND 1 (ìm^2)	TOTAL CRACK SURFACE AREA: ROUND 2 (ìm^2)
Plain	47.65	45.03
Silica Fume	33.19	38.16
Harilal Leak Seal	35.33	32.5
Superplasticiser	20.00	23.59

It is accepted that cracks vary in width along their length and that each crack may be of different depths so these figures may not correlate exactly with the leakage rates shown later, but the figures correlate well with those seen for shrinkage. The plain sample, which was seen to undergo the greatest amount of shrinkage, also has the highest crack surface area up to 30% more than the next highest of Harilal Leak Seal.

Comparing silica fume with Harilal, the total area of cracking is very similar, although with silica fume the area is spread out over a greater number of thinner cracks compared to the few wide cracks of Harilal. The mathematical analysis of flow through cracks in section 6 would suggest a greater leakage rate for the Harilal sample because of the extra width of the cracks. Whether this is the case or not can be seen in the next piece of analysis.

From the shrinkage results it was shown that mortar with silica fume present experienced a similar level of shrinkage compared to plain mortar, while the Harilal sample shrank considerably less, yet the resultant cracking is very similar. This is an important development because it confirms that the silica fume sample is likely to be much stronger than the Harilal mortar. The strength seems to have affected the distribution of the cracks also, because the silica fume sample has many more cracks than any of the other samples, a fact which is true for both rounds of experimentation.

Overall these results correlate well with those from the shrinkage experiments and the superplasticiser has performed the best in this area of investigation as well. The superplasticiser experienced up to a 43% reduction in cracking compared to the plain sample, from a 35% difference in shrinkage levels measured. Even though the superplasticiser has enjoyed a significant reduction in shrinkage induced cracking, the cracks are relatively wide when compared to examples from the silica fume sample. This is predicted to be undesirable due to the much bigger flow rates that wide cracks may be subject to. The final piece of analysis of the leakage results will show whether the superplasticiser's reduction in crack area will be of benefit if it results in wider cracks.

Chapter 14: Leakage Results and Analysis

14.1 Leakage Results

The results of the Leakage experiments carried out on the 2nd round of crack samples is provided over the coming four pages.

Table 14A – Volume of water lost per time period, plain mortar sample.
Table 14B – Volume of water lost per time period, silica fume sample.
Table 14C – Volume of water lost per time period, Harilal Leak Seal sample.
Table 14D – Volume of water lost per time period, Superplasticiser sample.

Table 14E and graph 14E – Steady flow rates, plain mortar sample.
Table 14F and graph 14F – Steady flow rates, silica fume sample.
Table 14G and graph 14G – Steady flow rates, Harilal Leak Seal sample.
Table 14H and graph 14H – Steady flow rates, Superplasticiser sample.

Table 14A

PLAIN	
Time Interval (minutes)	Water lost (ml)
1	7
1	4.9
1	4.1
1	3.8
1	3.2
1	2.9
1	2.5
1	2
1	1.6
1	1.2
5	5.1
5	5
5	5.2
5	4.9
5	4.7
5	4.6
5	4.9
5	5.1
5	4.9

Table 14B

SILICA FUME	
Time Interval (minutes)	Water lost (ml)
1	6.2
1	4.4
1	3.5
1	2.6
1	1.7
1	0.9
1	0.6
1	0.4
1	0.5
1	0.4
5	2.1
5	2
5	2.1
5	1.9
5	2
5	2.2
5	1.9
5	2
5	2.1

Table 14C

HARILAL LEAK SEAL	
Time Interval (minutes)	Water lost (ml)
1	7.1
1	5.4
1	4.1
1	3.6
1	2.8
1	1.1
1	0.8
1	0.5
1	0.4
1	0.4
5	1.7
5	1.6
5	1.7
5	1.5
5	1.7
5	1.6
5	1.6
5	1.5
5	1.6

Table 14D

SUPERPLASTICISER	
Time Interval (minutes)	Water lost (ml)
1	6.6
1	5
1	4.1
1	3
1	1.4
1	1
1	0.7
1	0.4
1	0.3
1	0.2
5	1.1
5	1
5	1.1
5	1.1
5	1.1
5	1.2
5	1.1
5	1
5	1.1

Table 14E

PLAIN	
Total time (minutes)	Flow rate (ml/hour)
1	420
2	294
3	246
4	228
5	192
6	174
7	150
8	120
9	96
10	72
15	61.2
20	60
25	62.4
30	58.8
35	56.4
40	55.2
45	58.8
50	61.2
55	58.8

Table 14F

HARILAL LEAK SEAL	
Total time (minutes)	Flow rate (ml/hour)
1	426
2	324
3	246
4	216
5	168
6	66
7	48
8	30
9	24
10	24
15	20.4
20	19.2
25	20.4
30	18
35	20.4
40	19.2
45	19.2
50	18
55	19.2

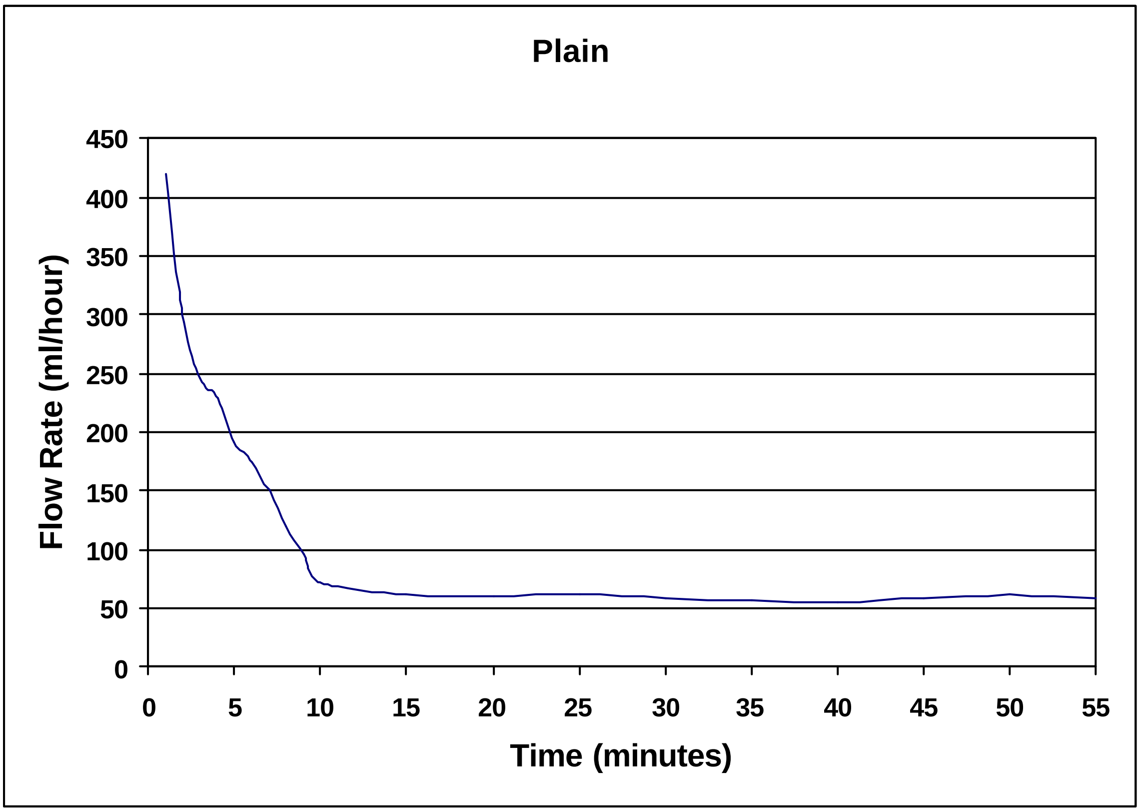
Table 14G

SILICA FUME	
Total time (minutes)	Flow rate (ml/hour)
1	372
2	264
3	210
4	156
5	102
6	54
7	36
8	24
9	30
10	24
15	25.2
20	24
25	25.2
30	22.8
35	24
40	26.4
45	22.8
50	24
55	25.2

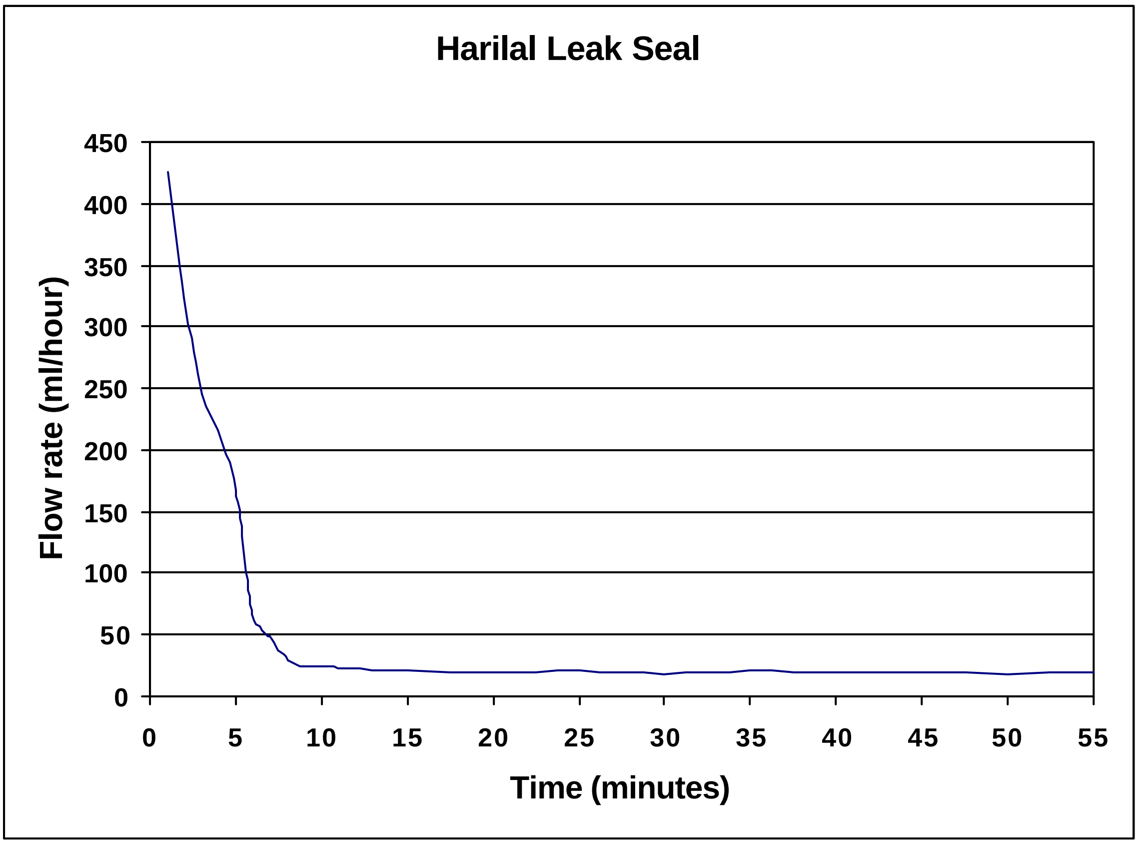
Table 14H

SUPERPLASTICISER	
Total time (minutes)	Flow rate (ml/hour)
1	396
2	300
3	246
4	180
5	84
6	60
7	42
8	24
9	18
10	12
15	13.2
20	12
25	13.2
30	13.2
35	13.2
40	14.4
45	13.2
50	12
55	13.2

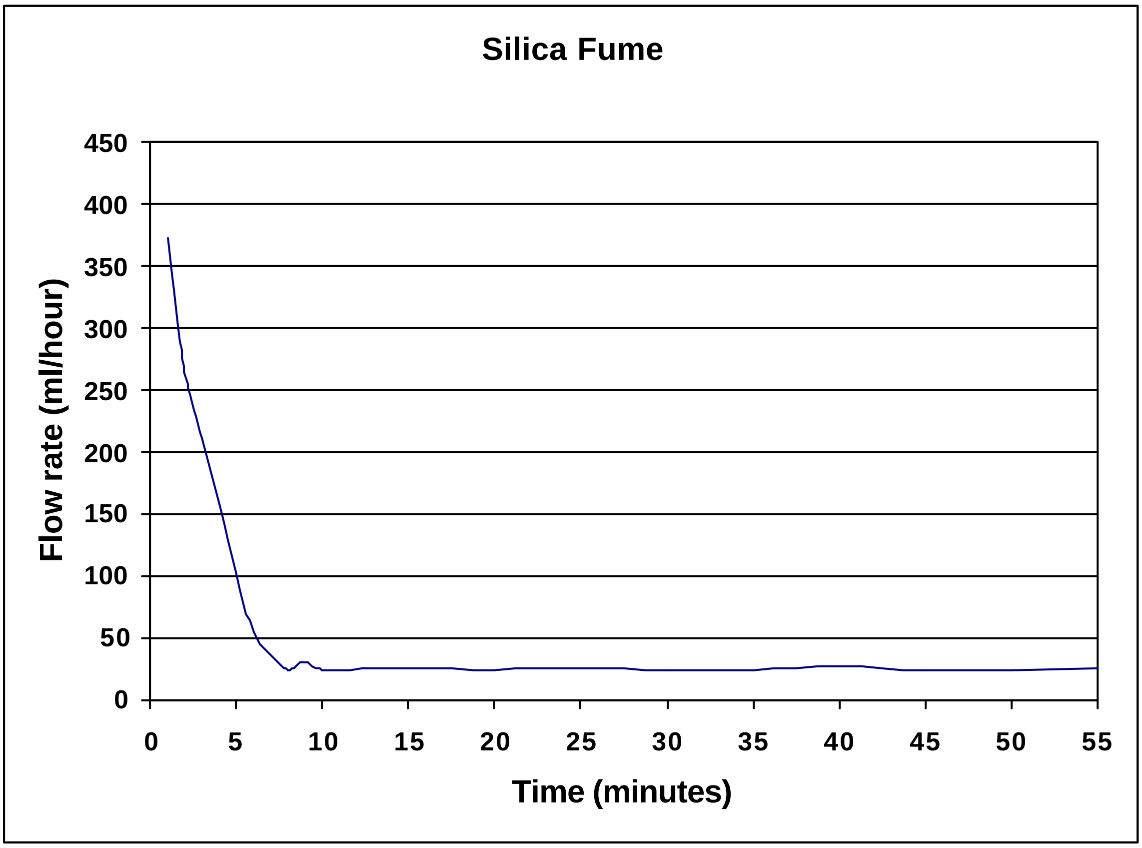
Graph 14E



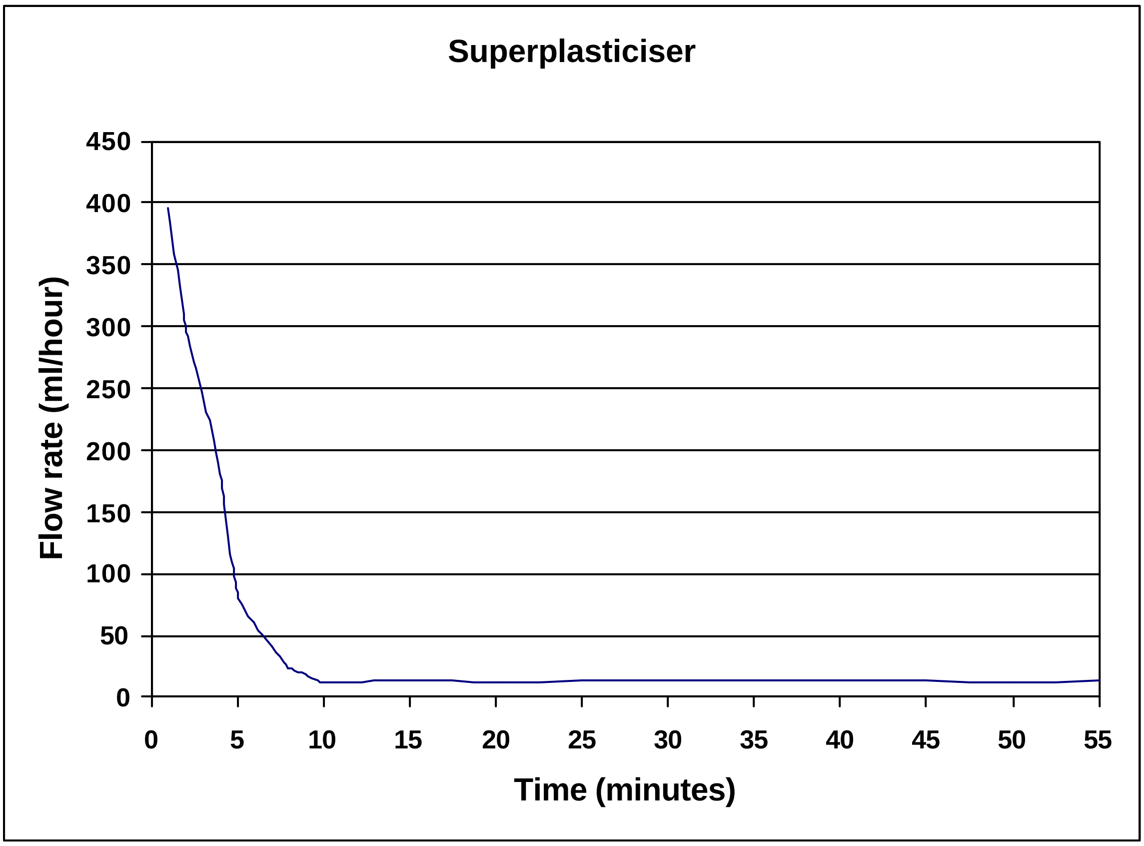
Graph 14F



Graph 14H



Graph 14H



14.2 Leakage Analysis

The first thing to check when looking at the results for the leakage experiments is whether initial flow rates for each sample are of a similar magnitude. This is an important factor because if one sample has a much lower initial flow rate when compared to the others, this may restrict the amount of leakage seen due to the water not flowing through the groove quickly enough. This doesn't seem to be the case because all initial flow rates are of the same order of magnitude and the final flow rates seen are much lower than initial rates so shouldn't be restricted in any way. It should be reiterated that leakage results are only available for the samples from the second round of crack investigation.

As expected water levels in the test equipment drop significantly in the first few minutes of experimentation which can be apportioned to the groove filling with water and saturating the string. On all of the samples it has taken approximately 7-10 minutes for the flow rates to level off to a constant rate. The first few seconds of very high flow is, as mentioned above, due to the groove in the testing rigs filling with water. The flow drops off very quickly but remains higher than final values until around the 7 minute mark. Prior to this the medium flow rates can be attributed to the cracks filling with water. Over the first 7 minutes of the of the experiment it can be assumed there is little or no actual leakage from the mortar, rather than the flow can be explained on the water gradually filling every crack before being able to seep out onto the surface. Evidence of this is that no water was visible on the surface of the mortar for the first 5 minutes or so of each experiment. Subsequent to this moisture was visible originating from some of the cracks.

It has been stated that steady flow rates are of the most interest in this round of experiments, and it was predicted that it might take a few minutes for the flow to settle to this steady level. Comparing the steady flow rates shows some interesting correlations with previous results. The plain sample, which was the most cracked prior to the leakage test, experienced a significantly higher flow rate than any of the others with the equivalent of approximately 59ml per hour lost. The silica Fume sample which had a crack surface area 15% lower than the plain sample, had a steady flow rate of only 24ml per hour, 40% lower than the plain sample. This result could be of extreme importance because it would confirm the theory that for any given area of cracking, it is desirable to have it spread over a larger number of thin cracks rather than over a smaller number or wider cracks.

Subsequent results correlate with the total area of cracking. Harilal Leak Seal leaks at a rate of approximately 20ml per hour leakage, while 13ml per hour is the rate for the superplasticiser.

Comparing the silica fume sample to the Harilal is of interest because they both underwent similar levels of cracking, but had different numbers and widths of cracks. The Harilal sample came out on top as it leaked approximately 17% less. When this is compared to the 16% less cracking it experienced, the result becomes more important, because in this case it seems to go against the theory that crack width is the most important factor when considering leakage through cracks in mortar. All of the crack surface area of the Harilal sample was taken up by 2 long wide cracks with an average width of 125ìm, while the silica sample had 6 much thinner cracks with an average width of only 54ìm.

The superplasticiser leaked the least of all of the four samples, 76% less than the plain sample. It did also crack the least out of the 4 samples so this was not entirely unexpected, but again the cracks were of greater width than many cracks seen on other samples so initially this results seems to go against the theory that width is the key to how much water leaks from any one sample. The study on whether crack width affects leakage has proved inconclusive, a matter which is discussed in a following chapter.

Chapter 15: Summary of Analysis

The results from the experiments of the three variables have provided extremely relevant information on the suitability of each type of mortar for use as a waterproofing render on water tanks. The correlation between the shrinkage recorded from examples of each type of mortar and the amount of cracking has been high. The only exception to the pattern of high shrinkage causing high cracking was the relationship between the plain mortar, and the sample with silica fume added to it. The addition of silica had negligible effect on the shrinkage of the mortar, but this sample cracked significantly less than the plain mortar, a fact that has been attributed to the extra strength of mortar with silica fume added to it.

The remaining two admixtures, Harilal Leak Seal and the superplasticiser, both led to reduced shrinkage recorded in the mortar when compared to a plain sample. This reduction in shrinkage was confirmed by the experiments on cracking, where the two admixtures had far less cracking than the plain sample.

The silica fume sample experienced greater cracking than the other two admixtures but the extra strength of the mortar caused a spread of the shrinkage over a greater number of cracks, which according to section 6 would have a large effect on how much water leaked from it. Unfortunately the experiments proved inconclusive on this subject, because there is evidence that both supports and contradicts this theory. Comparing the leakage rates and the crack characteristics of the silica fume sample with the plain sample supports the theory, while comparing the silica fume with the Harilal Leak Seal seems to contradict it.

If the reader refers to the appendix and the results of cracking in previously carried out work (Steve Turner's), it can be seen the similarity between the types of cracking on the mortar enhanced with silica fume. These samples underwent different curing times (3 months wet curing in this case) and the cracking resulting from drying is seen to consist of many thin cracks rather than wider ones as seen on other samples. Again the plain sample experienced fewer, but wider cracking. The superplasticiser performed well as it did in this set of experiments with all of its cracks relatively short and thin. The main result of interest out of these results is the performance of the nil coat layer. This method of applying mortar involves casting an initial layer of thin mortar, allowing it to dry, and applying an even thinner layer of cement slurry (no sand), and protecting this with a final coat of mortar. The thinking behind this is that when the first layer dries and cracks the pure cement layer will help fill any cracks, and because it is denser than mortar, will be more impermeable as well. There was no opportunity to explore this possibility in this investigation due to the failure of a round of experiments and the time involved in retesting them.

The Ferrofest sample also did well, but again there was no scope to test it in this investigation. Further experimentation into the performance of these type of "sandwich" layers is recommended.

A further admixture, Festegral, was used in Steve's samples, but was unavailable for testing in this experiment as there was insufficient quantities left to experiment on but a glance at the results of the cracking experiment using this admixture suggest a lack of performance.

Chapter 16: Discussion

The leakage experiment has been the source of most of the problems in this investigation. It needed careful planning when designing the experiment to make sure all of the possible unknowns were removed to ensure reliable results were produced. In hindsight there is still a problem with the experimental method used. Observation of the surface of the samples throughout the course of the experiment showed that not all of the cracks in the mortar were leaking. From here it can be concluded that even though the mortar layer is thin at 10mm, it is not definite that all of the cracks propagated the whole depth of the layer of mortar. If this was the case then any relationship between the level of cracking and the flow rate of water through any cracks is invalid.

For a crack to affect the rate of leakage of a sample, it must be deep enough to reach the water supply at the surface of the steel ring. If a crack does not propagate this deep, then it will in no way play a part in the observed leakage. Below is a picture of a crack in one of the samples (fig 15A). If an imaginary section is taken along the dashed lines, there is no way of telling how deep the crack goes. Figure 15B on the following page shows a diagram of how this crack may have propagated in depth.

Fig 15A

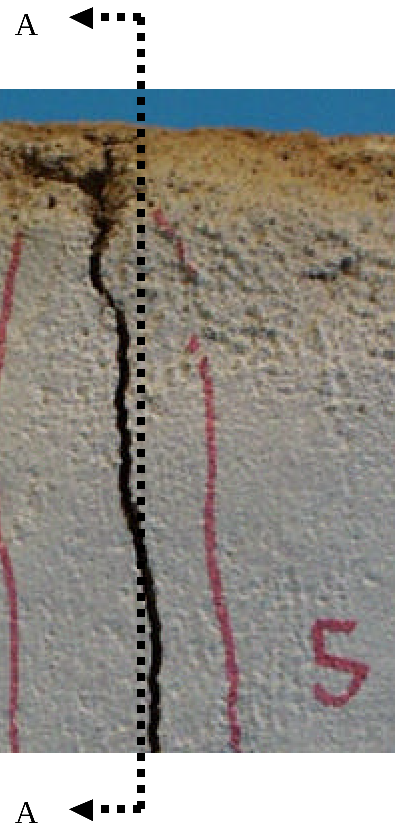
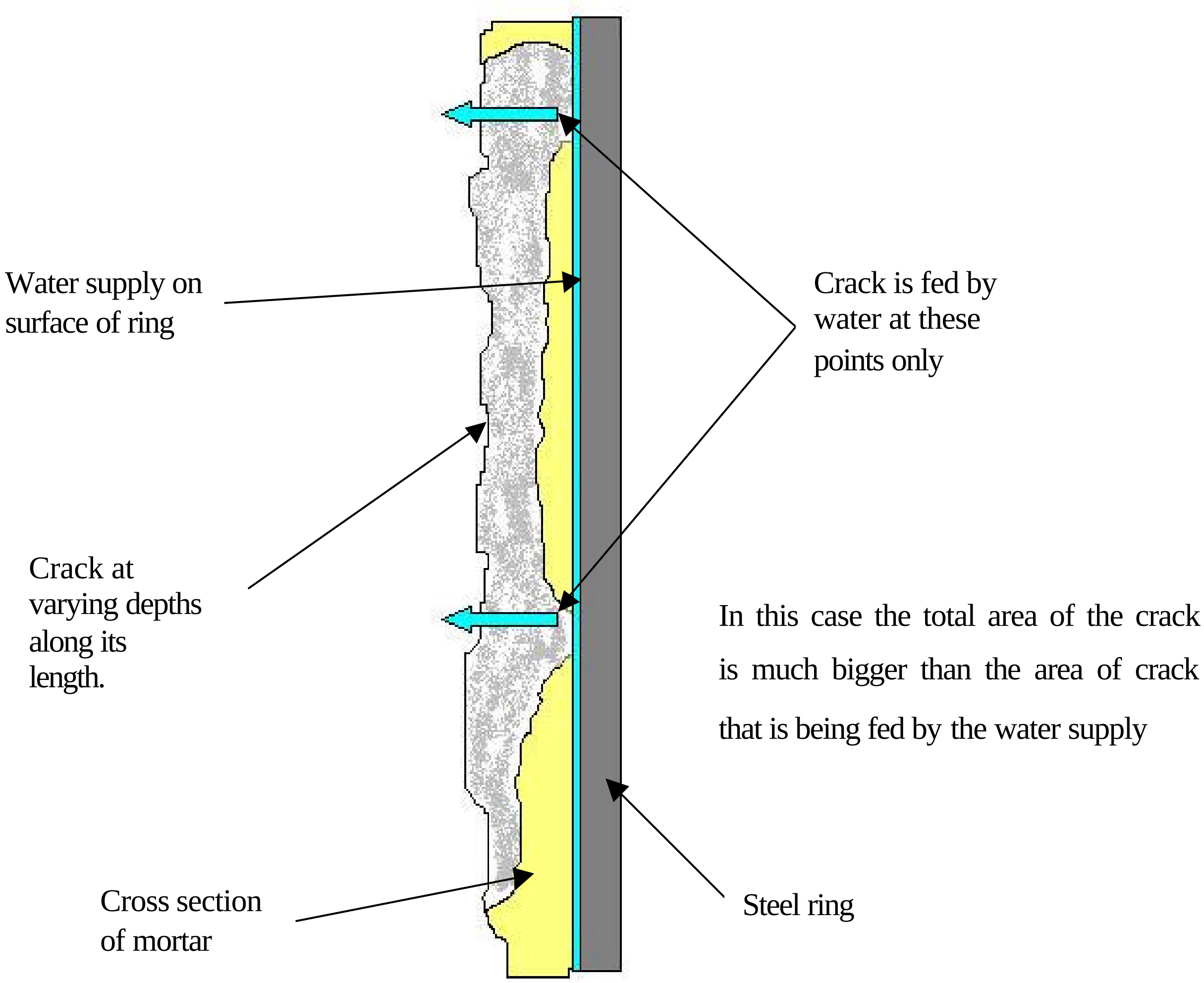


Figure 15B



To allow valid comparisons between crack area and leakage flow rates, the area of crack in contact with the water supply must be known. It was impossible to find this area in the bounds of this investigation. A possible way of doing it would be to cut down the length of each crack after the leakage results have been taken, but this was not possible using conventional methods because the disruption of the mortar would cause further crack propagation and be generally too intrusive to give accurate results.

Even though the comparisons between crack area and leakage may not be valid, the leakage results are still important. If a crack does not propagate the whole depth of the mortar layer, then it will not cause leakage and hence some tension has been released at no cost. It would be extremely desirable to have a mortar that relived the shrinkage-induced tension with purely superficial surface cracks, as it would then not leak at all. This hasn't been the case with any samples tested but anomalies that occur when comparing the leakage and the cracking of some samples can be explained by the possibility that not all of the cracks penetrate the full depth of the mortar. This helps to explain why the Harilal Leak Seal sample leaked slower than expected when compared to the silica fume sample.

There are many other factors to consider when looking at the experimental procedure for the leakage variable. Both shrinkage and crack measurements are relatively easy to perform reliably and accurately, but the leakage of water from a sealed vessel is notoriously unreliable, especially when the vessel is sealed with a material as coarse as mortar. The groove in the steel ring is basically a small water tank with the top and sides sealed with mortar. It must be well sealed to make any results for leakage useable. There are many places other than a crack in the mortar that water could leak from, as seen in the first round of leakage experiments where the water was lost at the boundary between the mortar and the retaining clip. The subsequent change in design of the experiment appeared to have cured this problem but the equipment can still not be guaranteed watertight. Had the first round of experiments not leaked, then two sets of results for leakage would be available for analysis, and this would show whether the results were repeatable, and hence increase their validity.

In chapter 6, it was suggested that the flow rate of water through a crack was related to the crack width by the relationship ÓLb^2 where L is the crack length and b is the width. This cannot be proved or disproved, as there is evidence both for and against it.

The only way to be entirely sure that any results from the leakage experiments are useful when considering water tanks, is to construct a tank as would be used in places like Uganda, render it with mortar and carry out a study of how much it leaks. In reality it would be impractical to do this for every type of mix, so the experiments used in this investigation are useful to determine which mixes of mortar are likely to be best suited to carrying out the desired task. However, the results provided should not be regarded as a guarantee of the suitability of a mix of mortar for the task it has been set to do.

Chapter 17: Conclusions

Analysis of the results from the three experimental methods has provided information from which conclusions can be made. The first conclusion is that admixtures have a definite impact on the properties of a mortar mix to which they are added, in some cases a dramatic impact. Of the three admixtures studied in this report, the superplasticiser has the largest favourable effect on the shrinkage properties of drying mortar, causing a 35% reduction in shrinkage. The silica fume had the least effect on shrinkage at only a 3.5% reduction when compared to plain mortar, although this is unsurprising because it is used as a strengthening admixture rather than one that reduces shrinkage. Prior to the results from the shrinkage experiment, it was unknown how the Harilal Leak Seal admixture would reduce leakage from a mortar rendered water tank. It can be concluded that it reduces the shrinkage experienced by a sample of mortar, rather than increasing the strength, although whether it affects the strength is unknown as no experiment was done on this variable.

The analysis of the cracking experiment shows correlation between shrinkage and cracking. The superplasticiser experienced the least cracking, which was expected after it was discovered how much it reduced shrinkage. It can also be concluded that increasing the strength of mortar will help distribute shrinkage over a greater number of thinner cracks. The superplasticiser underwent the least shrinkage but its cracks were wide when compared with those on the silica fume sample. Referring back to section 6, it was shown that increasing the width of a crack by a factor of 2 will increase the leakage by a factor of 4, so the type of cracking on the superplasticiser is undesirable when compared to the silica fume.

From the leakage experiment it can be seen that all of the three admixtures tested are favourable when compared to plain mortar. The plain mortar sample experienced most cracking by area, had the widest cracks, and had significantly greater flow rates through the cracks. Consistently throughout the investigation, the plain samples have preformed poorly when compared to the samples where admixtures have been used. Again the superplasticiser and the Harilal leak seal performed best in the leakage experiments. It was proposed in the discussion that any comparison between cracking and leakage would not be well founded due to the unknown surface area of crack being fed by the water supply. For this reason the result of the mathematical model in section 6 cannot be verified, although there is evidence both for and against it. The plain sample had by far the most significant cracking and had the highest flow rates by over a factor of 2 which supports the analysis in section 6, although the comparison between crack widths and leakage rates in the Harilal and silica fume samples go against the theory. This report recommends that an investigation into flow through channels of controlled widths be carried out to decide whether crack width has such a significant effect on flow rate.

The possibilities of mixing different admixtures in a sample of mortar was not looked into in this investigation due to the amount of time each round of experiments took, and the need to re-cast a round as explained previously in the report. From the results it is suggested that a mortar sample that has both silica fume and a superplasticiser added to the mix be investigated. The two factors highlighted in section 7 that affect the leakage through a mortar (strength and shrinkage), can be improved individually with these admixtures. It appears that no one admixture can improve both of these properties but combining the two that most significantly improve each one may combine the benefits of each.

Another possibility for further investigation would be to experiment on the ferrofest layer as described in section 7, and a similar layering technique using a sandwich of two plain coats and a thin layer of pure cement paste in the middle to act as a “filler” to block any cracks that form in the first layer. Many combinations of admixtures are available and many products are on the market that haven’t been tested in this investigation due to time constraints. An exhaustive study of these would show which combinations of admixtures would be most suitable.

However, the conclusion of this report using the results recorded is that if using a single admixture, the superplasticiser is most suitable for use in a mortar to render a water tank. The report also recommends that a combination of the superplasticiser and silica fume in a sample of mortar is likely to combine the benefits of each, and further study into this possibility is recommended.

Chapter 18: Literature Review

AUTHOR	YEAR	TITLE	PUBLISHER
A.M.Neville	1981	Properties of Concrete	Pitman
S.B.Watt	1978	Ferrocement water tanks and their construction	Intermediate Technology Publications
T.N.W.Akroyd	1962	Concrete: Properties and Manufacture	Pergamon Press
P.L.Critchell	1968	Joints and cracks in concrete	C R Books
Edited by M.R. Rixom	1977	Concrete admixtures	Construction Press
F. D.Lydon	1982	Concrete mix design	Applied Science Publishers
D.F.Orchard	1979	Concrete technology	Applied Science Publishers
T.H.Thomas, B. McGeever	1997	WP49 Underground Storage of Rainwater for Domestic Use	DTU, University of Warwick
T.H.Thomas	2000	WP55 Very low cost Roofwater Harvesting in East Africa	DTU, University of Warwick
D.Rees	2000	TR-RWH 01 - Partially below ground (PBG) tank for rainwater storage - Instructions for manufacture	DTU, University of Warwick
D.Rees	2000	TR-RWH 03 - Experimental Rammed Earth tank of 6 cubic metres - Instructions for manufacture	DTU, University of Warwick

Chapter 19: Appendix

File 1: Steve Turner's experiment notes.

File 2: Results taken from the cracking experiment.

Engineering Undergraduate Project

Development and Selection of Low Cost Handpumps for Domestic Rainwater Water Tanks in E. Africa

Vince Whitehead

University of Warwick

May 2001

Summary

This report gives details of the development and selection of a handpump suitable for use with domestic rainwater harvesting tanks in East Africa. The objective of the project was to develop a small low cost handpump, which can be manufactured, maintained and repaired with a minimum of tools and skill and that the materials can be found in most local hardware outlets and markets.

Four designs were proposed which were selected from a range of pump technologies for low head and low flow rates. From these, two were selected for their ease of manufacture, low skill level and expected reliability. The two handpumps ('Harold' and the 'Enhanced inertia') were subjected to a series of performance and durability tests. From these tests, both handpumps were found capable of lifting at least 15 litres per minute at 70 cycles per minute with acceptable hydraulic efficiencies. The actual lifting rate was significantly greater than the value given in the specification.

The durability tests showed very little evidence of wear in either handpump after 145 hours continuous running other than some potential splitting in the valve surfaces. An extended endurance test on the recommended handpump, the Enhanced inertia, resulted in it lifting around 300,000 litres and having an equivalent life of 8 years.

The handpumps were produced in Uganda for less than $10 for a 3.5m length, which was one of the main criteria in the specification. The pumps were successfully manufactured by a number of technicians in Uganda after a two-day training workshop and this illustrates that the design and technology is appropriate.

Contents

Acknowledgements

I would like to express my thanks to the following people:

All the staff at Kyera farm, Mbarara, Uganda who assisted in the first stage of this project and for their hospitality and encouragement whilst working there.

Everyone at the Development Technology Unit, University of Warwick for the experience and opportunity of working in Uganda.

To Andre and Bob Leliveld for the photographs and for their company during my stay in E. Africa.

Erich Baumann and the Handpump Technology Network (HTN) for their valuable assistance, knowledge and feedback throughout this project.

Also to Jonathan Keighley for his time in proof reading the report.

Glossary

CATRL	Consumer's Association Testing and Research Laboratory
DRWH	Domestic Rainwater Harvesting
DTU	Development Technology Unit
GBP	Great British Pound
lpcd	litre per capita day
NGO	Non governmental organisation
PAT	Portable electrical Appliance Test
PVC	Polyvinyl chloride
UGS	Ugandan shillings
UNICEF	United Nations International Children Emergency Fund
URDT	Ugandan Rural Development & Training organisation
WHO	World Health Organisation

1. Introduction

The aim of this project was to develop a suitable small low-cost handpump, which could be used for abstracting water from Domestic Roofwater Harvesting (DRWH) systems in East Africa. A low-cost DRWH system is shown in Figure 1, and consists of a roof to intercept the rain, a series of gutters and downpipes, and a purpose built tank into which the handpump is installed.

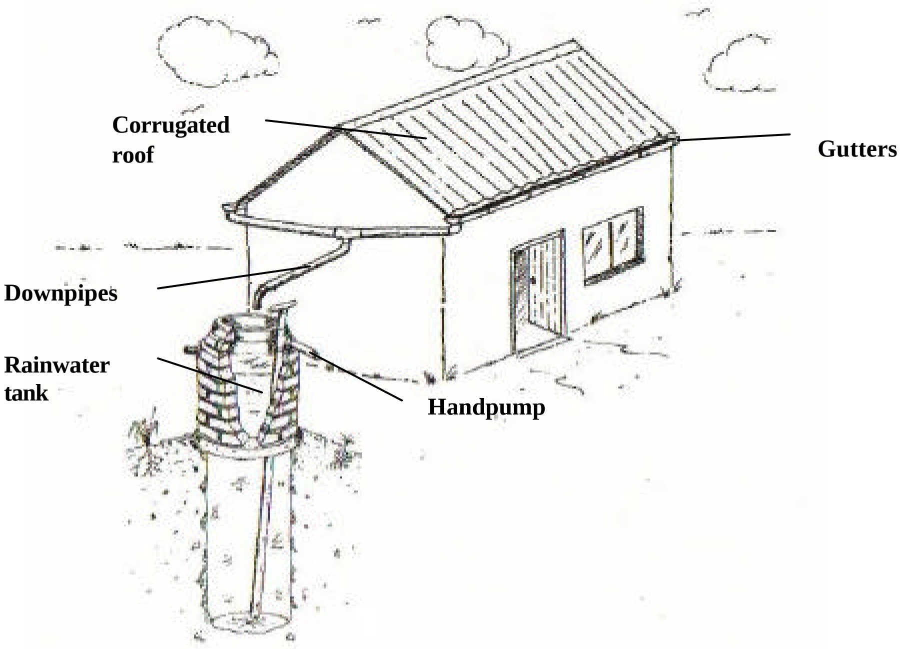


Figure 1 A domestic Rainwater harvesting system

This project was divided into two phases. Firstly an introductory phase, carried out in Uganda (chosen to represent African conditions), was used to identify constraints within the environment and expose four handpump designs to users. Secondly, the main phase of the project was to identify two candidate designs, refine them and carry out performance and endurance tests at the University of Warwick.

One of the main priorities in developing the handpumps was to ensure that the manufacture and materials could be made and/or sourced from within the local area.

The first phase was carried out by the author in Mbarara, (the fourth largest town in Uganda), during July/August 2000 and at Kyera farm, Mbarara. This involved assessing the manufacturing capabilities within the locality, material supplies and the availability of tools in local markets. Some prototype handpumps were manufactured and installed in DRWH systems in Uganda.

Many existing pumps may be regarded as over designed and too expensive to incorporate in to a DRWH system. They can also be difficult to maintain because of the high cost of spares, and the spares may be stocked some distance from the pump location.

The second and main phase was carried out at the University of Warwick. This involved choosing two of the four proposed designs and carrying out a series of performance tests, refining them and then subjecting each handpump to an endurance test.

To achieve the aims and objectives of this project, a plan was set out so the project could follow a logical sequence of tasks over the allocated period and finish completed on time. A software package was used to plan the projects tasks, this was then used to monitor the progress of the project and make adjustments should any arise. A hard copy of the project plan is shown in Appendix 1.

2. Analysis of need and development of specification

Many areas of East Africa have a very varied rainfall pattern and in particular regions, for example in Rwanda, this can result in a six-month dry season. Many rural families do not have access to an adequate and safe water supply. This can mean long treks to some distant water source, which may be of low quality and consume valuable hours from their daily duties.

Fetching water may often involve many hours a day in walking several miles to and from the source by either children or women. The time spent collecting water is a double burden, as it means less time is available for the productive activities on which subsistence economies depend[1].

Definitions given by WHO (1996[2]) are as follows:

- Access to water: In urban areas, a distance of not more than 200 metres from a home to a public standpost may be considered reasonable access. In rural areas, reasonable access implies that a person does not have to spend a disproportionate part of the day fetching water for the families needs.
- Adequate amount of water: 20 litres of safe water per person per day.

[1] Water Supply and Sanitation programmes, DFID

[2] WHO, catalogue of Health Indicators. Geneva.

- Safe water: Water that does not contain biological or chemical agents directly detrimental to health.

46% of the rural population of Uganda for example does not have access to safe water (UNICEF).

To ease the burden of the above points, a DRWH system, which incorporates a handpump as shown in Figure 1, can be used to supplement a household's daily need during this dry season.

2.1 Specifications

The following specification has been drawn up to represent the particular conditions under which a handpump will be used:

- The handpump must be of low cost (i.e. affordable by low-income households in Uganda, with a maximum cost of UGS 18,000 ≈ $US10).
- It must be possible to manufacture and maintain the handpump within E. Africa at village level with a set of basic hand tools.
- The handpump should be capable of raising at least 10 litres per minute from a depth of 3 metre.
- Reach water within 200mm of the bottom of a tank.
- It should have good durability i.e. capable of lifting a minimum of 100,000 litre before requiring replacement (based on a family of five people with a 20 lpcd for three years).
- Only require basic maintenance - say every 10,000litre before requiring maintenance.
- The footvalve must not leak faster than 0.1 litre per minute.

In addition, it is desirable, but not essential, that handpumps have the following characteristics:

- Be reasonably secure against children pushing stones or pouring liquids into the outlet.
- No part should be easily stolen or removed.
- The outlet must be at such a height that most collection vessels, especially jerricans, can be easily filled.
- It must be ergonomically suitable for a child of about 6 years old to use comfortably.
- Be capable of fitting various types of tank covers, including ferro-cement covers (dome), and through a parapet wall.
- Permit the rising main and footvalve to be withdrawn for maintenance purposes.
- Suitable for production by artisans as an income generating activity.

3. Review of water lifting techniques and selection of candidate pumps

There are four different mechanical principles of transferring water from one location to another and these are shown in Table 1. These can range from simple devices such as scoops to more complex centrifugal pumps.

For the first three methods given in Table 1 these can be further subdivided in to rotary and reciprocating categories, for a taxonomy of pumps see Appendix 2.

Table 1 Summary of four mechanical means of lifting water

Direct lift:	By using a container to physically lift the water
Displacement	Water can be regarded as incompressible and can therefore be displaced
Creating a velocity head	Flow or pressure can be created by propelling water at high speed
Using the buoyancy of a gas	Passing air bubbles through water will raise the level of the surface

(Fraenkel, 1997, p29)

Rather than go in to any detail here an outline of techniques for lifting water in the low head, low flow rate range are summarised below. For a more detailed account, these are well documented by Fraenkel (1997).

To briefly discuss the most common types of low head, low flow capacity lifting devices the following descriptions are given:

3.1. Direct lift

Many of the direct lift methods of lifting water require open access to the water surface, i.e. buckets or containers on ropes or a lever for mechanical advantage supported on a frame. Persian type wheels rotate scoops or buckets in to the water, which transfer the water on the down side of the rotation. These can be employed in small-scale irrigation and to fill cattle troughs. The construction of these is simple and basic requiring a very low skill level.

3.2. Displacement pumps

Lift and suction pumps fall in to the category of displacement pumps. These rely on a piston, which is close fitting within a cylinder containing water. Lift pumps physically lift the water that is above the piston up the pipe to the outlet. Suction pumps have the piston above the surface of the water. By lifting the piston a vacuum is created which displaces the water up the pipe. A one way footvalve is needed to stop the water in the pipe from flowing back in to the well/tank. Figure 2 shows the basic principles of lift, suction and displacement pumps.

3.3. *Suction pumps*

Suction pumps rely on a piston seal within the cylinder. On the upstroke a pressure difference occurs between the air at the water level and the air in the cylinder chamber. This forces water in to the cylinder, which gradually rises on each successive stroke. The annulus or gap between the piston and the cylinder will affect the performance of the pump. The annulus needs to be at a minimum or even have some interference, and may be lubricated in some

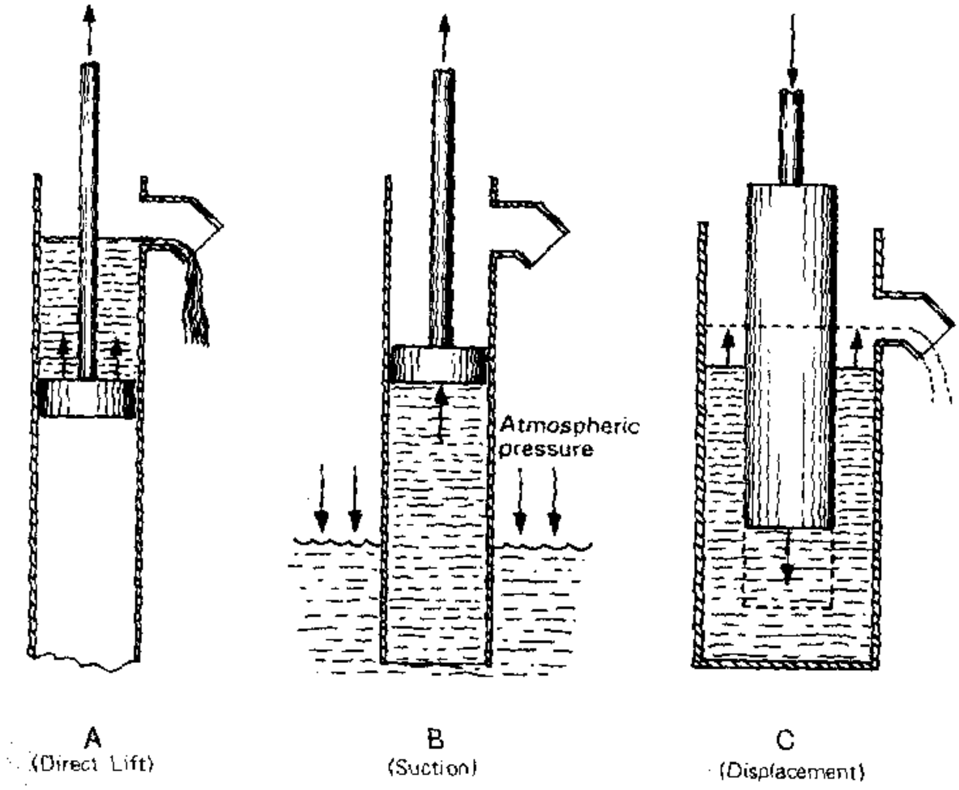


Figure 2 Basic principles of positive displacement pumps

cases to reduce friction. Priming may be required to get a pump to work, because water is more viscous than air it helps to improve the seal during the first few strokes. Priming can be achieved by physically pouring water in to the piston chamber or by retaining water in the chamber during non-operation of the pump. The latter requires a footvalve that does not leak or leaks at such a slow rate that the chamber is not emptied before the pump is used again.

There are limits to how high the suction lift can be. In theory, this is 10.4m at sea level, and in practice, 6.5m is a more practical limit. This will be further reduced by increased temperature of the water and higher elevations. For example, an increase in temperature from 20° to 30° will reduce the suction head by 7%, and for an elevation of 1500m the maximum suction will be around 5m [Fraenkel 1997, p14]. As a general rule for every thousand metres of elevation a loss of 1m suction head will apply.

3.4. Lift pumps

Lift pumps have some similarities with suction pumps in their components but differ in the position of the piston. For lift pumps the piston is below the surface level of the water, and by raising a handle, connected to the piston via a pull rod, water can be drawn up the rising main.
For lift pumps, it is preferable that there is a good fit between the piston and the cylinder but it not as critical as it is with suction pumps.

As Fraenkel relates there is a basic relationship between the discharge rate (Q), the piston diameter (d), the stroke length (s), the number of strokes per minute (n), and the volumetric efficiency (η_{vol}). The volumetric, or hydraulic, efficiency is an indicator of the actual discharge over the swept volume per stroke.

If the swept area of piston is A	$= (\pi d^2)/4$
Swept volume per stroke, V	$= As$
Discharge rate q	$= \eta_{vol} V$
Pumping rate per min Q	$= nq$
Then Q	$= 60n\eta_{vol} s\pi d^2/4$

The term slippage is sometimes used and refers to the difference between the swept volume and the actual discharge per stroke:
Slippage $X = V - q$
Slippage arises partly because the valves take time to close, they are often still open when the piston starts its upward travel, and because of back leakage past the piston or valve seats. Slippage is therefore normally less than unity, typically 0.1 or 0.2; it tends to be worse with shorter strokes and higher heads (Fraenkel, 1997, p38-39).
In some pumps the volumetric efficiency can be greater than 1. This arises in particular pumps that use the inertia of the water to raise an amount of water. As the column of water is accelerated upwards, it has inertia that keeps the column rising for a short time while the pump is being pushed downwards while the valve remains open. Therefore, the volume of water discharged is greater than the actual swept volume of the piston.

4. Manufacturing environment, competition and materials choice

4.1 Review of handpumps in Mbarara

Figure 3 Semi-rotary handpump

A search of hardware shops in Mbarara, Uganda was carried out to find what types of handpumps were available. The only one found was a semi-rotary type, as shown in Figure 3, and was made in Czechoslovakia. This consisted of a heavy cast iron chamber with a series of internal brass valves. The pump operates through rotating the handle repeatedly through approximately 120°. The pumps are generally very stiff to operate, and pumping is very exhausting work beyond ten minutes. The handpumps cost UGS600,000 (GBP250), no performance data was available with the pumps. This handpump is deemed too expensive and regarded as too difficult to operate, certainly by a child.

4.2 Manufacturing capabilities and materials available in Uganda

A reasonably thorough search of Mbarara, and to a lesser degree other towns, was carried out to find what trade outlets and manufacturing facilities were available which may be drawn on for the purpose of developing handpumps.

Like many Ugandan towns there are a large number of hardware shops dealing in a wide range of hand tools and plumbing fittings of reasonable to good quality products. There were also many steel stockholders and builders merchants in most towns visited. The steel stockholders did not have any stainless steel or brass sections in stock but some were willing to secure an order from Kampala.

Only one engineering workshop capable of any precision engineering was found in Mbarara. This consisted of a centre lathe, drill press, an off-hand grinder and one machinist.

There are many carpentry/joiners located in most towns, and mainly produced beds and cabinets, some of the larger establishment had wood lathes, and were capable of very high quality of craftsmanship. Also, there are plenty of roadside welding facilities available, usually fabricating burglar bars. For a list of common materials, tools and accessories found within a typical market in Uganda see Appendix 3.

4.3 Suitable materials for the rising main and cylinder

A durable, light weight and corrosion resistive material would be ideally suitable as a means of conveying water from the tank. The material must also be capable of being processed with simple and basic tools. This would rule out steel pipes as they are difficult to process without expensive equipment. One material that is widely available, non corrosive and lightweight is PVC.

4.4 The use of PVC as a suitable material for handpumps

There are several valid reasons for using PVC for handpumps, though there are some drawbacks as well. Table 2 gives some advantages and disadvantages of PVC. There is a range of PVC pipes available in E. Africa. These are thin walled low quality with no manufacturing marks for identification.The use of PVC has widely been accepted as a suitable and safe material for use with drinking water As Michael Dudden of the Consumers' Association Research & Testing Centre (CARTL) quotes:

> "The UK Drinking Water Inspectorate, the Swedish Environmental Protection Agency, the Swedish Water and Waste Waterworks Association, the World Health Organisation and the Organisation for Economic Co-operation and Development have confirmed the safety of PVC pipe. All these organisations have approved the use of PVC pipes to carry potable (drinking) water"

Table 2 Advantages and disadvantages of PVC for handpumps

Advantages	Disadvantages
Non-corrosive (esp. in aggressive water conditions)	UV degradation (causes embrittlement)
Light weight	Low impact strength
Low cost	Above ground parts may be subject to high forces: from animals using the pump as a scratching post, pipes being used as a resting post or being accidentally hit with full jerricans and possibility of malicious damage.
Flexibility (i.e. heat manipulation,)	
Ease of transportation (easily carried by bicycle)	
World wide availability	
Secondary uses (recyclable)	
Low cost joining ability (solvent welding)	
Non toxic (through usage) or taste tainting	

5. Calculations of power and efficiency

5.1. Power required from specifications

Determining the power required to operate a handpump is important for both its efficiency and to match the prime mover. The power capabilities of humans at various ages and durations are shown in Table 1 (Fraenkel, 1997, p118). As we are only interested in lifting 20 to 40 litres at a time, the first column is of most relevance.

Table 3 Power capabilities of human beings

Age	**Human power by duration of effort (Watts)**					
Years	5 min.	10 min.	15 min	30 min	60min	180 min
20	220	210	200	180	160	90
35	210	200	180	160	135	75
60	180	160	150	130	110	60

(Fraenkel, 1997, p118)

Table 4 Handpump specifications

Detail	**Symbol**	**Units**	**Value**
Flow rate (discharge)	Q	litre s^{-1}	0.167
Head (maximum)	H	m	4
Inside diameter of riser	d	m	32×10^{-3}
Stroke length	l	m	0.3
cadence	n	Cycles s^{-1}	1.167

To determine the power required for the handpump operating under the specifications in section 2.1, and shown in Table 4 the following calculations show that if:

$P_0 = E.n$

where: P_0 = power (water Watts), E = output energy, n = cadence in strokes per second

and $E = mgH$

where: m = mass of water lifted per cycle, g = gravity, H = head

$m = v.\,\boldsymbol{r}$

v = swept volume of stroke, ρ = density of water

Therefore the swept volume of half cycle is:

$v = \pi . r^2 . l \quad = \pi \ (19.5 \times 10^{-3})^2 . \ 0.3 = \mathbf{3.58 \times 10^{-4} \ m^3}$

$E = 3.58 \times 10^{-4} \ m^3 \times 1000 kg \ m^3 \times 9.81 ms^{-2} \times 4m = \mathbf{14J}$

Therefore: $P = 14J \times 1.167 \ s^{-1} = \mathbf{16.38 \ Watts}$

If the pump is 40% efficient then the power input P_i = **41Watts**

From Table 3, it can be seen that a 20 year old human is capable of producing 220 Watts effort for a duration of 5mins. From this, we can see that for the power required for lifting water, at the given specifications, a direct lift type handpump would be suitable.

5.2. Losses in the system

It is inevitable that there will be losses for any pump and its prime mover, however for the purpose of this project the pump is the main concern. It takes power to lift the water and to overcome any losses in the system. These losses may be mechanical, hydraulic or combination of the two. The following list shows sources of power losses in a pump:

- Friction in straight pipes (hydraulic)
- Friction from sliding components (mechanical)
- Leakage through pipes and badly sealing valves
- Flow friction through valves
- Headloss at changes in cross-section or flow direction
- Water leaving the handpump has kinetic energy
- Valve operation (delays in opening and closing causes losses)

5.2.1. Pipe friction

To get a reasonable and quick value for frictional losses it can be easier to use charts (as shown in Appendix 4). Using the chart method for a flow rate of 0.3 litre s^{-1} and an internal pipe diameter of 32mm, the headloss equates to about 0.58m per 100m. This is for cast iron pipe and a modifying factor for smooth PVC pipe is given as 0.8, which gives 19mm for a 4m head. Therefore pipe friction at these low flow rates and low head can be regarded as a negligible. But if smaller pipes are used higher frictional head values will be found, for example a 20mm PVC pipe will have 200 mm headloss loss for the same flow rate.

6. Selection of suitable handpumps

From the taxonomy of pumps shown in Appendix 2 it can be seen that there are a number of pumps that are suitable with head ranges far beyond the 4m limit given in the specification at the beginning of this report. The main types that are within the specification are the direct lift reciprocating/cyclic types.

Because of the open access to the water surface for lowered 'container' type lifting devices these incur a high risk of contamination from the container. Moreover there is also a potential for mosquito breeding in any tank without a permanently sealed cover.

For the 'Persian wheel' types the physical size of the tanks makes it unsuitable for abstracting water.

The rotary velocity pumps (propellers, mixed flow, etc) are suitable for the required head but demand a high degree of manufacturing process and precision, which would take the handpump beyond the $10 cost. In addition, the manufacturing capabilities in Uganda or most of E. Africa are not adequate for this at present.

This leaves generally the suction and lift pumps and possibly the rope and washer pumps.

7. Four designs of handpump

From the materials, tools and manufacturing search carried out in Uganda as well as the points made in the above sections a suction pump and three lift pumps were chosen.

The suction pump was based on the Tamana handpump developed in Sri Lanka, which makes use of standard PVC pipe fittings. The three lift pumps chosen were:

- The DTU handpump. A simple bicycle pump modification using a leather washer as the piston (Thomas T, *et al*, 1997).
- The 'Harold' pump which uses a non-contacting simple moulded cup (Whitehead, 2000) and does not rely on any fine precision to produce a lifting action.
- An Enhanced inertia pump which has no piston and relies partially on the inertia of the water in the system.

Details on the manufacture of these four pumps are not included in this report as they are detailed in technical release No TR.-RWH 09 (Whitehead, 2000).

7.1. *The DTU Handpump*

The DTU Handpump, (an exploded view is shown in Appendix 5) is a simple lift pump and uses a leather stirrup-pump piston, which is available from most cycle shops. The principle of operation is as follows: As the handle is lifted, the water above the leather washer is lifted with it. During this stage, the footvalve is opened and the water fills the rising main below the leather piston. On the downstroke, the footvalve is closed and the water in the lower section bypasses the leather washer to the upper section. Repeating operations transfers water to the outlet. During operation of the handpump, water continues to be discharged from the outlet even on the downstroke: this is because the volume of the push rod displaces water within the rising main.

7.2. *The Tamana Handpump*

This slightly modified version of the Tamana handpump, (an exploded view is shown in Appendix 6) is a suction pump. The pump relies on a seal between the piston-valve and the bore of the PVC cylinder.

During the upstroke, the piston-valve closes (flat on a PVC support), this creates a negative pressure below the piston, and this draws water into the cylinder through the footvalve. On the downstroke, the piston-valve is opened and water flows through the holes in the support to the cylinder above the piston-valve. On both strokes water is discharged through the outlet, as with the previous handpump the volume of the pull rod displaces water within the cylinder on the downstroke.

Labyrinth seals (a series of seals) can increase the performance of the seal. This version uses only two as a demonstration but more could be added. A suitable length of 1½ PVC pipe is connected to a reducer at the bottom of the cylinder and leads in to the DRWH tank where a floating valve is used for the intake.

7.3. *The "Harold" handpump*

The Harold handpump is a lift pump (an exploded view is shown in Appendix 7), but differs in the fact that it does not rely on a seal or a flexible membrane within the rising main. The piston, as such, is a moulded plastic cup, which is slightly smaller than the bore of the rising main. This is shaped in such a way that it has greater resistance to leakage on the up stroke and water is lifted by the cup. A small, but acceptable, amount of water will leak past the annulus around the cup. If the cadence is very slow, the leakage past the cup will be large.

The sequence of operation is shown in Figure 4, on the upstroke a), the footvalve opens allowing water into the rising main. On the downstroke b), the footvalve closes, and the water within the rising main flows around and above the cup. Repeated operation c) lifts water to the outlet. Very little water is displaced on the downstroke because of the small volume of the pull rod.

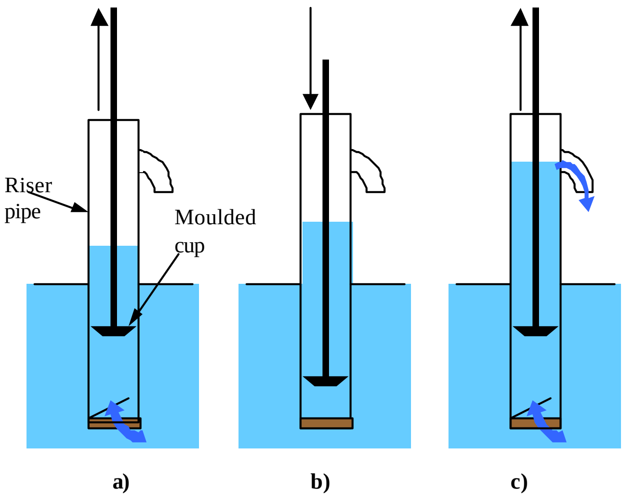


Figure 4 Sequence of operation for the Harold pump

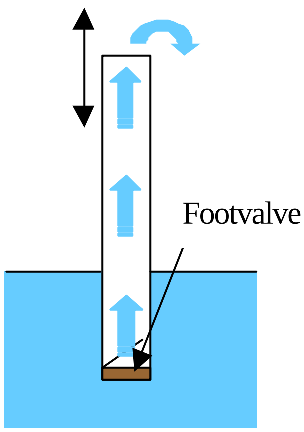


Figure 5 A 'joggle pump

7.4 The Enhanced inertia handpump

This pump, (an exploded view is shown in Appendix 8) does not rely on a seal within the rising main, but uses a central tube to lift the water, which overflows in to the rising main.

To explain the principle of operation it is first easier to see how the 'joggle' pump works. If an open top pipe with a footvalve is moved rapidly up and down the inertia of the water will gradually discharge water as shown in Figure 5. One limitation to this is that it will not work at very slow cadences.

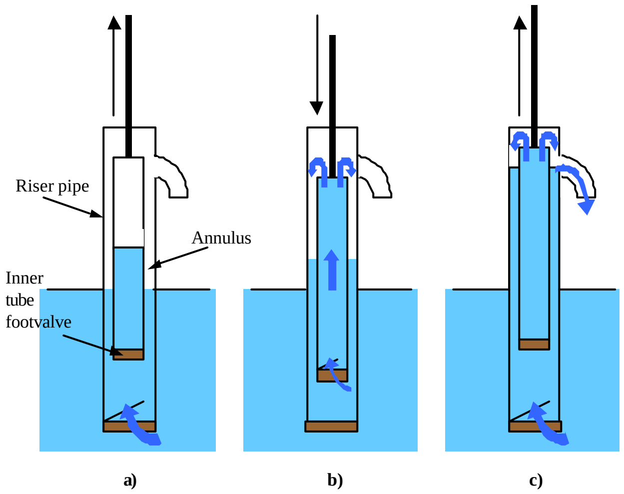


Figure 6 Sequence of operation for the Enhanced inertia pump

By combining this principle with an

outer tube, also with a footvalve, an enhanced principle is observed. A good commercial example of this is the "New Zealand Pump" (www.nzpump.co.nz). A very simplified sequence of operation is shown in Figure 6. During the upstroke a), the inner footvalve is closed and the main footvalve is opened letting water in to the rising main. On the downstroke b), the inner footvalve is opened letting water in to the central tube, meanwhile the main footvalve is closed. Repeated operations c) gradually brings water up the central tube, this then flows through a series of holes in the central tube in to the rising main and is eventually discharged at the outlet.

This handpump seems to operate best when short fast strokes are used. The flow is similar on both strokes of operation, again because of the high displacement from the central tube, which is full of water on the downstroke.

8 Critical components common to all four designs

8.1 Surface roughness and roundness of cylinders

The DTU and the Tamana handpumps rely on a good seal within the rising main cylinder, therefore it is preferable that the surface of the cylinder is as smooth and as round as practically possible.

To determine the smoothness of the bore several samples of uPVC pipe, from different hardware outlets in Uganda, were checked for surface roughness at the Centre for Micro-Engineering and Metrology at the University of Warwick.

At this level the surface roughness is expressed by its Ra value, and uses units in the µm range. Using a pump cylinder with as smooth a bore as possible can reduce the amount of friction (and subsequent wear on the piston) which the user may directly feel as a force to overcome by additional effort. The wear rate will also depend on the hardness of the material used for both the cylinder and the piston seal. A rough pipe surface (a high Ra value) can quickly wear the piston seal and reduce its out flow rate and hence its efficiency.

Table 5 shows the mean surface roughness of two sample cylinders for several popular handpumps available in the early 1980’s.

Table 5 The Ra value of several handpumps with various cylinder materials

Handpump Name	Cylinder material	Ra (mm) Sample 1	Ra (mm) Sample 2
Vergnet	Machined steel	0.57	0.60
Rower	Extruded uPVC	0.55	0.58
Volanta	Glass reinforced plastic	0.57	0.75
Briau Nepta	Extruded brass	0.06	0.21
Bangladesh No 6	Machined cast iron	2.40	2.40
Ethiopia BP50	Extruded uPVC	0.60	1.50
Vew A 18	Chromed brass	0.17	0.18
Bandung	Enamelled steel	0.33	0.60

Compiled from World Bank Technical Paper No 19

In comparison to the Ra values for the manufactured cylinders, Table 6 shows the results of two tests (carried out in the Centre for Micro Engineering, University of Warwick 9/11/00) on five different batches of uPVC obtained in Mbarara, Uganda. Tests 1 and 2 are the values from the same sample on two different areas. Table 6 Ra values from uPVC purchased in Uganda.

Table 6 surface roughness values from Ugandan purchased PVC pipes

Sample No	Ra (mm) Test 1	Ra (mm) Test 2
1	7.00	8.93
2	1.89	2.75
3	1.70	1.80
4	5.38	8.81
5	2.80	2.16

This shows, with the exception of the machined cast iron, the values of the uPVC from Uganda are all higher than those shown in Table 5.

The consequence of having a high surface roughness is that the performance of the handpump will diminish over time rather than preferably remaining reasonably constant. The peaks of the surface will abrade the outside of the piston and decrease its diameter.

9 Selection of two handpumps out of the four designs

To assist in the selection of two candidate pumps, any manufacturing difficulty or specific skill level, as well as the amount time required need to be considered.

9.1 Ease of manufacturing the four handpumps

As mentioned in section 2.1, it must be possible to manufacture and maintain the handpump within East Africa at village level with a set of basic hand tools. Table 7 shows a comparison of the manufacturing time, the number of tools required and the skill level required for manufacturing the handpumps. All the tools used to manufacture the handpumps were sourced from the local market.

Table 7 Manufacturing time and skill level required for the four designs

	DTU	Tamana	Harold	Enhanced Inertia
No of tools required	8	8	10	9
Time to manufacture (hrs)	4	4	3	2
Skill level required	high	Medium	Low	Low
Total number of parts	13	15	13	12
Technicians preferred choice[c]	-	-	1 x 1st choice 8 x 2nd choice	9 x 1st choice 1 x 2nd choice

(Whitehead, 2000)

[c] Based on 10 technicians choice after completing the manufacture of four pumps at a two day workshop at Kyera Farm, Uganda 23rd August 2000.

9.2 Pros and cons of the four designs

To assist in the selection process a review of the four handpumps was carried out, and the responses from technicians who attended the training workshop in Mbarara, Uganda August 2000 were also considered. The benefits and drawbacks of the four designs are given in Table 8.

Table 8 Benefits and drawbacks to the four handpumps

The DTU handpump	Pros:	Cons:
	• Low No. of tools required • Parts which need replacing are low cost and easy to obtain •	• Time to manufacture is long compared with other the handpumps. • Skill level for manufacture is high • Removal of handpump for repair is time consuming • Fairly high resistance during operation • The leather is susceptible to wear • The output for the input effort was low
The Tamana handpump	**Pros:**	**Cons:**
	• Removal of the pump is easy as it is separate from the tank • High output of water • Skill level required is medium • Very low cost • Very low additional cost per metre • Low no of tools required • Positioning of handpump is ergonomically better for most users	• The rubber pistons wear very quickly • Priming is required if the level of the water is lower than the cylinder • High resistance during operation • Cutting pistons to correct size is time consuming • Manufacturing time is comparatively higher
The "Harold" handpump	**Pros:**	**Cons:**
	• Lower manufacturing time than the previous two handpumps • Very little effort required for operation • Low skill level for manufacture • Expected reliability is good • Lower No of parts	• Highest No. of tools required • Pull rod prone to corrosion • Lower hydraulic efficiency because of gap round the moulded cup • Removal of handpump for repair is time consuming
The Enhanced inertia handpump	**Pros:**	**Cons:**
	• Low manufacturing time • Very little effort required for operation • Low skill level required for manufacture and maintenance • Very good expected reliability • Most favoured to make and use • Small fast stroke length gives a relatively steady flow rate • Acceptable hydraulic efficiency at operators preferred cadence	• Higher cadence required • Most expensive to manufacture • Lower output than other pumps • High additional cost per metre for deeper tanks • Steel screws for the flap valve prone to corrosion (no stainless screws found)

9.3 Costing of the handpumps

A limit of $10 was set as a maximum cost for a handpump as this represents a significant proportion (30%) of the total cost of a plastic tube tank (Rees, 200).

A cost comparison of the four handpump designs was carried out and this showed that all four designs could be manufactured for less than $10 for a 3.5m length pump. It can be seen that there is a significant increase in the cost for each metre added to the length of certain pumps. The individual costs for three lengths and cost per additional meter are given in Table 9 (Whitehead, 2000)

Table 9 Cost comparison for varying length of handpumps

Length	DTU ($)	Tamana[b] ($)	Harold ($)	Enhanced inertia ($)
1.5m	6.50	7.25	4.86	5.52
2.5m	8.14	7.89	6.16	7.68
3.5m	9.78	8.53	7.46	9.84
Additional cost/m of handpump	*1.64*	*0.64*	*1.30*	*2.16*

(Whitehead, 2000)

[b]This includes the footvalve and pipe work in to the tank.

This clearly shows that the Tamana is much lower cost per additional metre than the other handpumps. This arises because the only additional cost is the 1/2" uPVC pipe in to the tank. Compared with the Enhanced Inertia the Tamana is 60% lower in cost per metre.

From the four proposed designs, a selection of two handpumps were chosen on the balanced merits of performance, expected reliability, low precision demand and ease of manufacture as expressed by technicians trained in handpump manufacture in Mbarara, Uganda. The selection process eliminated the DTU and Tamana handpumps for the following reasons.

The DTU handpump gave the lowest discharge rate of the four pumps and the following points show that:

- The force required to operate the handpump was comparatively high.
- The pull rod is prone to buckling, at higher cadences (possibly leading to localised wear).
- Retaining the leather washer on to the pull rod is difficult.
- The leather washer became saturated after a short time and could eventually disintegrate.

- The surface roughness for uPVC pipe was high and would wear the piston.

The Tamana did have the highest discharge rate from the Ugandan performance tests, but summarising the following points, the Tamana showed that:

- The surface finish in the PVC bore was variable.
- The diameters of the pipes are inconsistent.
- The roundness of the pipe could not be guaranteed.
- Rapid wear occurred in the piston valves because of the surface roughness.
- Priming is necessary when the water level is lower than the bottom of the cylinder.
- It was one of the least preferred handpumps to manufacture.

This gave sufficient reason to eliminate the DTU and Tamana handpumps. The Harold and Enhanced inertia handpumps were considered more suitable for a number of reasons, these were:

- Neither of the pumps required any fine precision.
- The manufacturing times were much less.
- A lower skill level was required for manufacturing them
- The reliability was expected to be higher
- They were preferred choice of the technicians.

10 Valve Design and leakage tests

A footvalve is required so that the cylinder retains the water during the downstroke of the piston. There are many styles of valves which operate in different ways, for this project a simple design was required which could be made from easily obtained materials and be made with a set of basic tools. The first design is the DTU valve (Thomas *et al*, 1997), which is made from PVC pipe and a strip of rubber. The second is the Low cost valve (Whitehead, 2000) which is made from a wood and a small rubber disc. Wood was chosen because it is easily obtained, very low cost and is simple to work with. Both valves are shown in Appendix 9.

An ideal valve will have zero 'forward' flow resistance and infinite 'reverse' flow resistance. It will also have an instantaneous response, as the pressure gradient reverses, when opening and closing the valve. Two tests were carried out on the Low cost valves. Firstly, the ratio of the sum of inlet holes area to the pipe area was varied to see if this affected the flow. Secondly, the rate at which the valve leaked was found from a simple test. The DTU valve was only tested to determine the leakage rate. Table 10 shows the dimensional values for the pipes and the inlet holes for three 1mm incrementally larger sizes.

Table 10 Ratio of sum of inlet hole areas to the total inlet area (low cost valve)

Æ 1 1/2" - 40mm pipe	Units	No 1	No 2	No 3
Inside diameter of pipe	mm	34.25	34.25	34.25
Area of inner pipe bore	mm^2	921	921	921
Diameter of inlet hole	mm	6.0	7.0	8.0
Area of inlet hole	mm^2	28.3	38.5	50.25
No of holes in inlet	No	5	5	5
Flow passage ratio =		0.15	0.21	0.27
Æ 1 1/4" – 32mm pipe	**Units**	**No 4**	**No 5**	**No 6**
Inside diameter of pipe	mm	29.75	29.75	29.75
Area of inner pipe bore	mm^2	695	695	695
Diameter of inlet hole	mm	6.0	7.0	8.0
Area of inlet hole	mm^2	28.3	38.3	50.25
No of holes in inlet	No	4	4	4
Flow passage ratio =		0.16	0.22	0.29

The test on the low cost valve was carried out by operating the pump at different cadences and recording the time to fill a 5 litre container. The results of these are shown in Table 11.

Table 11 Results of low cost valve inlet ratio test

Test	Cadence cycles/min	Time to fill 5 litre container (seconds)
Valves: No1 & No4	40	58
Valves: No2 & No5	40	58
Valves: No3 & No6	40	55
Valves: No1 & No4	60	37
Valves: No2 & No5	60	38
Valves: No3 & No6	60	35

This shows that the ratio has almost negligible affect on the flow out of the handpump at these cadences. No detectable change in effort was felt by the operator as the inlet holes were varied.

A larger size hole may eventually collapse if the wall section between the inlet holes is too thin. It was observed that the inlet holes, after approximately 48hours, showed signs of becoming oval. This is arises because of the wood swelling and compressing perpendicular to the grain. A wood that resists water, or is little affected by it, should be used if available (i.e. in the UK Elm would be used). Alternatively, some method of protecting the wood could be done i.e. heating the inlet in food grade oil.

10.1 Valve leakage tests

Ideally, it is preferable that the handpump holds its prime so that next time the handpump is used the first stroke would discharge water. To achieve this the footvalve would have to seal perfectly, in practice, this would be almost impossible to achieve and from the specification we can tolerate a minimum leakage of 0.1 litre per min.

To determine the amount of leakage a series of short tests were carried out which involved filling the rising main with water and measuring the amount of water at timed intervals as it leaked past the valve. The valve end was placed above a container with graduated markings of 20 ml. At 15 second intervals the volume of water in the container was recorded. This test was carried out on both the DTU valve and the Low-cost valve. The graphical results of these tests are shown in Appendix 10, and these illustrate the different characteristics of both valves.

The graph of the DTU valve shows that the leakage rate actually rises (almost to a square law) with pressure across it. This suggests a roughly consistent leakage aperture. It was expected that the water pressure acting on the inner tube section over the perforated pipe would be greater at higher heads. Then at lower heads, the pressure would be less and the rate of leakage would increase but this was not the case.

The Low cost valve showed a more complex three-point characteristic. Initially at the higher head, leakage is high but falls as the pressure falls. Following this is a zone of almost constant leakage rate that is independent of pressure over a 0.5m range. Finally, the leakage rate rises over the last metre as the pressure falls. A fast leakage rate at the start may be because of some settling of the valve and/or some 'puckering' of the valve instead of laying flat over the inlet holes.

All this suggests that the leakage aperture varies with pressure. It was expected that the leakage rate would gradually increase as the pressure is reduced on the valve, and leaking faster as the head remaining tended to zero. The total time for the column to fully discharge was 6.5 minutes, showing that the low cost valve has a mean leakage rate twice that of the DTU valve. In both tests, the leakage rate varies with the pressure drop across the valve. The two mechanisms at work here are: a) higher pressure forces the water faster through the apertures in the valve body, b) the aperture size is reduced by the pressure forcing the valve flap harder onto the inlet holes in the valve body.

The low cost valve was chosen as the most suitable design mainly because the force required to operate the pump was significantly less than that for the DTU valve. The DTU valve performance depended on getting the right sized inner tube to the inlet pipe, older and less elastic tubes worked more efficiently. Whereas the response and efficiency of the low cost valve was much more desirable despite the lower leakage rate.

11 Performance tests

A series of performance tests were carried both in Uganda and within the laboratory at the University of Warwick. In Uganda, this consisted of some basic preliminary tests on four demonstration models to compare the handpumps performance. A spring balance was attached to the handpump handle to show the force required during the upstroke. A container was placed at the outlet, of known volume, and filled and the time subsequently recorded. The results given in Table 12, show that force required to lift water was 7 and 8 fold less for the Harold and Enhanced inertia handpumps over the DTU and Tamana handpumps respectively. The Harold and Enhanced inertia handpumps also showed lower flow rate output than the Tamana handpump, but higher flow rate than the DTU handpump.

11.1 Ugandan-based performance tests

Table 12 Performance comparison of the four handpumps

Variable	DTU	Tamana	Harold	Enhanced inertia
Internal diameter of rising main (mm)	39	39	39	39
Length of rising main (mm)	530	530	530	530
Stroke length (mm)	330	254	406	102
Kg force to lift water	8	7	1	1
No of cycles/jerrican	134	114	159	142
Output Litres/min	7.55	11.6	8.93	8.43
Minutes to fill 20 litre jerrican	2.65	1.91	2.24	2.37
Apparent vol. efficiency	0.38	0.58	0.26	1.16
Reliability[a]	low	low	Medium/high	High

(Whitehead, 2000)

[a] This is based on the limited field trials carried out in Uganda, and is the expected reliability: low = 2 months, medium = up to 6 months and high = 12 months.

The Table 12 shows that the volumetric efficiency of the DTU and Harold pumps are quite low. The volumetric efficiency of the Enhanced inertia is greater than unity. Though inertia type pumps, as mentioned earlier in section 3.4, can give a value greater than one it seems unlikely when there is a short column of water. There seems little else to explain this high value and a repeat of the test under the same conditions needs to be carried out to confirm this high value.

11.2 University based performance tests

The performance tests carried out at the University were achieved using the set up shown in Figure 7. During the tests the head, cadence and stroke length were varied over a suitable range. The time to fill a 5 litre container and the operators heart rates were recorded. Any comments by the operator were also noted. The results of the performance tests carried out in The University of Warwick are shown in Appendix 11

The performance tests had two main functions. Firstly, that both handpumps could be compared to each other show any differences in their performance. Secondly, to see what changes the variables have on the operator with respect to input effort. Three males and one female were used with ages ranging from 20 to late 30's.

The cadences used were 50, 60 and 70 cycles per minute, 40 cycles was used in the first tests but was regarded as too slow and consequently dropped from the remaining tests. The cadence that most operators preferred was 60 cycles/minute.

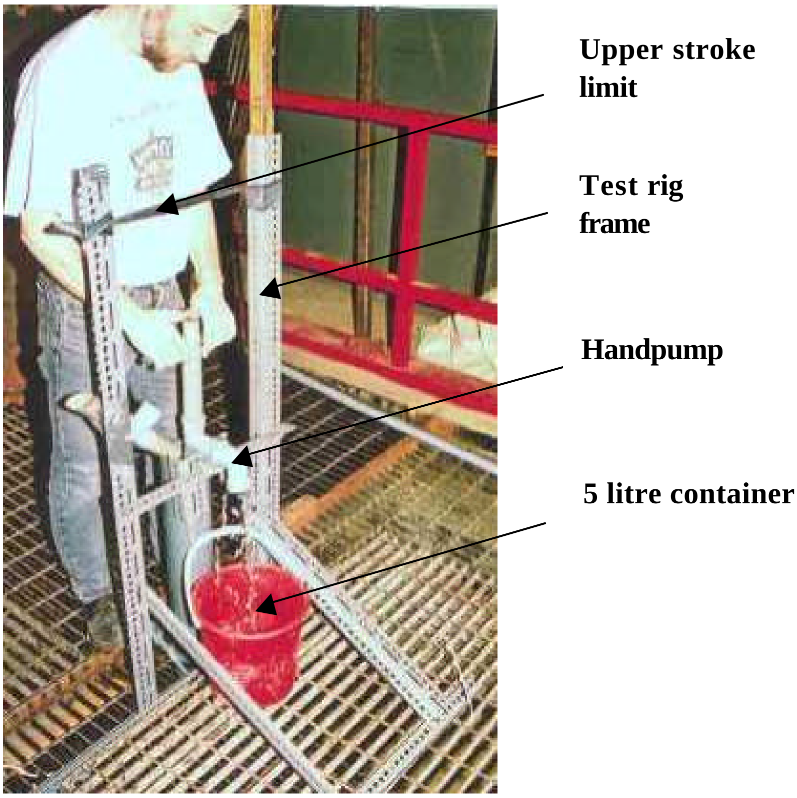


Figure 7 Performance test set up

It was expected that the flow rate and the volumetric (or hydraulic) efficiency would increase with higher cadence and longer stroke lengths, which it did. Though there is a limit to this, as it becomes increasingly difficult to operate at higher cadences with long stroke lengths. In addition, the returns on volumetric efficiency, for a higher cadence, are not worth the additional effort as will be seen later in section 12.

11.3 Heart rate monitoring

During the performance test each operator's heart rate was monitored with the aid of a standard electronic monitor worn around the chest as used by athletes. As the cadence and head was increased it was reasonable to expect an increase in the heart rate as well. This would give an indication of the amount of additional effort the operator had put in as the head and cadence were increased.

From the results in Appendix 11, it can be seen that there are some cases where the results are conflicting. For three of the operators, their maximum heart rates had increased by very much the same (avg. 12%). These had mainly occurred towards the highest heads and highest cadences. But one operator showed their highest heart rate increased on two occasions, firstly at the lowest head and highest cadence and secondly at a much lower cadence and a mid-range head.

The female operator showed a much larger increase in heart rate (33%), this had occurred at a higher cadence but also at a mid-range head.

In general, it has shown, given a small number of tests, that the increase in heart rate is small and did not show any of the operators to be expending much of there potential.

11.4 Moulded cup size tests for Harold handpump

To see how varying the diameter of the moulded cup affects the performance a short series of tests were carried out. This involved timing how long it took to fill a 5litre container at a cadence of 60 cycles and a 0.25m stroke length for five different diameter moulded cups. This showed that, as was expected, the volumetric efficiency increased as the cup size increased. It also showed that the effort required in pushing the handle down increased on the upstroke as the cup size increased. It may seem more desirable to either have the same or similar effort to operate the handpump on both the up and down strokes. To rectify this a series of holes were drilled around the cup and a valve incorporated. This had the desired effect but meant more work on the component was required. The results of the tests are shown in Table 10.

Table 10 Results of moulded cup tests for 0.25m stroke length

Cadence (cycles / min)	Bore diameter of pipe (m)	Diameter of moulded cup (m)	Time to fill 5 litre (s)	Flow rate (litre / min)	Upstroke effort (1 to 10)	Downstroke effort (1 to 10)	Volumetric efficiency
50	0.036	0.032	Too slow	-	0	1	-
60	0.036	0.033	240	1.25	1	2	0.08
60	0.036	0.034	70	4.29	2	3	0.28
60	0.036	0.035	30	8.57	2	6	0.56
60	0.036	0.036	25	12.00	6	9	0.79

11.5 Modification to designs

A modification to the design of the Enhanced inertia pump was required because UK-made pipe differed in size to that purchased in Uganda. This difference resulted in a reduced annulus and an unacceptable performance.

Figure 8 shows a cross-section of the Enhanced inertia pump clearly showing the annulus between the two pipes. The area of the annulus can be expressed as a ratio of its area to the outer pipes bore area. The annulus ratio for UK-made pumps was 0.14, and a higher value (0.17) was found for those made in Uganda. Even at slow to moderate cadences some of the water within the riser was unable to discharge through the outlet and vented through the annulus at the top of the rising main instead, as shown in Figure 9 a). This was solved by removing a 0.4m section of the upper central pipe and replacing it with a steel pull rod as shown in b). This was fixed to the

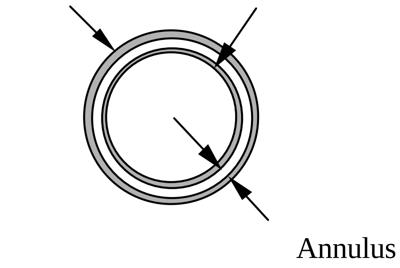


Figure 8 Cross-section of inner and outer pipes

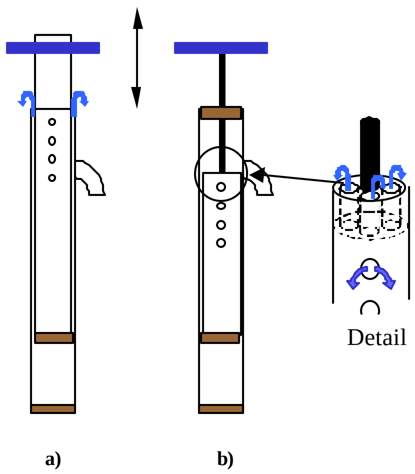


Figure 9 Modification of the pump and water flow path

inner pipe with a wood inlet as used in the footvalve. The details in Figure 9 b) shows the flow path through the wood inlet after the modification.

For a constant flow rate (Q) reducing the annulus area (A) increases the water velocity (v) (from mass continuity: Q = vA). Given the increased velocity caused by the reduced annulus area a significant fraction of the water continues the short distance to the top of the riser pipe and leaks out. By increasing the annulus, the velocity of the water is reduced such that all the water flows through the outlet.

One drawback to this modification is that using a steel pull rod will lead to corrosion and reduce the quality of the water. Stainless steel is not readily available in Uganda. Galvanising the pull rod may be an option as this process is used on corrugated roofing sheets in Uganda and would greatly reduce the level of corrosion.

12 Durability testing of the handpumps

It is important that the handpump performs satisfactorily over a period of time before the pump is either beyond repair or no longer lifts sufficient water for the household. A reasonable expected life for the pump had been decided in the specification as three years. If a family of five people use the handpump to abstract 20 lpcd over three years then this amounts to 109500 litre over the expected life of the pump. From the performance tests 15 litres / minute could be taken as a reasonable discharge rate, and this would equate to 122 hours or 5 days continuous use. To replicate this a durability test rig was designed and built at the University of Warwick to give a reciprocating motion powered by an electric motor and geared down through two variable speed gearboxes. The output speed could be varied between 17 and 400 cycles / min.

The output shaft of the final gearbox was attached to an arm 0.15m from centre of rotation, giving a stroke length of 0.3m. The head was set at of 2.65m. The reciprocating arm was linked to the motor arm and the pull rod with rod end bearings, This would allow for any slight misalignment in the motors rotational plane and the handpump's translational plane.

Because of the physical size of the test rig it was necessary to build it over an existing 2.5m deep pit in the Engineering workshop at the University. A sketch of the endurance test rig is shown in Figure 10 indicating the main components. The water discharged from the handpump was re-circulated back in to a large reservoir in the bottom if the pit, via a flow detection chamber. Because the discharged flow from the outlet is in a non-steady state and difficult to measure, the flow was diverted from the outlet in to a 15 litre container and the number of cycles to fill this was recorded with a tally counter. This was repeated during the test to show any changes in the outlet flow over the life of the handpump.

As the handpump was to run continuously over 5 days, there was a possibility that the motor would still run even if no flow occurred. Therefore, the flow detection chamber housed a horizontal float switch, and operated a relay to cut the power supply if the flow stopped.

A digital clock was fitted which showed the lapsed hours and minutes whilst flow occurred. As the flow rate per cycle is known a reasonably accurate number of litres pumped could be found.

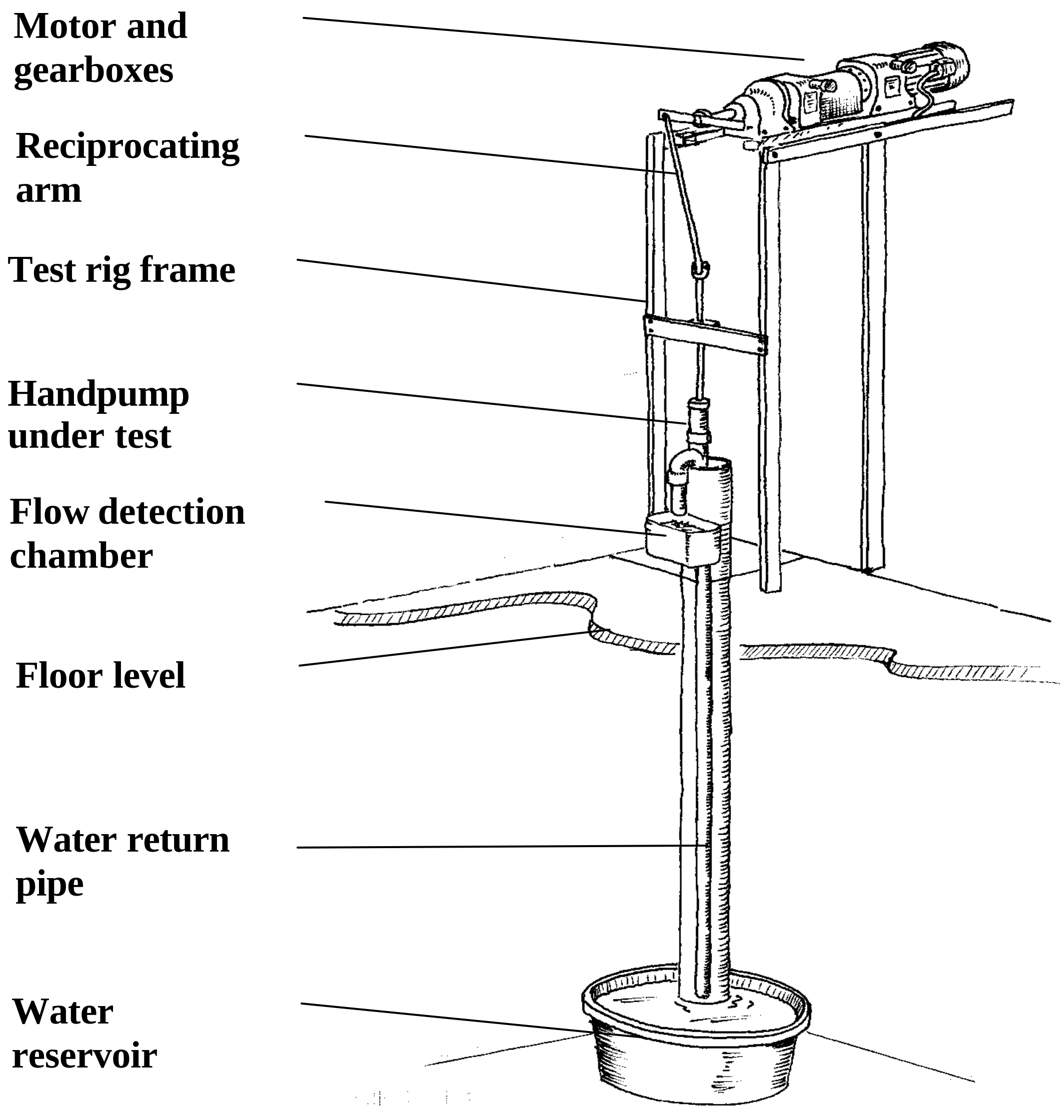


Figure 10 Endurance test set-up

The objectives of the durability test were to indicate the following points:

- The failure mode of the handpump
- Reduction rate of out flow over time
- Where any localised wear occurs and how much it may have worn by
- Time between failures
- Durability comparisons between each handpump

The Harold handpump was tested first and a number of dimensional checks were carried out before the test commenced. Firstly the moulded cup for the Harold handpump was measured across its diameter in three places (120°). This is because accurate roundness of the cup during manufacture can not guaranteed. The dimensions of the moulded cup are shown in Figure 11.

The hole size of the pull rod support bush was also checked, as this was regarded as high wear

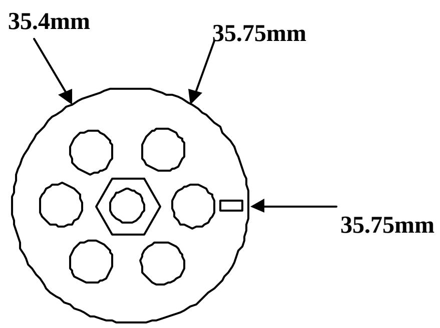


Figure 11 Dimensions of the molded cup at start of the endurance test

area. The diameter of the hole at the start of the test was ∅8.8mm.

The reason for failure most expected from the Harold handpump were that firstly, the wear in the moulded cup would reduce its performance until the flow fell below 10 litre per minute before the end of the endurance test. Secondly, one of the valves may fail (tears or splits) during its half-million plus cycles.

A two hours 'bedding-in' period was carried out prior to the tests so that any stiffness in the system may be reduced or that any problems with the set up could be detected and rectified. The results of the durability tests for the Harold and Enhanced inertia handpumps are shown in Table 12 and 13 respectively.

Table 12 Harold handpump durability results

Cumulative water lifted (litre)	Normalised Flow rate	Volumetric efficiency	Failure mode	Wear in piston Æ (mm)	Wear in rising main Æ (mm)	Diameter of hole in pull rod bush Æ (mm)
0	1.00	0.71	-	0	0	8.8
56,767	0.87	0.64	-	0	0	-
63,302	0.91	0.66	-	0	0	-
74,474	0.89	0.65	-	0	0	-
95,974	0.87	0.64	-	0	0	-
113,081	0.94	0.69	-	0	0	-
120,245	0.94	0.69	-	0	0	9.5

12.1 Observations of the Harold handpump

After completing 143 hours, and 590,000 cycles, of continuous running, the handpump was dismantled and the following points were observed:

- The inlet valve showed signs of indentation from the water pressure acting on the area of each of the five inlet holes (see Figure 12a)
- Moulded cup showed no visible sign of wear
- Stress marks evident on the edge of some of the holes in the moulded cup (see Figure 12b) N.B. these are not at the thinner sections between the holes but at the top of each hole.
- Moulded cup valve starting to show signs of being cut from the moulded cup holes (see Figure 12c)
- Bottom section of the handpump rising main was removed and no significant wear was detected, only small surface scratches on one side if the pipe.
- On removing the handpump, the volume of water remained in the rising main with a very low leakage rate. This was suspected to be fine debris settling and compacting under the valve and actually giving a better seal!

Figure 12 a) b) c)

It can be seen from the results in Table 12 that the volumetric efficiency had dropped by 7% during the first 57,000 litres, with a relatively level efficiency for the next 40,000 litres. After this, there is a rise to within 2% of the original efficiency. One explanation that could be given for this is that some particle may have become lodged under the moulded cup valve, of sufficient size to cause some back leakage. This then may have been dislodged before the last 25,000 litres. This would explain a lower volumetric efficiency and account for the reduction and final increase in the flow rate as shown in the normalised flow rate in Table 12.

12.2 Observations of the Enhanced Inertia handpump

Table 13 shows the results of the endurance test, which ran for 167 hours. During this period no mode of failure or decline in flow rate or efficiency was found.

Table 13 Enhanced Inertia handpump durability results

Cumulative water lifted (litre)	Normalised Flow rate	Volumetric efficiency	Failure mode	Wear in inner pipe Æ (mm)	Wear in rising main Æ (mm)	Diameter of hole in pull rod bush Æ (mm)
0	1.000	0.77	-	0	0	9.5
22,012	0.997	0.77	-	0	0	-
48,020	0.997	0.77	-	0	0	-
74,106	0.997	0.77	-	0	0	-
95,270	0.997	0.77	-	0	0	-
164,422	0.997	0.77	-	0	0	12.5

Inspection of the internal parts of the handpump, after the endurance test, showed little sign of wear either on the inner pipe or on the bore of the outer pipe. Both valves showed signs of deformation (similar to the Harold handpump) where the rubber had repeatedly been depressed into the wood inlet holes.

The only other indication was some localised surface scratches on the outer pipe bore where the inner pipe had contacted it during its 700,000 cycles. The only external part that had worn was the wood support bush for the pull rod, and this had become oval but of no detriment to the handpump’s performance.

Figure 13 shows the standardised flow rate against cumulative litre for both the Harold and Enhanced inertia pumps.

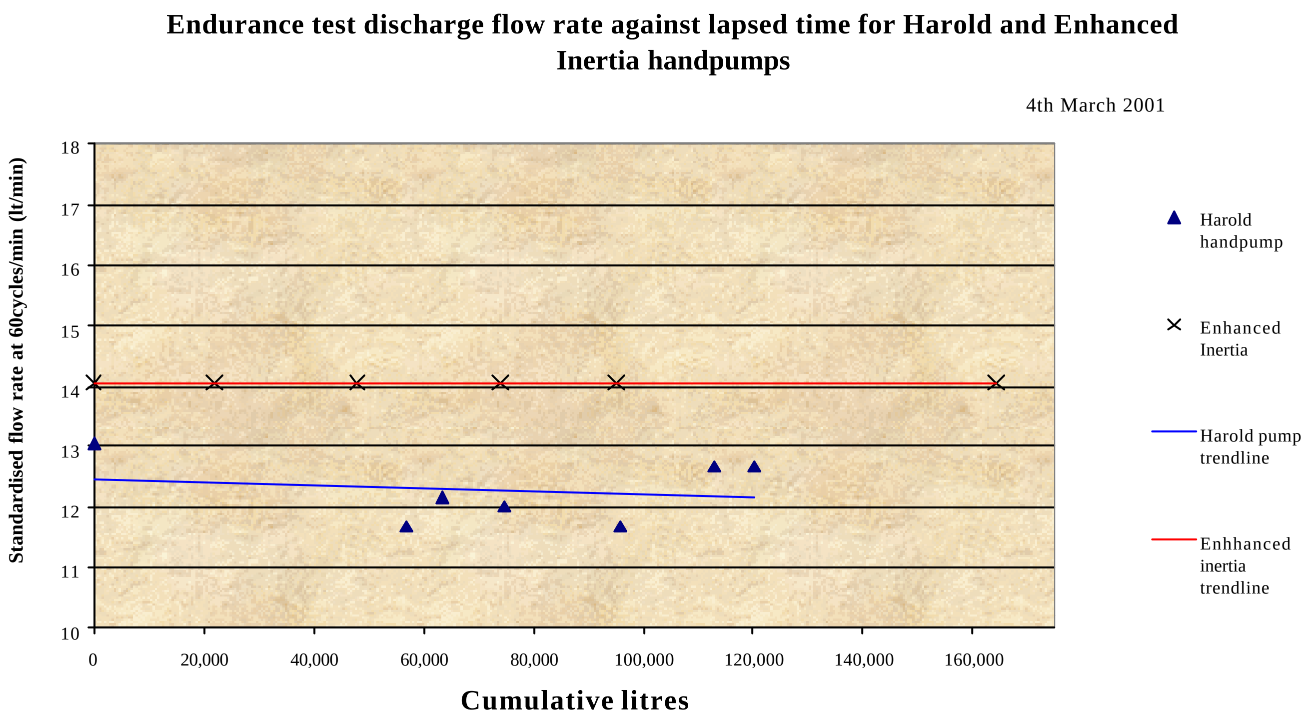


Figure 13 Comparison of the Harold and Enhanced inertia endurance tests

It was hoped that some failure or drop off in efficiency had occurred so that the handpump could be analysed and possibly improved. To try to cause a mode of failure the cadence was increased by 1.5 and run for a few hours. This resulted in a sudden failure rather than it being gradual as the riser pipe was forced off the outlet tee. The joint was thoroughly cleaned and re-cemented, and ran at the same cadence without any other failure for several hours.

Figure 14 shows a graph of volumetric efficiency vs. cadence for two stroke lengths, the dotted lines are the upper and lower limits for comfortable operation. This was done to see how a doubling of cadence and a halving of stroke length (which gives the same volume per unit of time) affects the efficiency. This shows for example at a cadence of 35 cycles per minute and 0.3 stroke length the efficiency is 62%, and if the speed is doubled and stroke length halved the efficiency is 60%, showing very little difference. Through the comfortable operating range, the longer stroke length is more efficient for a given cadence.

If the cadence were increased much beyond 70 cycles per minute an operator would find it difficult to maintain a 0.3m stroke length. The graph in Figure 15 shows the product of the two stroke lengths and cadence against efficiency. This shows that for lower cadences there is very little improvement in efficiency for given different stroke lengths. Though there is a tendency for the longer stroke length to show some slight improvement in efficiency at higher cadences.

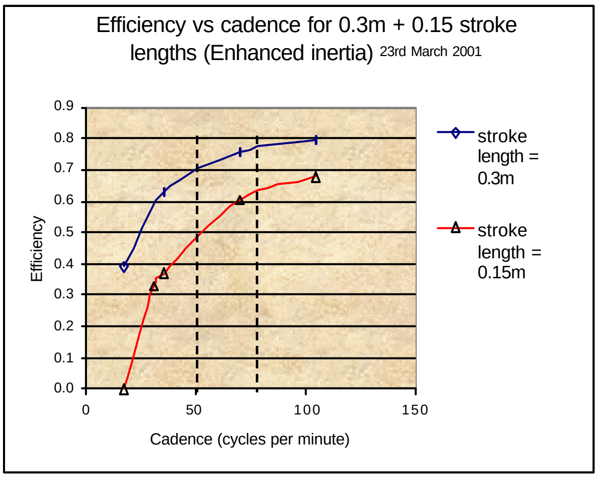


Figure 14 Graph of efficiency and cadence for two stroke lengths

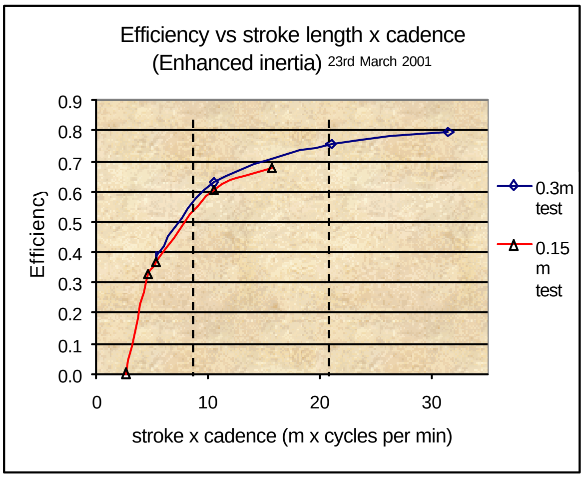


Figure 15 Graph for the product of stroke length and cadence vs. efficiency

In summary of the Enhanced inertia handpump endurance test: because there was no failure mode or a gradual decline in the flow or volumetric efficiency, very little can be said other than the handpump has been shown to be a very durable and reliable handpump. Also, that the handpump would need little maintenance and was more than capable of lasting three years under normal operating conditions.

Last minute update: after completing the Enhanced inertia endurance tests, the pump was further tested at a cadence of 70 cycles per minute and a 0.3m stroke length and was left to run until it stopped. The result of this was that it was still running after 267 hours, and had lifted 261,000 litres of water.

12.3 Safety aspects of the endurance tests

Because of the risk of injury to persons from the reciprocating motion and prolonged test period (two-weeks continuous running), a significant amount of thinking and work was done to assess and remove potential dangers as far as is practically possible. This involved some liaison with the Health and Safety Officer at the University and with the Chief Technician in the engineering workshop.

After consultation with the above, the main points considered and relative actions were as follows:

- Prior to building the test rig a PAT test was carried out by an electrical technician to ensure the electric motor was safe to use.
- Mesh guarding was put around all moving parts
- Bunting was put around the test rig area
- Electrical lights and conduits in the pit were checked as suitable for outdoor weather use
- Continuous running notice was attached to test rig (and relevant people informed: security)
- Power supply had:
  a) over current protection, b) no-volt drop out, c) earth leakage protection.
- A float switch was incorporated to detect handpump delivery flow: power cut out if no flow detected.
- Railings surrounded the pit, and bunting was put up within the pit on open floor level areas.
- Water flow was highly unlikely to reach 415v supply during testing

Table 14 shows possible modes of test rig failure and of the procedure or protection that was used to reduce the danger.

Table 14 Test rig failure and protection procedure

Area of failure	Protection/procedure
· Linkage/reciprocating arm/rod end breakage	· Flow from pump would stop and float switch will cut power to motor
· Frame and/or motor mounting bolts become loose	· Periodic checks to ensure tightness

13 Feedback from Uganda on training and handpumps installed on tanks

13.1 Training

Following a seminar on DRWH tanks built at Kyera Farm, which was attended by several members of NGO's from around E. Africa, a training workshop for handpump manufacture was held for ten representatives on August 22 - 23rd 2000. This involved each participant building four short demonstration handpumps. A handpump manufacturing manual and certificate was presented to each participant on completion.

Correspondence from Moses Byaruhanga, (a co-ordinator for URDT, Kampala) 17 weeks after the training course, stated that "so far from the knowledge we got from Kyera farm, we have trained another 20 new local masons in pump fabrication and repair"
A similar one-day workshop was held for the ten masons and labourers who built the tanks at Kyera Farm. One mason, who had attended the one-day workshop, was building a 25m^3 tank in Mbarara and had decided to make and install an Enhanced inertia handpump himself during September 2000 (no feedback on this at present).

13.2 Handpumps installed in Uganda

A Harold and an Enhanced Inertia handpump were installed on two plastic tube tanks (see Figure 13) built on Kyera Farm, Mbarara in August 2000. Both these handpumps were installed and made by the first group of trainees at the training workshop.
From returned survey forms for August to November on tank use the remarks for the Harold handpump were:

August:	Rust was evident in the abstracted water
September:	Not functioning
October:	Not functioning
November:	Functioning

The reason for not functioning for two months or how the handpump was repaired was not recorded. The Enhanced Inertia handpump functioned for four months without any problems.

A separate and more detailed survey from (see Appendix 12) was sent with specific questions relating to the Harold and Enhanced inertia handpumps performance. This was sent to Kyera farm in Mbarara, Uganda in December 2000, a summary of eight questionnaires returned showed that:

- The two Harold handpumps installed in August 2000 were used to fill one jerrican everyday for each household.
- The time to fill a 20 litre jerrican was between 5 and 6 minutes.
- One breakdown had occurred on one handpump in five months since installation: this lasted two weeks, this occurred because of the moulded cup becoming detached from the pull rod. This was repaired by one of the trainees who attended the workshop mentioned in section 13.
- Rusting was still a problem.
- Children found the pump difficult to use.

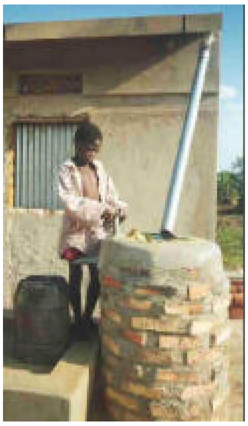


Figure 13 Pump height problem for a young boy

This clearly indicates, as shown in Figure 13 that the handpump is too high, at the bottom of the stroke the handle is above waist height. This would make any reasonable upstroke length difficult for the child.

The time to fill the jerricans was far too slow, and was equivalent of 3.5 to 4 litre per minute. This either suggests that the operator was using too low a cadence, an estimated time rather than an accurately timed one has been recorded or the poor performance indicates some malfunction of the valve or moulded cup. Also, the handpumps are not being used for abstracting anywhere near 100 litre per day. This would mean an extended life for the pump well beyond three years.

For the four Enhanced inertia handpumps that were installed the following main points were noted:

- Almost all the pumps were used to fill on average two jerricans each day.
- The average time to fill a jerrican was 4 minutes.
- No breakdowns had occurred, although on one occasion children had put stones through the outlet, but the pump was soon repaired.
- Children find the handpump difficult to use because of its height.

Again, the height of the pump is not suited to children, as the height set from the jerrican stand to the handle was 0.775m. It may have been better to measure the height of all users hands at their lowest position and found a compromise in favour of the most frequent users height before the handpumps were installed.

14 Final recommendations

There seems little doubt from the test results of all the performance and endurance tests that the Enhanced inertia pump has proved to be the most durable and reliable handpump of the two. This is backed up by the feedback from Uganda on the pumps installed in August last year showing the Harold handpump was much less reliable, and the Enhanced inertia pump was still working satisfactorily. On the strength of these points, the Enhanced inertia pump is recommended as the final choice of handpump to install.

The Harold pump could be recommended in circumstances where the cost to the user is of concern. From section 9.3 it was shown that for a 3.5m length pump, the Harold pump cost $2.4 less than the Enhanced inertia pump. However, if the pump is more prone to reliability problems, then the long-term cost of the Harold pump could be greater.

A set of technical drawings for the Enhanced inertia handpump is included at the back of this report.

15 Means of propagation

The purpose of propagation is to reach and disseminate such information specific to this work to those that may benefit from it. The benefit may be from actual use (an improvement in water quality, or a reduction in time spent walking to some other source) or that the handpumps could generate income and improve the wealth of the individual/family. Some means of propagation have already been mentioned in earlier sections but are reiterated in the following list:

- A one-day training workshop was held for 10 fundies (craftsmen) at Kyera farm, Mbarara, for handpump manufacture.
- A two-day training workshop for handpump manufacture was held for 10 NGO representatives in July 2000 at Kyera farm, Mbarara. Mostly
- The Technical release: '*TR-RWH 09 - Low cost handpumps for water extraction from below ground water tanks - Instructions for manufacture'*, has been on the DTU's web site since September 2000 accessible at: http://www.eng.warwick.ac.uk/DTU/workingpapers/tr/tr09/tr09.html
- The handpumps were signposted in 'Footsteps, No.46 March 2001' Appropriate Technologies, by Tear Fund, a quarterly newsletter for development workers around the world.

16 Further work

The Enhanced inertia handpump has shown that it is capable of pumping water on condition that some annulus size is met. The precise way this handpump operates has proved to be very difficult to analyse and remains to be explained. It was found that the pump's performance is sensitive to the size of the annulus and some optimum size or annulus area ratio needs to be satisfied. As pipe sizes can and do vary over different batches, checking the size is important. Further work is recommended to determine the pumping principle and from this find the optimum size of pipes to give the best flow rates. Some method of controlling the diameter, at the top section of the inner pipe, may prove better than replacing it with a steel rod as explained in section 11.5. This may be achieved by heating the top section and a pushing it through an orifice machined to the required size.

17 Conclusions

This project has proved successful in a number of ways and the majority of the criteria have been fulfilled. It is regarded successful inasmuch as the project was completed on time and one of the handpumps, which was thoroughly tested for endurance, can be recommended for a DRWH system. This has demonstrated that a handpump can be manufactured with very low precision at low cost and be capable of lifting water above 10 litre per minute.

The Enhanced inertia handpump has proved to be a very reliable and durable method of abstracting water for low heads and low flow rates. The performance tests showed that the pump exceeded the specified minimum by 50%, and at a cadence of 70 cycles per minute, 15 litres per minute could be discharged with little exertion by the user.

The specification gave a life of the handpump as 3 years, this may have been underestimated as the endurance test showed it capable of working the equivalent of at least 7 years (and lifting 255,000 litre). A 5 year working life may have been a better specification in retrospect.

One of the main criteria was the cost of the pump, though this was kept just under $10 (including labour cost) for a 3.5m length pump it is doubtful the cost could be reduced further unless material prices came down.

Feedback from handpumps installed in Uganda showed that the hydraulic efficiency is suspected to be low gauging from the time required filling a 20 litre jerrican. From the results of this project, some indication of optimising the efficiency, (higher speed and short stroke versus lower speed and longer stroke), needs to be disseminated with the handpumps. However, an operator's preference in cadence and a stroke length may probably over ride a higher efficiency.

The final two designs were regarded as suitable for production by artisans This was demonstrated by the technicians participating at the training workshop in Mbarara, Uganda who built the Enhanced inertia pumps in two hours! Though whether this would be an income generating activity remains to be seen.

The endurance test was run continuously over a number of days and can therefore be regarded as dissimilar to the actual operation of the handpump. On this basis, the handpump may fail for other reasons such as UV degradation, corrosion of any of the small steel screws in the valve or the wood inlet perishing. Some form of protection would be required to prolong the life of the pump.

The two main failings were firstly, that the low cost valve did leak faster than specified, but this is a minor problem as it takes very few strokes before water is discharged even at higher heads. Secondly, if a steel pull rod is used in the modification of the Enhanced inertia pump, corrosion will affect the quality of the water which would be unacceptable for potable water. However, this may be overcome by galvanising if the cost would permit it.

The enthusiasm of all the technicians and others who have come across the pumps via the web site have shown that there is a need for these pumps. By installing an appropriate DRWH system and incorporating an Enhanced Inertia handpump, a large number of people's lives could be improved.

References

Fraenkel. P, (1997), '***Water pumping device's***, IT Publications.

Gould. J, Nissen-Petterson. E, (1999), '***Rainwater Catchment Systems for Domestic Supply'***, IT Publications.

Morgan. P, (1990), ***'Rural Water Supplies and sanitation'***, Macmillan.

Rees. D, (2000), ***'Plastic Tube Tank (600 litres) - Instructions for manufacture'***, DTU Technical Release Series - TR-RWH08.

Thomas. T. H, McGeever. B, and members of URDT, Kagadi, Uganda, (1997) ***'Underground storage of rainwater for domestic use'***, DTU Working Paper No 49.

Whitehead. V, (2000), ***'The Manufacture of Direct Action Handpumps for use with Domestic Rainwater Harvest Tanks'***, a DTU Technical Release No: TR.-RWH 09

World Bank, (1984), Technical Paper No 19: ***Rural Water Supply Handpumps Project***

Bibliography

Reynolds, J. (1992) ***'Handpumps: Toward a Sustainable Technology'***, UNDP World Bank Water and Sanitation Program.

Kjellerup B. and Ockelford J. (1993), ***'Handpump standardisation in Cambodia'***, Waterlines Vol. 12 No1 (July) pp23-25, IT Publications.

Gould J. and Nissen-Petersen E. 1999, ***'Rainwater catchment systems for domestic supply***', IT Publications.

Skinner, B. (1996) ***'Handpump standardisation'***, 22nd WEDC Conference, New Delhi, India.

Wood, M. (1994) ***'Are handpumps really affordable?'***, 20th WEDC Conference, Colombo, Sri Lanka,.

DFID, (1998) ***'Guidance manual on: Water supply and sanitation programmes'***.

Rees. D, Nyakaana. S, Thomas. T, (2000), ***'Very low cost roofwater harvesting in East Africa'***, DTU Working Paper No. 55

Michael. A. M, Kephar. S. D, (1989), ***'Water Well and Pump Enginerering'*** Tata McGraw Hill, Delhi.

Webliography

- Author: Balaji Industrial and Agricultural castings.

http://www.eepc.gov.in/balaji/frame4.htm

Accessed: 16th Nov. 2000.

- Author: Michael Dudden, Consumers' Association Research & Testing Laboratory

http://www.mailbase.ac.uk/lists/htn/1999-04/0003.html

Accessed: 27th Nov. 2000.

- Author: Rees. D, Case study 2, Underground brick dome tank, Sri Lanka

http://www.eng.warwick.ac.uk/DTU/cs/cs2.html

Accessed: 24th June. 2000.

- Author: UNICEF

http://www.unicef.org/statis/

Accessed: 19th March. 2001.

- Author: New Zealand Hand Pump Company

http://www.nzpump.com/village-pump.html

Accessed November 2000.

Appendix 1 Project Plan

Appendix 2 Taxonomy of pumps and water lifts

Category and name	*Construction*	*Head range (m)*	*Power range (W)*	*Output*	*Efficiency*	*Cost*	*Suction lift?*	*Status for irrigation*
I DIRECT LIFT DEVICES								
Reciprocating/cyclic								
Watering can	1	>3	★	★	★	★	x	✓
Scoops and bailers	1	>1	★	★★	★	★	x	✓
Swing-basket	1	>1	★	★★	★	★	x	✓
Pivoting gutters and 'dhones'	2	1-1.5	★	★★	★★	★★	x	✓
Counterpoise lift or 'Shadoof'	2	1-4	★	★★	★★	★★	x	✓
Rope & bucket and windlass	1	5-50	★	★	★	★	x	✓
Self-emptying bucket or 'mohte'	2	3-8	★★	★★★	★	★★	x	✓
Reciprocating bucket hoist	3	100-500	★★★★	★★★★	★★★	★★★★★	x	x
Rotary/continuous								
Continuous bucket pump	2	5-50	★★.	★★	★★★	★★	x	✓
Persian wheel or 'tablia'	2	3-10	★★	★★★	★★★	★★	x	✓
Improved Persian wheel 'zawaffa'	2	3-15	★★★	★★★★	★★★★	★★★	x	✓
Scoop wheels or 'sakia'	2	>2	★★	★★★★	★★★★	★★★★	x	✓
Water-wheels or 'noria'	2	>5	★	★★	★★	★★	x	✓
II DISPLACEMENT PUMPS								
Reciprocating/cyclic								
Piston/bucket pumps	2 & 3	2-200	★★★	★★★	★★★★★	★★★★	✓	✓
Plunger pumps	3	100-500	★★★	★★	★★★★	★★★★★	✓	?
Diaphragm pumps	3	5-10	★★	★★★	★★★★	★★★	✓	✓
'Petropump'	3	10-100	★★	★★	★★★★★	★★★★	✓	?
Semi-rotary pumps	3	5-10	★	★★	★★	★★	✓	x
Gas or vapour displacement	3	5-50	★★★★	★★★★	★★★	★★★	✓ or x	?
Rotary/continuous								
Gear and lobe pumps	3	10-20	★	★	★★	★★★	✓	x
Flexible vane pumps	3	10-20	★★	★★★	★★★	★★★★	✓	x
Progressive cavity (Mono)	3	10-100	★★★	★★★	★★★★	★★★★	x	?
Archimedean screw	3	>2	★★	★★★★	★★★	★★★	x	✓
Open screw pumps	3	>6	★★★★	★★★★★	★★★★	★★★★★	x	✓
Coil and spiral pumps	2	>6	★★	★★	★★★	★★★	x	✓
Flash wheels and treadmills	2 & 3	>2	★★	★★★★	★★	★★	x	✓
Water ladders 'dragon-spines'	2	>2	★★	★★★	★★★	★★★	x	✓
Chain (or rope) and washer	2 & 3	3-20	★★★	★★★	★★★★	★★★★	x	✓
Peristaltic pump	3	>3	★	★	★★★	★★★	✓	x
Porous rope	3	3-10	★★	★★	?	?	x	?
III VELOCITY PUMPS								
Reciprocating/cyclic								
Inertia and 'joggle' pumps	2 & 3	2-4	★	★★	★★★★	★★	x	✓
Flap valve pump	1 & 2	2-4	★	★	★★	★	x	✓
Resonating joggle pump	2	2-10	★★	★★★★	★★★★	★★★	x	?
Rebound inertia	3	2-60	★★	★	★★★★	★★★	✓	x
Rotary/continuous								
Propeller (axial-flow) pumps	3	5-3	★★★★	★★★★★	★★★★	★★★★	x	✓
Mixed-flow pumps	3	2-10	★★★★	★★★★★	★★★★	★★★★	x	✓
Centrifugal (volute) pumps	3	3-20+	★★★★★	★★★★★	★★★★	★★★	✓	✓
Centrifugal (turbine) pumps	3	3-20+	★★★★★	★★★★★	★★★★	★★★★	✓	✓
Centrifugal (regenerative) pumps	3	10-30	★★★	★★★	★★★	★★★★	✓	x
Jet pumps (water, air or stream)	3	2-20	★★★	★★★	★★	★★★	x	x
IV BUOYANCY PUMPS								
Air lift	3	5-50	★★	★★★	★★	★★★★	x	x
V IMPULSE PUMPS								
Hydraulic ram	3	10-100	★★	★★	★★★	★★★	x	✓
VI GRAVITY DEVICES								
Syphons	1, 2 & 3	1-(–10)	—	★★★★★	—	★★	—	✓
Qanats or foggara	2	—	—	★★	★★★★★	—	—	✓

Construction: 1 Basic 2 Traditional 3 Industrial

Very low ★; Low-medium ★★; Medium ★★★; Medium-high ★★★★; High ★★★★★

Suction lift: Yes ✓ No x

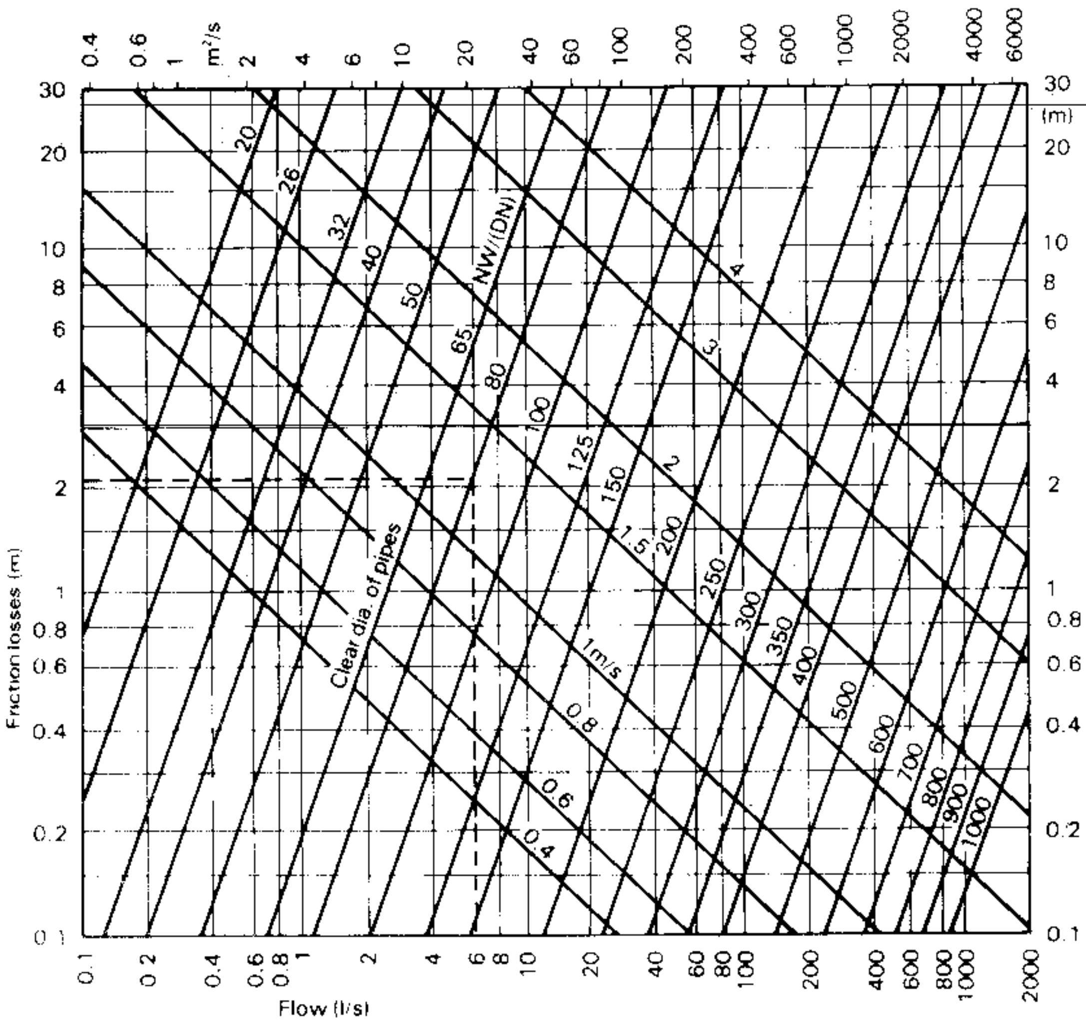
Status for irrigation: Yes ✓ Possible ? Unlikely x

Appendix 3 Materials and tools prices in Mbarara

Price list of materials and tools from Mbarara, Uganda July /August 2000

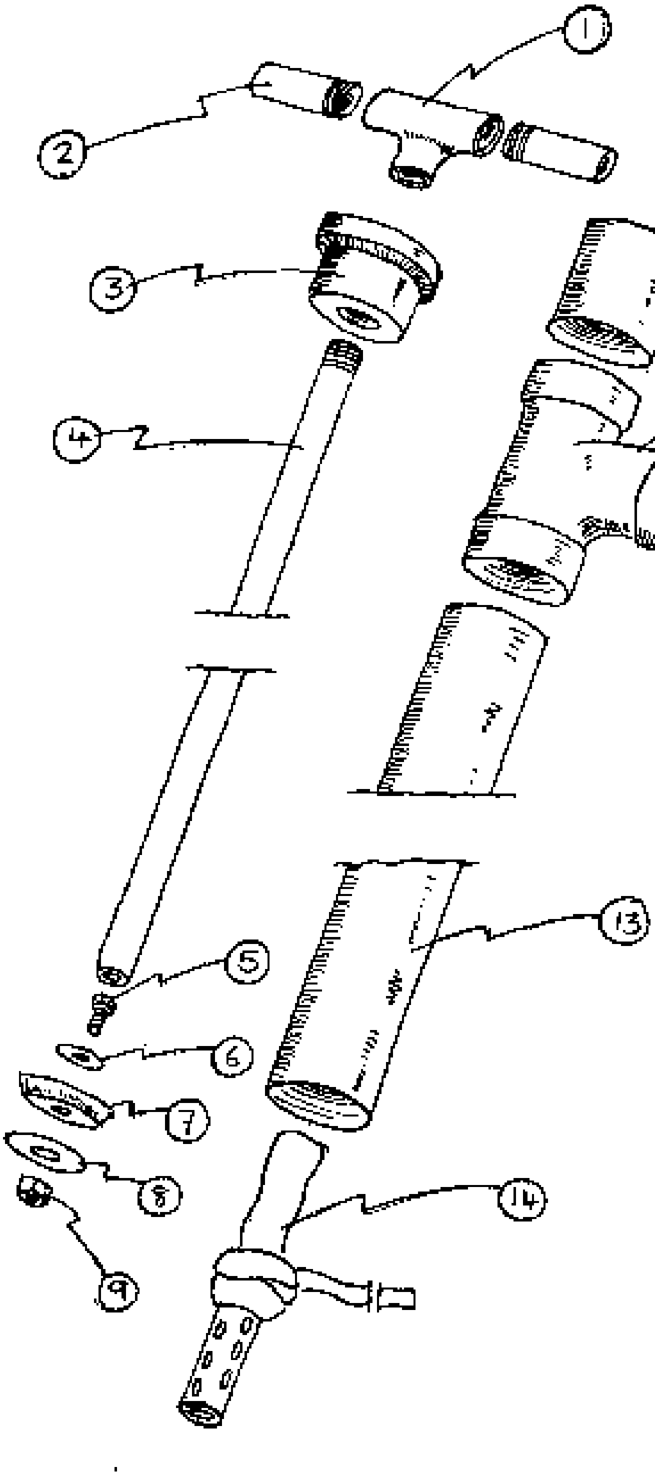
No	Item	length (feet)	value UGS	value £'s
1	Pipe PVC 1 1/4" diameter	20	11,000	4.78
2	Pipe PVC 1/2" diameter	20	7,500	3.26
3	Pipe PVC 3/4" diameter	20	10,000	4.35
4	Pipe PVC1 1/2" diameter	20	12,500	5.43
5	2" x 4 " hardwood	12	3,000	1.30
6	3" diamter GI pipe	4	5,000	2.17
7	3/4" hose pipe	1	500	0.22
8	3/4" wood chisel		13,000	5.65
9	3/8" bolts x 3"		500	0.22
10	3/8" washers		200	0.09
11	7/8" drill bit		2,500	1.09
12	Basin PVC		1,500	0.65
13	Bearing (OD = 40mm, ID = 12mm)		8,000	3.48
14	Bicycle (Indian)		80,000	34.78
15	Binding wire 1kg		2,000	0.87
16	Casual labour wages/day		3,000	1.30
17	Cement (50kg)		10,000	4.35
18	Cement (PVC) 1 tin		5,000	2.17
19	Chains	3	2,000	0.87
20	Charcoal (5ltr tin)		500	0.22
21	Cycle inner tube		2,000	0.87
22	Develop film		2,500	1.09
23	Elbow 3" dia GI		3,000	1.30
24	File 10" rough		2,000	0.87
25	Fired bricks		40	0.02
26	Guttering GI	6	4,500	1.96
27	hacksaw		2,500	1.09
28	Hacksaw blade		1,000	0.43
29	hammer & chain		3,000	1.30
30	Hammer (claw)		3,500	1.52
31	Handrill		14,000	6.09
32	Hinges (pair steel 3")		500	0.22
33	Inlets & bushes (for Harold & NZ handpumps)		267	0.12
34	Jerrycan		2,300	1.00
35	Jubilee clips (4" dia)		3,000	1.30
36	Leather washers (1 1/2" diameter)		500	0.22
37	Masons wages/day		5,000	2.17
38	Mole grips		5,000	2.17
39	Mossi net (PVC)	6	4,000	1.74
40	No 4 x 1 1/4"wood screws		1,000	0.43
41	Nuts & bolts		5,000	2.17
42	Padlock (small)		3,600	1.57
43	Pipe wrench (10")		5,000	2.17
44	Pliers		2,500	1.09
45	Rough file		2,000	0.87
46	Rubber strips	4	300	0.13
47	Screwdriver (medium flat)		3,000	1.30
48	Selotape roll 2" wide		1,300	0.57
49	Spanner (adj 10")		5,000	2.17
50	Tees 1 1/2" PVC		2,500	1.09
51	Tees 1" GI		1,500	0.65
52	Tees 1" PVC		2,000	0.87
53	Tees 1/2" GI		500	0.22
54	Tees 1/2" PVC		500	0.22
55	Tees 3/4" GI		1,000	0.43
56	Tees 3/4" PVC		800	0.35
57	Toolbox (large made from GI sheet)		10,000	4.35
58	Wood screws 1 1/2" long		2,000	0.87
59	Wood screws 3/4" long		1,000	0.43

Appendix 4 Chart for head friction losses in straight pipes



[Fraenkel, 1997, p13]

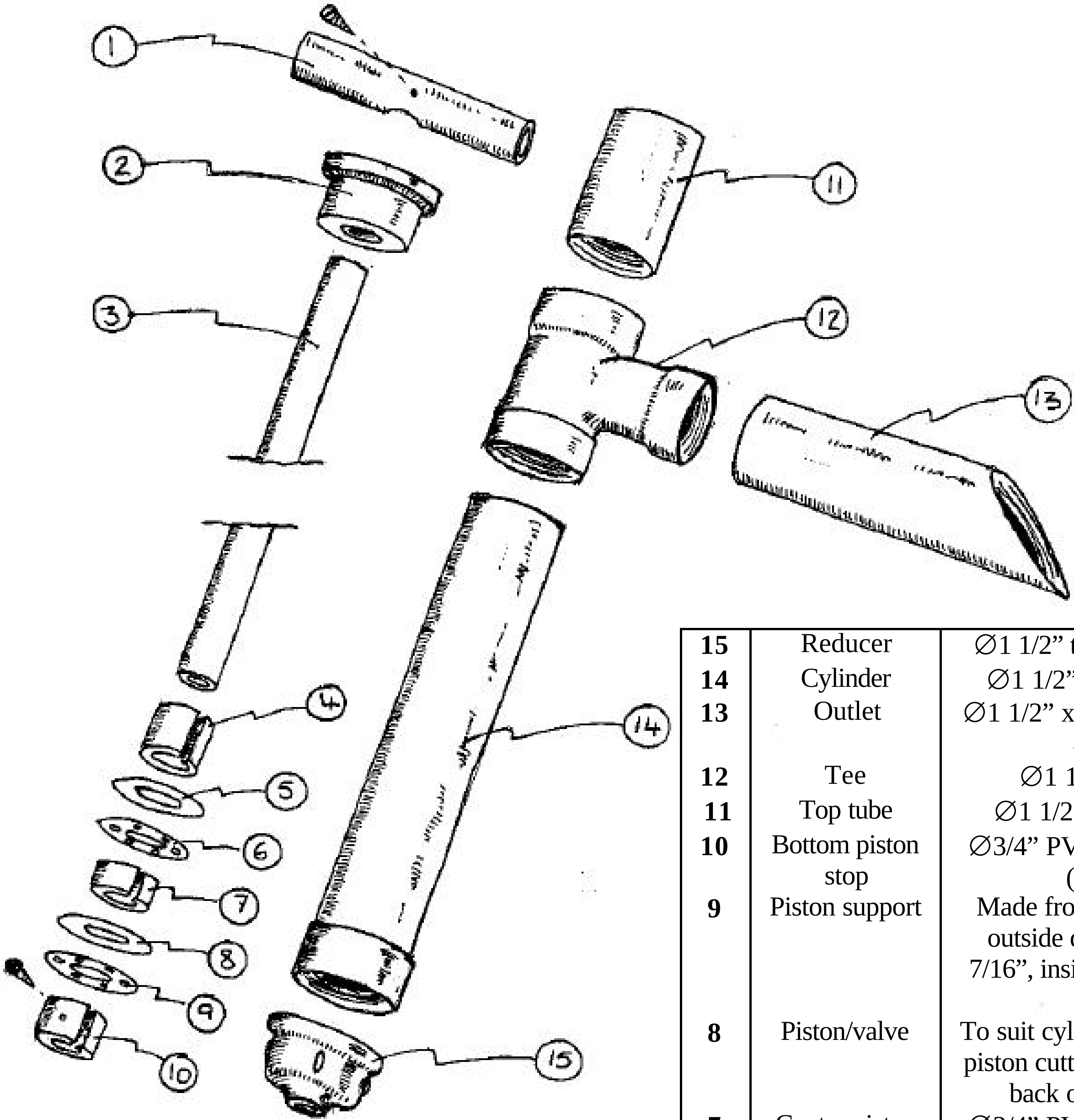
Appendix 5 DTU Handpump assembly drawing



14	Valve	DTU type (see back of manual for details)
13	Rising main	∅1 1/2” PVC (length to suit depth of tank)
12	Outlet	∅1 1/2” x 8” (end at 45°)
11	Tee	∅1 1/2” PVC
10	Top tube	∅1 1/2” x 8”
9	Nut	3/8” BSW or M8
8	Washer	Made from PVC pipe, outside diameter = 1 1/4”, inside diameter = 3/8”
7	Piston	Leather washer from stirrup pump
6	Washer	Made from PVC pipe, outside diameter = 1”, inside diameter = 3/8”
5	Piston screw	3/8” BSW or M8 x 3/4”
4	Pull rod	1/2” PVC pipe (length to suit rising main)
3	Pull rod bush	To suit pipe (see detailed drawing at back of manual for sizes)
2	Handles	1/2” PVC pipe x 4” (2 pieces)
1	Tee	1/2” PVC or GI

(Whitehead, 2000)

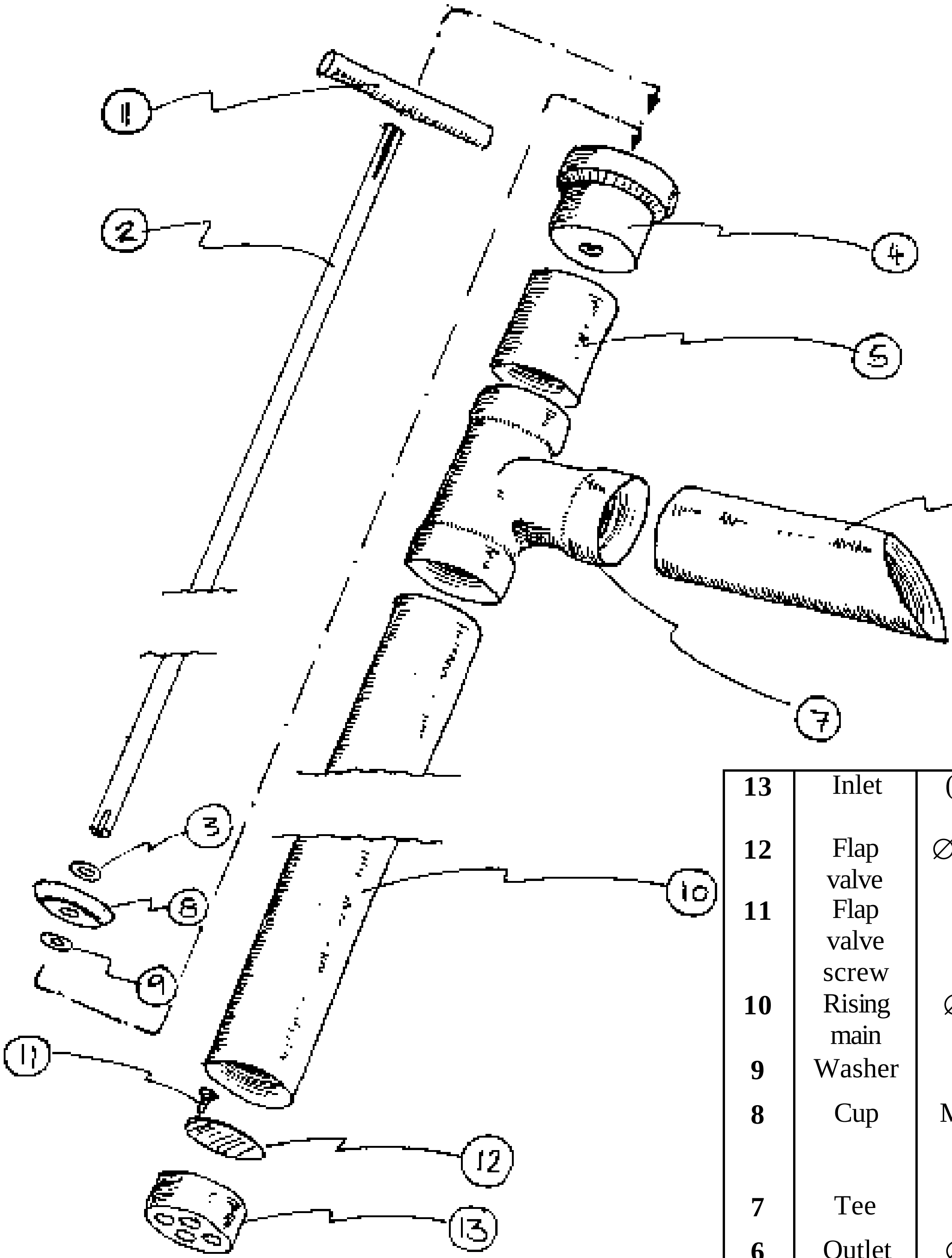
Appendix 6 Tamana handpump assembly drawing



15	Reducer	∅1 1/2” to ∅1/2” G.I.
14	Cylinder	∅1 1/2” PVC x 18”
13	Outlet	∅1 1/2” x 8” (end cut at 45°)
12	Tee	∅1 1/2” PVC
11	Top tube	∅1 1/2” PVC x 6”
10	Bottom piston stop	∅3/4” PVC pipe x 5/8” (split)
9	Piston support	Made from PVC pipe, outside diameter = 1 7/16”, inside diameter = 7/8”
8	Piston/valve	To suit cylinder bore (use piston cutter as shown in back of manual)
7	Centre piston stop	∅3/4” PVC pipe x 5/8” (split)
6	Piston support	Made from PVC pipe, outside diameter = 1 7/16”, inside diameter = 7/8”
5	Piston/valve	To suit cylinder bore (use piston cutter as shown in back of manual)
4	Top piston stop	∅3/4” PVC pipe x 5/8” (split)
3	Pull rod	∅1/2” PVC x 25”
2	Pull rod bush	To suit pipe (see detailed drawing for sizes)
1	Handle	∅3/4” PVC x 8 “

(Whitehead, 2000)

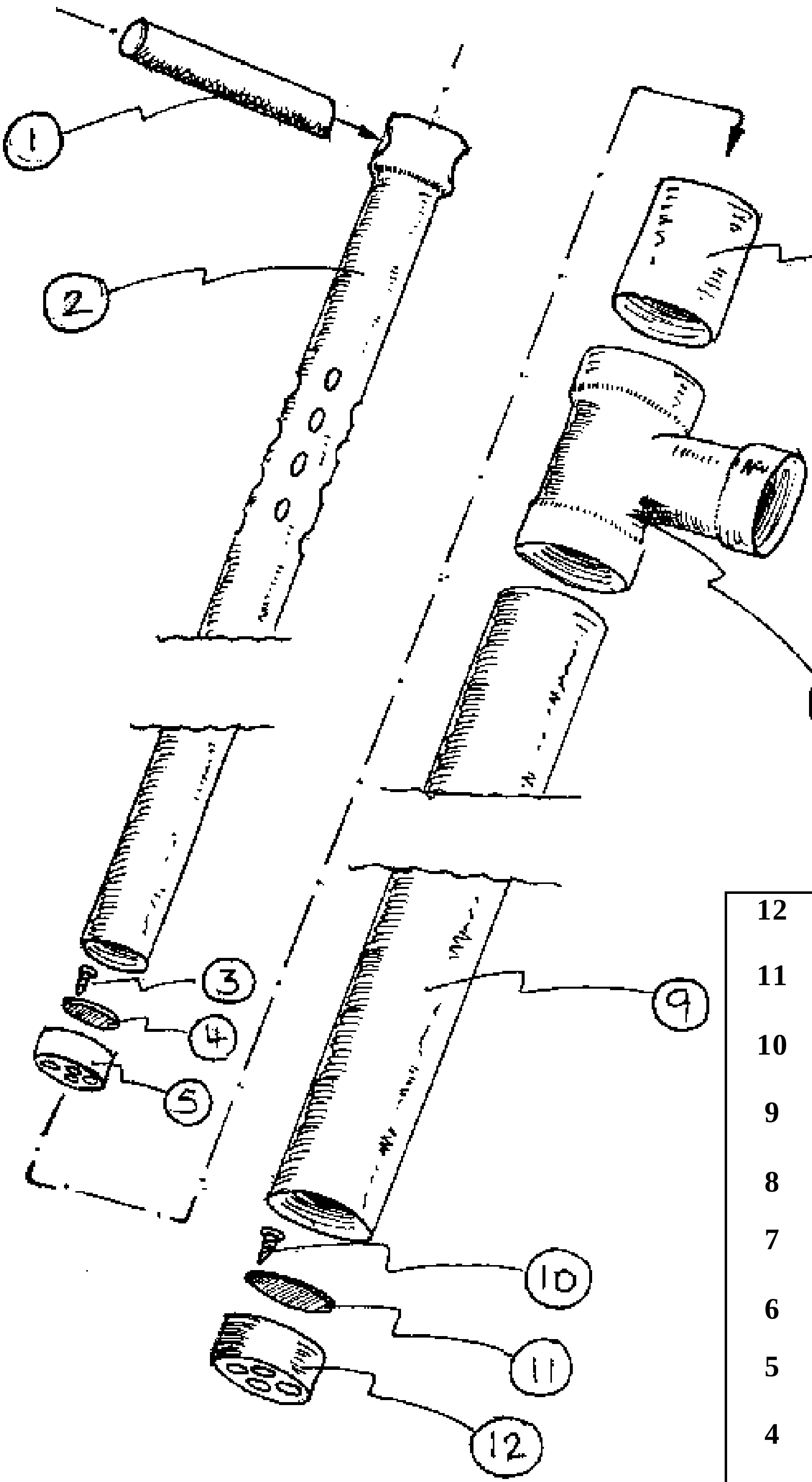
Appendix 7 Harold handpump assembly drawing



13	Inlet	(see detailed drawing in back of manual)
12	Flap valve	∅1 7/16" cycle inner tube
11	Flap valve screw	No 4 x 3/4"
10	Rising main	∅1 1/2" PVC pipe x (to suit depth of tank)
9	Washer	∅3/8"
8	Cup	Moulded PVC (see back of manual for manufacture)
7	Tee	∅1 1/2" PVC
6	Outlet	∅1 1/2" PVC pipe x 8" (end cut at 45°)
5	Top tube	∅1 1/2" PVC pipe x 8"
4	Pull rod bush	To suit pipe (see detailed drawing in back of manual for sizes)
3	Washer	∅3/8"
2	Pull rod	∅3/8" steel x (to suit depth of rising main)
1	Handle	∅1/2" PVC x 8"

(Whitehead, 2000)

Appendix 8 Enhanced Inertia handpump assembly drawing

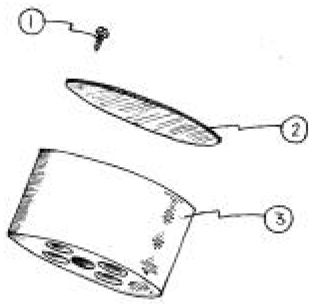


12	Main Inlet	(see detailed drawing in back of manual)
11	Flap valve	∅1 7/16” cycle inner tube
10	Flap valve screw	No 4 x 3/4”
9	Rising main	∅1 1/2” PVC pipe x (to suit depth of tank)
8	Tee	∅1 1/2” PVC
7	Outlet	∅1 1/2” PVC pipe x 8” (end cut at 45°)
6	Top tube	∅1 1/2” PVC pipe x 8”
5	Central inlet	(see detailed drawing in back of manual)
4	Flap valve	∅1 3/16” cycle inner tube
3	Flap valve screw	No 4 x 3/4”
2	Central tube	∅1 1/4” PVC pipe x (to suit rising main)
1	Handle	∅1/2” PVC x 8”

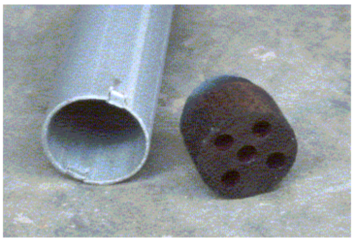
(Whitehead, 2000)

Appendix 9 Valve designs

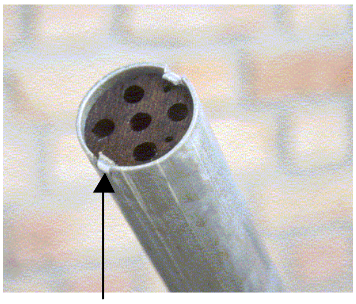
The Low Cost valve



3	Inlet
2	Flap valve
1	Screw

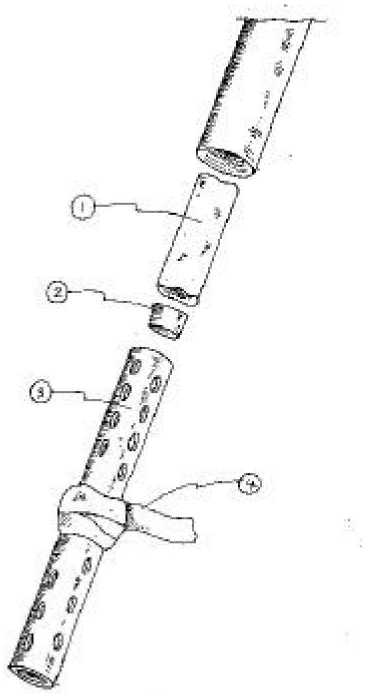


Riser pipe and valve

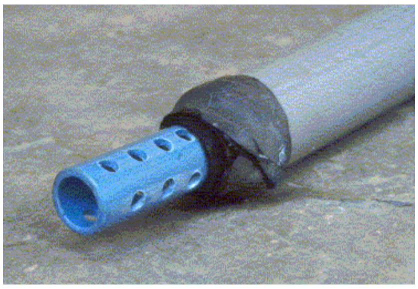


Retaining tabs bent over after fitting the valve

The DTU valve

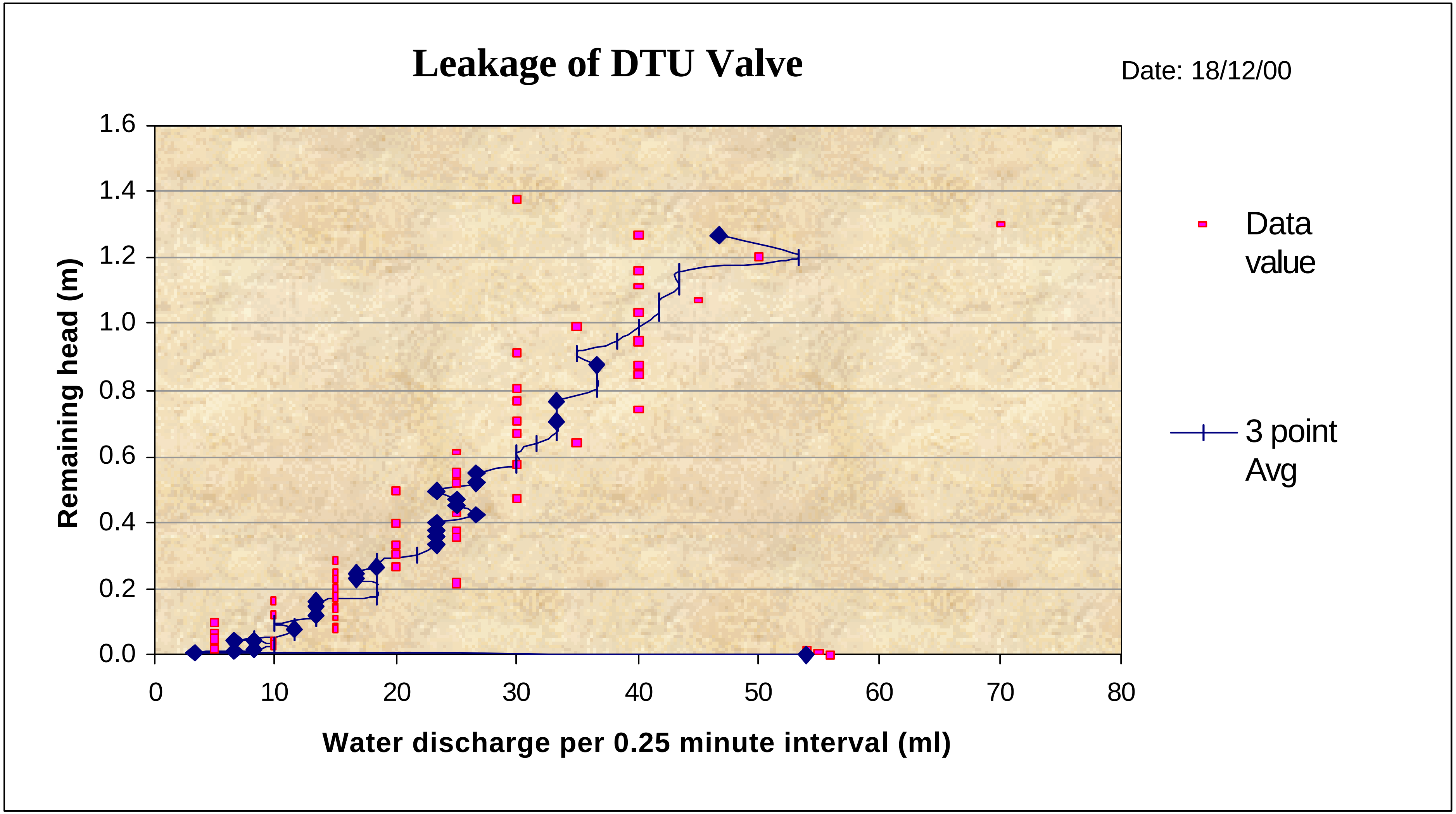


4	Rubber strip
3	Ø3/4” PVC pipe X 8” long
2	Wood plug (to suit)
1	Rubber inner tube X 4”

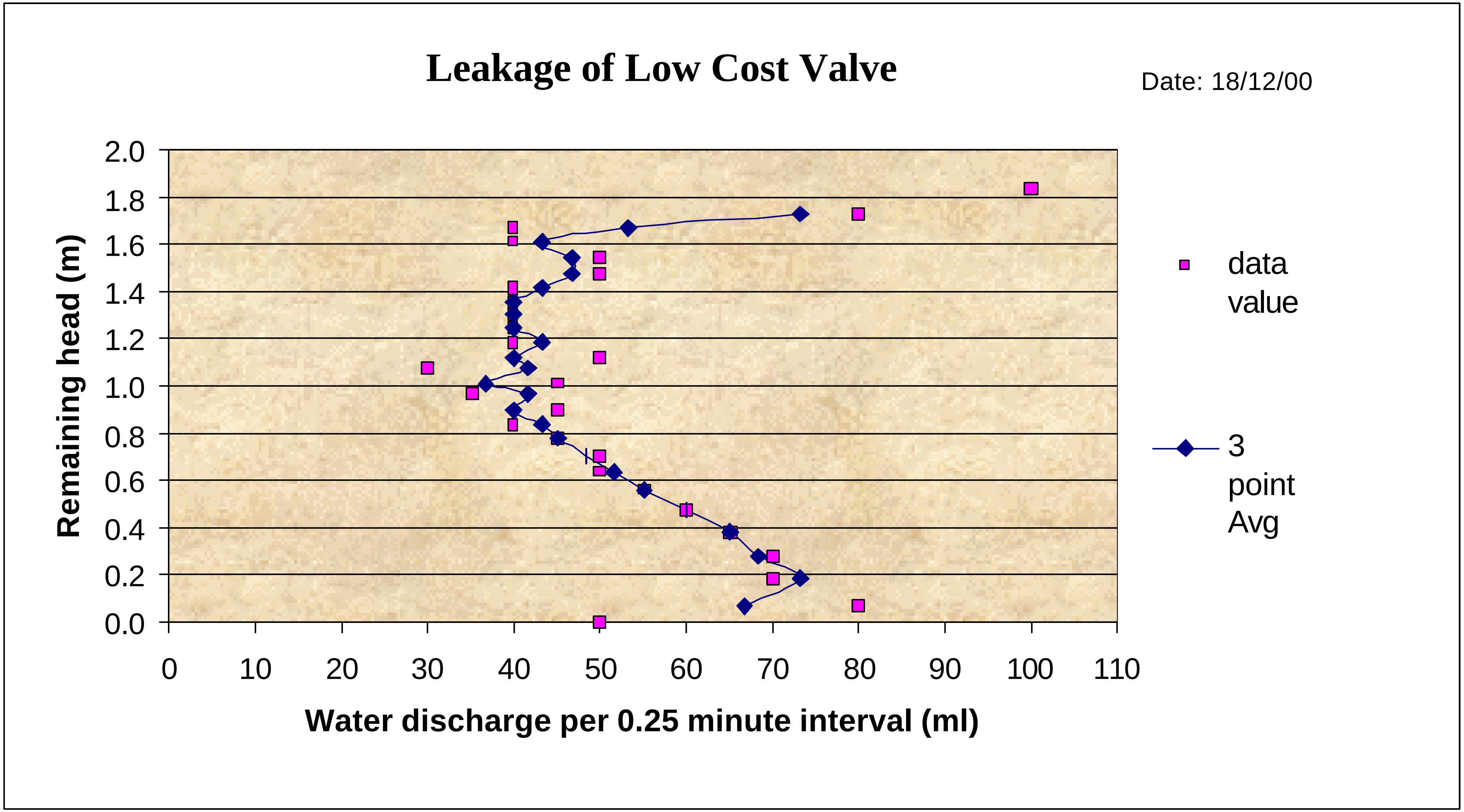


DTU valve fitted and sealed on to riser

Appendix 11 Performance test results



Appendix 11a



Handpump Performance Test No 1

Date	dd/mm/yy	18/12/00
Operator	Name	O. Beresford
Sex	m/f	m
Age	(years)	20
Heart rate at start	(bpm)	107
Handpump	Name	Harold
Molded cup size	(m)	0.035

Cadence (cycles/min)	Bore diameter of pipe (m)	Head (m)	Stroke length (m)	Time to fill 5ltr (sec)	Flow rate (ltr/min)	Volumetric efficiency	Heart rate (bpm)
40	0.036	1	0.25	72	4.2	0.41	116
40	0.036	2	0.25	62	4.8	0.48	105
40	0.036	2.5	0.25	66	4.5	0.45	108
40	0.036	1	0.365	37	8.1	0.55	114
40	0.036	2	0.365	41	7.3	0.49	111
40	0.036	2.5	0.365	45	6.7	0.45	106
Cadence (cycles/min)	**Bore diameter of pipe (m)**	**Head (m)**	**Stroke length (m)**	**Time to fill 5ltr (sec)**	**Flow rate (ltr/min)**	**Volumetric efficiency**	**Heart rate (bpm)**
50	0.036	1	0.25	52	5.8	0.45	114
50	0.036	2	0.25	51	5.9	0.46	118
50	0.036	2.5	0.25	46	6.5	0.51	107
50	0.036	1	0.365	26	11.5	0.62	115
50	0.036	2	0.365	26	11.5	0.62	110
50	0.036	2.5	0.365	28	10.7	0.58	106
Cadence (cycles/min)	**Bore diameter of pipe (m)**	**Head (m)**	**Stroke length (m)**	**Time to fill 5ltr (sec)**	**Flow rate (ltr/min)**	**Volumetric efficiency**	**Heart rate (bpm)**
60	0.036	1	0.25	36	8.3	0.55	113
60	0.036	2	0.25	33	9.1	0.60	111
60	0.036	2.5	0.25	32	9.4	0.61	109
60	0.036	1	0.365	20	15.0	0.67	115
60	0.036	2	0.365	22	13.6	0.61	112
60	0.036	2.5	0.365	22	13.6	0.61	107
Cadence (cycles/min)	**Bore diameter of pipe (m)**	**Head (m)**	**Stroke length (m)**	**Time to fill 5ltr (sec)**	**Flow rate (ltr/min)**	**Volumetric efficiency**	**Heart rate (bpm)**
70	0.036	1	0.25	25	12.0	0.67	113
70	0.036	2	0.25	20	15.0	0.84	115
70	0.036	2.5	0.25	26	11.5	0.65	115
70	0.036	1	0.365	15	20.0	0.77	118
70	0.036	2	0.365	16	18.8	0.72	111
70	0.036	2.5	0.365	15	20.0	0.77	109

Remarks:

highest increase of heart rate = 10 %

Operator comments:

Operator had a preference for 0.3m stroke length, and cadence of 60 cycles/min

Handpump Performance Test No 2

Date	dd/mm/yy	12-01-01
Operator	Name	M. Lyon
Sex	m/f	f
Age	(years)	24
Heart rate at start	(bpm)	89
Handpump	Name	Enhanced Inertia
Molded cup size	(m)	-

Cadence (cycles/min)	Bore diameter of pipe (m)	Head (m)	Stroke length (m)	Time to fill 5ltr (sec)	Flow rate (ltr/min)	Volumetric efficiency	Heart rate (bpm)
50	0.036	1	0.27	33	9.1	0.66	89
50	0.036	1.5	0.27	34	8.8	0.64	115
50	0.036	2	0.27	28	10.7	0.78	101
50	0.036	2.5	0.27	31	9.7	0.70	101

Cadence (cycles/min)	Bore diameter of pipe (m)	Head (m)	Stroke length (m)	Time to fill 5ltr (sec)	Flow rate (ltr/min)	Volumetric efficiency	Heart rate (bpm)
60	0.036	1	0.27	28	10.7	0.65	89
60	0.036	1.5	0.27	29	10.3	0.63	115
60	0.036	2	0.27	25	12.0	0.73	101
60	0.036	2.5	0.27	26	11.5	0.70	96

Cadence (cycles/min)	Bore diameter of pipe (m)	Head (m)	Stroke length (m)	Time to fill 5ltr (sec)	Flow rate (ltr/min)	Volumetric efficiency	Heart rate (bpm)
70	0.036	1	0.27	20	15.0	0.78	89
70	0.036	1.5	0.27	20	15.0	0.78	118
70	0.036	2	0.27	20	15.0	0.78	100
70	0.036	2.5	0.27	22	13.6	0.71	96

Remarks:

highest increase of heart rate = 33 %

Operator comments:

60Cyles was comfortable but 70 was acceptable

Handpump Performance Test No 3

Date	dd/mm/yy	17-01-01
Operator	Name	G. Still
Sex	m/f	m
Age	(years)	21
Heart rate at start	(bpm)	84
Handpump	Name	Harold
Molded cup size	(m)	0.035

Cadence (cycles/min)	Bore diameter of pipe (m)	Head (m)	Stroke length (m)	Time to fill 5ltr (sec)	Flow rate (ltr/min)	Volumetric efficiency	Heart rate (bpm)
50	0.036	1	0.25	26	11.5	0.91	84
50	0.036	2	0.25	30	10.0	0.79	90
50	0.036	1.5	0.365	29	10.3	0.56	86
50	0.036	2.5	0.365	29	10.3	0.56	89

Cadence (cycles/min)	Bore diameter of pipe (m)	Head (m)	Stroke length (m)	Time to fill 5ltr (sec)	Flow rate (ltr/min)	Volumetric efficiency	Heart rate (bpm)
60	0.036	1	0.25	24	12.5	0.82	85
60	0.036	2	0.25	25	12.0	0.79	88
60	0.036	1.5	0.365	25	12.0	0.54	88
60	0.036	2.5	0.365	27	11.1	0.50	89

Cadence (cycles/min)	Bore diameter of pipe (m)	Head (m)	Stroke length (m)	Time to fill 5ltr (sec)	Flow rate (ltr/min)	Volumetric efficiency	Heart rate (bpm)
70	0.036	1	0.25	19	15.8	0.89	90
70	0.036	2	0.25	22	13.6	0.77	86
70	0.036	1.5	0.365	22	13.6	0.52	85
70	0.036	2.5	0.365	25	12.0	0.46	96

Remarks:

highest increase of heart rate = 14 %

Operator comments:

60 cycles/min felt comfortable
Height of pump was okay

Handpump Performance Test No4

Date	dd/mm/yy	15-01-01
Operator	Name	D. Rees
Sex	m/f	m
Age	(years)	38
Heart rate at start	(bpm)	78
Handpump	Name	Enhanced Inertia
Molded cup size	(m)	-

Cadence (cycles/min)	Bore diameter of pipe (m)	Head (m)	Stroke length (m)	Time to fill 5ltr (sec)	Flow rate (ltr/min)	Volumetric efficiency	Heart rate (bpm)
50	0.036	1	0.2	37	8.1	0.80	76
50	0.036	1.5	0.2	45	6.7	0.65	85
50	0.036	2	0.2	46	6.5	0.64	87
50	0.036	2.5	0.2	43	7.0	0.69	80

Cadence (cycles/min)	Bore diameter of pipe (m)	Head (m)	Stroke length (m)	Time to fill 5ltr (sec)	Flow rate (ltr/min)	Volumetric efficiency	Heart rate (bpm)
60	0.036	1	0.2	33	9.1	0.74	80
60	0.036	1.5	0.2	37	8.1	0.66	78
60	0.036	2	0.2	35	8.6	0.70	79
60	0.036	2.5	0.2	39	7.7	0.63	85

Cadence (cycles/min)	Bore diameter of pipe (m)	Head (m)	Stroke length (m)	Time to fill 5ltr (sec)	Flow rate (ltr/min)	Volumetric efficiency	Heart rate (bpm)
70	0.036	1	0.2	24	12.5	0.88	78
70	0.036	1.5	0.2	29	10.3	0.73	83
70	0.036	2	0.2	32	9.4	0.66	81
70	0.036	2.5	0.2	31	9.7	0.68	81

Remarks:

highest increase of heart rate = 12 %

Operator comments:

Cadence of 60Cyles was comfortable and 70 was still acceptable

Appendix 12 Handpump questionnaire

Handpump Questionnaire

Handpump type (Harold or Enhanced inertia)

Enter today's date:

Date or month of handpump installation:

1	Who uses the handpump mostly (tick any which apply)	Girl	Woman	Boy	Man
2	How old is the boy girl that uses the pump?				
3	Does the child find it hard to use the pump? (right height etc)				
4	How many days is the handpump used each week?				
5	How many jerricans are filled on average each day?				
6	Is the time to fill a jerrican too slow or acceptable? (if possible give the time it takes and who filled it: boy or girl etc)				
7	How hard is it for a child to use and fill a 20 litre jerrican (easy, moderate or difficult)				
8	Has it broken down since it was installed. (if the answer is no go to question 9)				
8a	If so what was the reason for the breakdown.				
8b	How long was it before it was repaired (days)				
8c	Who repaired it? (Were they trained at or by Kyera farm)				
9	What are your feelings about the handpump? i.e. What do you think is good about the handpump. Is there any improvement that could be made to the handpump? Any other comments				

Photo gallery of finished handpumps at Kyera, Mbarara, Uganda August 2000

Figure 1 A 20 litre Jerrican under the Enhanced inertia handpump installed in a plastic tube tank

Figure 2 A Harold pump cemented into a plastic tube tank at Kyera Farm

Figure 3 A Ugandan operating the Tamana handpump

Figure 4 The DTU (left) and the Tamana handpumps fitted to a partially below ground tank

Figure 3 Participants after completing the two-day 'Handpump Manufacturing Workshop' at Mbarara, Uganda August 2000.

EDAT 3rd Year Project Report

The Effect of Fines in Sand for the Fabrication and Application of Concrete in Developing Countries

Victoria Fernandes
9902930

Department of Engineering
University of Warwick

23rd April 2002

Summary

This project focuses mainly on the needs of developing countries, where operations are basic and money is short. Good quality sand is available in developing countries but is in short supply and is expensive due to the high transportation costs. Poor quality sand, which consists of a significant proportion of clays and silts and is poorly graded, is readily available but needs to be improved if it is going to be used for building with.

After an insight into the aggregate industry had been gained, experimental laboratory work commenced and the first original objective of examining the effect of 'low quality' sand on mortar was investigated. During the third week of the project, a significant finding was made. It was discovered that when the seven day dry compressive strength tests of comparable concretes with identical water-cement ratios were measured, the strength of the concrete *unexpectedly increased* as the fraction of fines in the concrete increased. This was a fairly surprising result since the original aim of the project was to wash sand to remove the fines to make it a better quality. This result caused me to revise my original aims and the focus point of the project was changed to examining the properties of concrete made from sand with a high fines content.

The properties examined were strength, shrinkage and cyclic movements. Construction materials and processes were optimised and strength was measured using a destructive compressive strength test and the PUNDIT test. Drying shrinkage and cyclic movements were measured using point dial gauges.

It was discovered that despite the suitable dry compressive strength of the material with a large number of fines, that the use of this material will be hindered due to its large cyclic movements and drying shrinkage. This type of sand is only suitable for concrete and mortars in unconstrained applications such as block making and lintels and in situations where the material does not undergo cyclic movements, such as non water-proof rendering. If this material is used for applications such as bricklaying then the wall should be built in small sections, maybe one metre per day, without expansion joints.

There are many other properties of concrete, other than strength, drying shrinkage and cyclic movements that can be specified, but the duration of this project limited the amount of factors that could be examined. It is expected that other research in this field will be carried out to improve the quality of concrete made with 'high fines' sand, possibly by washing it, as the original project suggested.

Contents

List of Tables & Illustrations

Acknowledgement

I am particularly indebted to Dr. T. H. Thomas, my project supervisor, for his help and encouragement throughout the duration of this project. I am also indebted to Mr. R. Bacon, the Production Manger at Gill Mill Quarry in Witney, Oxfordshire, for his time during my site visit to Gill Mill Quarry. It is also a pleasure to thank Mr. C. Banks, the Civil Engineering technician for all his patience and time spent helping me with my experiments; as well as my boyfriend, Mr. T Jenkin, for his advice and criticism on the draft report.

Notation

V	Compression wave velocity
E	Dynamic modulus of elasticity
P	Density
v	Dynamic Poisson's ratio
$\mathbf{f_c}$	Equivalent cube strength
A	Constant
B	Constant
e	Base of natural logarithms
x	Arithmetic mean
σ	Standard deviation
$\mathbf{v_c}$	Coefficient of variation
n	Sample number
Z	Test Statistic
N	Normal distribution
$\mathbf{H_0}$	Null hypothesis
l	Length

1. Introduction

1.1 Background

This project focuses mainly on the needs of developing countries, where operations are basic and money is short.

"Sand is primarily used as an aggregate or bulking material, in the main for concrete and mortar, accounting for approximately 75% of total production", (Mineral Resources Consultative Committee, 1988).

Good quality sand is available in developing countries, but is in short supply and is expensive due to the high transportation costs. Poor quality sand, which consists of a significant proportion of clays and silts and is poorly graded, is readily available but needs to be improved if it is going to be used for building with. This project was originally established to identify an effective method of improving the quality of readily available poor quality sand to make it more fit for use in making concrete or mortar. This was to be done by exploring existing methods of washing poor quality sand, such as sieving and sedimentation and modern methods used in the commercial industry in the UK.

Within this overall aim, the original study had three specific objectives:

- To examine the effect of "low quality" sand on mortar to define a just satisfactory standard.
- To compare sieving and sedimentation techniques for manually washing poor quality sand up to that standard under artisanal conditions in a developing country.
- To develop an effective method of, or a machine for improving sand, possibly by washing it, to make it more fit for use in making concrete or mortar. The method should be compatible with the operations, finances and normal sand sources of an artisanal builder in a developing country, such as Africa.

The project was started with little knowledge about the aggregate and concrete industry. It was assumed alongside the general opinion of the aggregate industry in the UK and in accordance with the British Standard BS 812, that a high 'fines' (silts and clays) content in sand for fabricating concrete is unacceptable, and to improve the quality of concrete the majority of the fines should be removed. Gill Mill Quarry in Witney, Oxfordshire was visited to get a general feel of the sand and aggregate industry, and to examine the existing methods of sand washing to remove the fines, (The letters of correspondence for this visit are shown in Appendix A1). Removing these smaller particles from the sand is a very costly process in terms of energy and money, and the existing sand and aggregate plants in the UK do not deliberately remove the fines from the sand, but instead grade the different sizes of sand and gravel particles. When separating the larger sizes of particles, wet screening is used, which permits the sand, aggregate and water to fall through a various number of sieves of different sizes that are at 20° to the horizontal.

Figure 1.1: Wet screening at Gill Mill Quarry, Oxfordshire

Figure 1.2: A Hydro-cyclone

The remaining sand and water enters a hydro-cyclone, which imparts a rotational effect to the mix, with the resulting centrifugal force causing the heavier particles to be pushed against the inside surface of the hydro-cyclone. As a result of the continuing feed of material and the inverted cone shape, the heavier sand particles fall and the water and fines pass upwards into the overflow and are discharged away through an overflow pipe into a settling pond. The overflow pipe induces a syphon effect, which in turn creates a vacuum in the hydro- cyclone. This vacuum is sufficient to hold the discharge regulator closed thus trapping the majority of the water and fines, which are discharges via the overflow. When the weight of the solids inside the hydro-cyclone becomes sufficient to overcome the vacuum, the discharge regulator is forced open and allows discharge of the sand.

It was decided that this method of separating the different sizes of particles is too sophisticated and energy costly for developing countries to use, so either an alternative method for improving the quality of sand for concreting needs to be established, or it needs to be demonstrated that washing the sand is not essential.

1.2 The Project Report

The project report has been organised into the following chapters (after this introductory chapter 1):

- Chapter Two explains the aims and objectives of the research, and how the objectives were changed from the original ones
- Chapter Three includes the literature review, describes the purpose of the project and discusses the expected problems associated with using a high fines content in sand when making cementitious materials.

- Chapter Four details what materials were to be tested, what factors would be examined and methods of experimental testing were chosen.
- Chapter Five describes the methodology of each testing method and how the analysis of each result should be carried out.
- Chapter Six is the results section. Each factor has been divided into a sub section and discussed individually. Statistical analysis has been performed on applicable results, graphs have been drawn, and explanations of results have been given.
- Chapter Seven states what these results mean in terms of its use to builders in developing countries. It explains whether building with a material with a high fines content is advisable, and whether it would be suitable for some purposes, rather than others.
- Chapter Eight recommends future work, followed by conclusions in Chapter Nine.

2. Revision of Project

After an insight into the aggregate industry had been gained, experimental laboratory work commenced and the first original objective of examining the effect of 'low quality' sand on mortar was investigated. During the third week of the project a significant finding was made as it was discovered that when the seven day dry compressive strength tests of comparable concretes with identical water-cement ratios were done, the strength of the concrete unexpectedly increased as the number of fines in the concrete increased. This was a fairly surprising result since the original aim of the project was to wash sand to remove the fines to make it a better quality. This result suggested that washing sand prior to use, to remove the fines is unnecessary and indicated that further experiments needed to be done to determine the effect of a high fines content in concrete. This caused me to revise my original aims and objectives of the study.

The new main objective of this study was to test whether sand with a high fines content can be used in the fabrication and application of concrete. The purpose of this was to make a useful contribution to the living standards in developing countries through the development of appropriate technologies. In this case the 'appropriate technology' is the use of local sand 'as found' rather than the use of transported 'no fines' sand.

Within this new overall aim, the study had two specific objectives:

- To use the strength of concrete as a constraint to optimise water content, workability, vibration compaction time, the number of 'fines' and curing time, with respect to costs and skills related to developing countries.
- To determine the effect of the presence of 'fines' on shrinkage and swelling of concrete.

For the remaining duration of this project, laboratory based experiments have been conducted to satisfy the new objectives, and it is hoped that the research will be of use to artisanal builders in developing countries.

3. Fines in sand in cementitious materials

3.1 The Cement Literature

Books have provided little information on the actual effect of fines in concrete but have proved useful in the Properties of Concrete. As an overview on general aspects of concrete, A.M Neville and J.J Brooks provide a highly accessible and relevant text. J.M Illston also provides a very useful introductory section on the basics of concrete.

For concreting aspects related to developing countries, Spence and Cook provide an excellent section on the problems of skill, equipment and climate. This enabled a clear distinction between concrete technology in the UK and developing countries.

During the examination of the strength of concrete, Popovics book was very useful. It provided detailed descriptions with clear explanations, especially in the testing of concrete. Another book which proved useful on this area was "Testing of Concrete Structures" by J.H Bungey and S.G. Millard.

Statistical analysis was explored using various textbooks such as "Statistics and Experimental Design in Engineering and the Physical Sciences" by N. Johnson, but the clearest explanations were in "Statistics 2", written by G. Attwood and G. Dyer.

Due to the complexity of the analysis of shrinkage and swelling due to a high fines content, full descriptive analysis was not incorporated into this report. However, the basics of shrinkage and swelling are described in the general concrete books mentioned above, and additionally a detailed account of the behaviour of clay is given by J.K. Mitchell, "The Fundamentals of Soil Behaviour".

Other literature that was often consulted was the Building Research Establishment Report, 1998 and the British Standards. The British Standards were accessed through the World Wide Web at www.bsonline.co.uk. The main relevant standards that were consulted were:

- *BS 12: 1978*: Ordinary and Rapid Hardening Portland Cement.
- *BS 812: Part 1:1975*: Sampling, shape, size and classification.
- *BS 1881: Part 3: 1970*: Methods of Making and Curing Test Specimens.
- *BS 1881: Part 115*: 1983: Specification for Compression Testing Machines of Concrete.
- *BS 1881*: *Part 116: 1983*: Method for the Determination of Compressive Strength of Concrete Cubes.
- *BS 1881*: *Part 202: 1986*: Measurement of the Velocity of Ultrasonic Pulses in Concrete.
- *BS 5497: Part1: 1979*: Guide for the Determination and Repeatability and Reproducibility for a Standard Test Method.

3.2 Review of Issues

In the UK, it is stated in the British Standard BS 812, that it is considered unacceptable to make cementitious materials with a high fines content. It is thought that the reason for this is that fines interfere with the bond between the aggregate and the cement paste, causing abnormal and variable behaviour of the properties of concrete. This is suggested in Neville and Brooks (1987), but the exact reasons of why fines in sand should not be used could not be found in literature.

In the UK, the main properties of concrete that are considered and tested are wet and dry compressive strength, shrinkage, cyclic movements, permeability, durability, and resistance to freezing and thawing, (Neville & Brooks, 1987). The extra fines in the sand are likely to have some effect on all of these properties, and it was the aim of this project to examine certain properties, to discover how the changes in behaviour would affect the use of concrete in certain applications. It was expected that the strength, shrinkage and cyclic movements would be affected greatly by a high fines content in the sand. However since this project focuses on concrete for use in the (high-temperature) tropics, properties such as resistance to freezing and thawing did not need to be examined.

The application of cementitous materials in developing countries is mainly for bricklaying, rendering, and as reinforced concrete. There has been a trend in recent years to manufacture housing components such as lintels, roofing, and hollow core flooring units in an effort to reduce costs. In India, pre-cast concrete frames for doors and windows have been investigated because of the expense of conventional wooden frames, (Mohan and Rao, 1977).

The strength of the concrete is important for most of the applications stated above, but is not so critical for bricklaying and rendering. Concrete compressive strength is typically between 10-70MPa for most applications and is not generally used at strengths lower than 10MPa.

Concrete is strong in compression but weak in tension. Concrete in compression can withstand 2000 μ-strain and in tension approximately 200 μ-strain (Tensile stress is approximately 1/10 the compressive stress). Hence, cracks could occur due to tensile forces or strains exceeding the tensile capacity of the concrete. Tensile forces and strains can arise from many causes, including cyclic movements and shrinkage, and they give rise to many different forms of cracks.

For bricklaying, the concrete should not have large changes in volume, since if this happens unevenly around a building, cracking may appear in the brickwork. Consequently, if the brickwork if jammed between two rigid elements, it could cause the wall to bow. If the brickwork is in compression, it is expected that the wall can withstand about 600 μ-strains, so this magnitude of shrinkage can be tolerated. If the brickwork is in tension, then a much lower value can be tolerated, typically 200 μ-strains.

Typically renders use gypsum since it doesn't crack, but as a cheap alternative, cement renders can be used. Cracks can be dangerous if they occur in a wall between two points, but are also a nuisance and should not be visual. It is thought that waterproof renders can only tolerate small cyclic movements, generally <100 μ-strain, and non-waterproof renders should not shrink or expand more than 200 μ-strain. If renders are used for decorative value then a millimetre crack per metre is visible, and would be unacceptable.

Steel reinforcement in concrete is common in the UK, but structural steel is rarely used in developing countries for building because of its cost but steel is obviously essential for the reinforcement of concrete. As few countries have sources of iron ore suitable for steel making, they must resort to importing pig iron and processing it to make steel. When steel reinforcement is used, movement of a section in the concrete member should be prevented because a gradual transfer of load from the concrete to the reinforcement results. If there is an external constraint and the concrete expands, it is being prevented from moving and compressive stresses develop uniformly across the section. If large shrinkage occurs, typically over 600 μ-strains, the concrete is being prevented from contracting so that tensile stress is induced. In consequence, cracking will take place across the section.

Changes in concrete properties for the fabrication of lintels should not cause any problems, since it is common practice to leave a gap in the mortar of about 10mm at both ends to allow it to expand without cracking. If the cyclic movements are larger than 1000 μ-strain then the gap can be expanded.

This project examined the effect of fines in sand when making concrete to decide whether a high fines concrete can be used for the applications mentioned above, for the building industry in developing countries.

4. Choosing the Sand and Method for Testing

4.1 Sand chosen for testing

"Natural sand and aggregate is formed by the decomposition of rocks, and through the subsequent removal, transportation, sorting, deposition and weathering of the products of decomposition", (Spence & Cook, 1983).

In some areas, some or all of the products of weathering remain in place, in conjunction with the parent rock. These are typical of broad, flat, poorly drained upland areas such as Africa and Indian plateaux. In other areas, the products of weathering are transported, that is they are removed by water or ice, or sometimes wind. In the process of the transportation by water larger particles are further broken down, shaped and resorted, and then deposited in river terraces or lakes. Dissolved silicates react with the soluble salts of sodium, potassium and calcium to form new minerals with distinctive properties known as clay minerals.

Clay, those particles of less than 0.002mm in size, consists predominantly of the clay minerals. Clay may be present in sand in the form of surface coatings which interfere with the bond between the sand and the cement paste. In addition silt, which has a grain size between 0.002 and 0.06mm, may be present either as surface coatings or as loose material.

"It is assumed that clay particles should not be present in large quantities because, owing to their fineness and therefore large surface area, they interfere in the bond between the sand particles and the cement paste matrix", (Spence & Cook, 1983).

It is also thought that excess fines cause an increase in the water demand and as a result the strength of the concrete will decrease, (as a consequence of the water cement ratio rule, which is investigated later).

Sand used for building purposes in the UK follow the recommendations in the British Standard BS 812, that only good quality sand is used with a minimal amount of fines. This is also shown on the U.S Bureau of Public Roads Classification System (for soil) which is based on particle size analysis. The chart is divided into 10% bands, with clay occupying the top third of the chart, sand and silt in the bottom corners, and sand-silt-clay mixtures between them. UK standards specify for use the material in the bottom left hand corner of the chart, usually between 90-95% sand, as shown in blue. In order to test a less well selected aggregate, a synthetic ‘poor sand’ “X” was devised comprising of 75% sand, 4.8% silt and 20.2% clay, as shown in red. This material will be tested against the same sand but without the large clay proportion, which is denoted as material “Y”. Material “Y” is comprised of 95% sand, 4.8% silt and 0.2% clay, and is shown on the diagram in green.

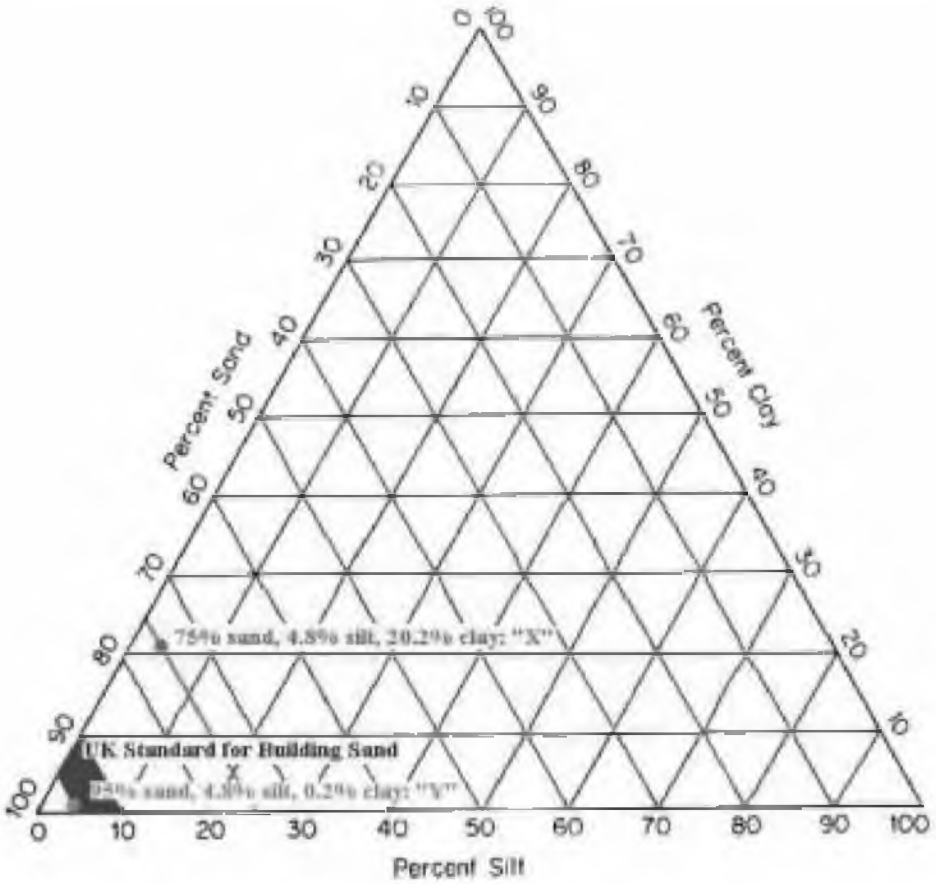


Figure 4.1: U.S Bureau of Public Roads Classification System

Ideally, the effect of a significant amount of silt as well as clay in concrete needs to be examined. Pure clay is readily available but it was not possible to find a source of pure silt, although several attempts were made. Material from a settling pond was collected to examine how much silt was present, but unfortunately the material contained 89% clay, with only the remaining amount being silt, (see Appendix A2 for the sedimentation test and result of the determination of silt content). It was decided that this material could not be used to represent silt, so the fines added to the sand to degrade it will be clay only.

The clay used was Kaolin as it is relatively inexpensive and easily available at the University. Degrading the sand to this quality is adjusting the sand so that it is of a “poor” but not “extremely poor” quality. It was decided that for the purpose of this project that this would be the only type of sand tested, as the project has time constraints with many other factors to be investigated.

Sand and aggregate used in the UK for the purpose of building is usually well graded, as the graduation of particles affects strength. This is because if there are voids present in the concrete due to the inability of the aggregate to fill the voids, then a high strength cannot be achieved. Figure 4.2 is a particle size distribution graph which shows a good sand and aggregate distribution, (drawn in blue), which lies within the limits (shown dashed) recommended by the British Standard BS 812 for medium sand. This good particle size distribution is represented by a shallow curve which slopes from left to right and meets the top of the chart at the size of the coarsest particles present. The builders “Y” sand that was used to make the very poorly graded sand “X” is shown in green. This was already outside the grading limits set by BS 812 as it had 5% more fines than recommended and 12.2% extra 0.6mm particles than the maximum allowed. It also had very few particles exceeding 2.36mm resulting in the sand being deficient in larger particles. Extra clay was added to this sand to give an even poorer distribution, increasing the fines content from 5% to 25% higher than the recommended limit. This material “X” is represented by the curve shown in red. Throughout the duration of this project sand “X” represented by the red curve and sand “Y” represented by the green curve have been used for experimental work.

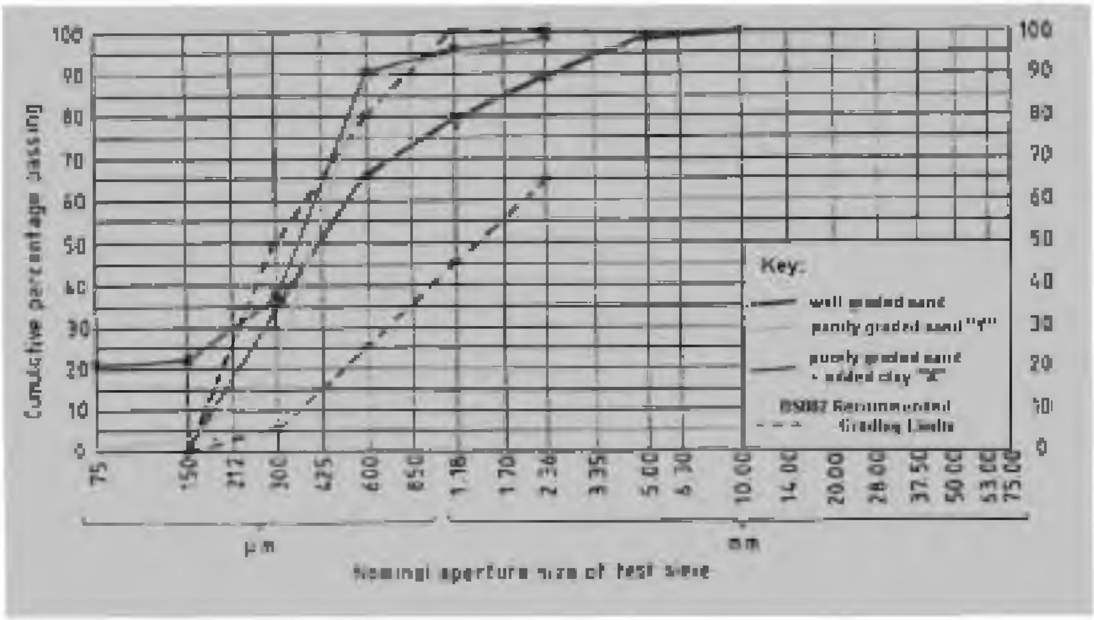


Figure 4.2: Particle Size Distribution Graph

4.2 Factors & method chosen for testing

4.2.1 Strength

Concrete is a structural material, so it is understandable that the most sought-after property of a concrete is strength. The strength of concrete is its resistance to rupture. Structural elements must be capable of carrying the imposed load and the maximum value of bearable stress is usually taken as the strength.

"Concrete strength appears to be a good index of a number of other technically important properties and routine strength tests in general are relatively simple to make", (Kesler, 1966).

Some of the many important properties of concrete that improve with an increase in strength are density, resistance to deterioration, porosity and permeability; hence strength may be used as a criterion of general quality.

It was therefore decided that strength would be an important factor to test, but the type of strength to be tested needed to be decided. In structural situations concrete is subject to one of a variety of types of loading, resulting in different modes of failure.

Knowledge of the relevant strengths is therefore important. There are 7 main different types of strength:

- *Compressive strength*: Required for columns or reinforced concrete beams.
- *Tensile strength*: Important for the cracking of concrete slabs. Tensile strength is about 10% of the compressive strength.
- *Shear strength*: Important in reinforced concrete beams to control the diagonal cracking of the beam. The pure shear strength is about 20% of the compressive strength.
- *Torsion strength*: Important for a reinforced concrete beam in pure torsion before cracking. The torsion strength of concrete is calculated by plastic theory and is usually higher than tensile strength.
- *Impact strength*: Important in concrete piles and in certain military establishments. Impact strength is approximately half of the compressive static strength.
- *Fatigue strength*: Has a practical significance since the majority of structures are subjected to repeated loading.
- *Multi-axial loading*: Important because concrete in structures is practically always subject to multi-axial combination of compression, tension, shearing and torsional stresses.

The two most common types of strength that can be tested for concrete are tensile strength and compressive strength. The tensile strength of concrete has a fundamental role in the fracture mechanism of hardened concrete. It is an accepted view that the fracture in concrete occurs through cracking caused by tensile stresses. This means that concrete fracture is essentially tensile failure regardless of whether the fracture is caused by compression, freezing, or other factors. Therefore, the mechanical properties of a hardened concrete are controlled to a great extent by the fact that its tensile strength is about one tenth of the compressive strength. To avoid undue cracking in structures such as concrete beams and slabs, the tensile strength is of special importance, despite its low magnitude. The case of uni-axial tension is rarely encountered in practice and in laboratory tests can only be obtained with care.

"The compressive strength of concrete is one of the most important technical properties. In most structural applications concrete is employed to withstand compressive stresses", (Popovics, 1998).

In those cases where other stresses (e.g. tensile stress) are of primary importance, the compressive strength is still frequently used as a measure of the resistance because this strength is the most convenient to measure. For the same reason, the compressive strength is generally used as a measure of the overall quality of concrete.

It was therefore decided that compressive strength would be tested, but a decision had to be made on whether the wet or dry compressive strength should be used. In the stabilised soils literature it can be seen that the compressive strength is mainly referred to as the wet compressive strength, but in the concrete literature the dry compressive strength is used.

It was expected that the sand being used would affect the wet compressive strength dramatically showing a cyclic loss of mass in the material as well a decrease in strength. It was decided that the shrinkage and swelling would be important factors to investigate due to the high fines content in the sand, so would be examined independently, and not combined with the strength test to form one single test. The wet compressive strength indicates what happens to the strength as well as taking into account the shrinkage and swelling, but the compressive strength indicates the strength only and was chosen to be tested.

According to Witmann, (1968) the dry compressive strength of concrete is higher than that of a comparable wet concrete because dryness decreases the volume of the hardened cement paste. The volume reduction is caused by surface tension that increases in the water filled small pores during drying, pulling adjacent grains of solid together: thereby reducing the average distance between surfaces in the hardened cement gel. This increases the secondary bonds between the surfaces, increasing the strength. Since rewetting the hardened cement paste causes volume increases, and therefore an increase in the average distance between surfaces of the cement gel, this would explain the lower strengths of wet cement paste and concrete specimens.

Compressive strength can be tested at various specific ages, where various ages only give a proportion of the final strength.

"The strength developed by a concrete made with given materials and given proportions increase for many months under favourable conditions, but in the majority of specifications the strength is specified at an age of 28 days", (Building Research Establishment Report, 1988).

Traditionally, concrete strength is determined 28 days after casting and rejection of a particular batch of concrete, or adjustment of the concrete production is delayed by the substantial time difference between casting and testing. It was therefore decided to test the specimens at an age of 7 days to begin with in order to get a respectable number of results to be analysed. After the main trends that the factors were following for 7 day tests, specimens were made to be tested after 28 and 56 days.

Numerous test methods have been recommended for the determination of concrete strengths. Some of these methods measure fundamental properties of the concrete whereas others do not. Also, various simplifying assumptions are used in the procedures of different test methods to convert a measured load to a calculated failure stress. Some of these procedures require more doubtful assumptions than others, which may considerably influence the relationships between various concrete strengths.

The principle laboratory test methods that could have been used were:

- *Destructive Compressive Strength Test*
- *PUNDIT Test*
- *Schmidt Hammer*
- *Penetration Resistance*
- *Pull-out Test*

A summary of the principle test methods are shown in the table below:

Method	Standard Number	Principle Applications	Principle Properties Assessed	Surface Damage	Type of Equipment	Remarks
Destructive Compression Test	BS 1881	Strength Measurement	Compressive Strength	Completely	Mechanical	Load rate should be of a few minutes
PUNDIT Test	BS 1881-203a	Comparative Surveys	Elastic Modulus	None	Electronic	Two opposite smooth faces preferably needed, strength calibration affected by moisture and mix
Schmidt Hammer	BS 1881-202	Comparative Surveys	Surface Hardness	Very Minor	Mechanical	Greatly affected by surface texture
Penetration Resistance	BS 1881-207a	In Situ Strength Measurement	Strength Related	Moderate / Minor	Mechanical	Specific calibrations required, surface zone test
Pull Out Test	BS 1881-207a	Quality Control (In Situ Strength)	Strength Related	Moderate / Minor	Mechanical	Pre-planned usage

Table 4.3: Summary of Principle Test Methods

It was decided that the compressive strength would be measured using the standard destructive *compressive strength test*, as recommended in the British Standard BS 1881, and the non-destructive *PUNDIT test*. Because the PUNDIT test is quick and non-destructive it allowed a large number of readings to be taken without damage to the blocks so as to retain some blocks to obtain further readings as the specimens aged. The detailed procedure of these two tests and problems associated with them are described in the research methodology.

The Schmidt hammer, also known as the rebound or impact hammer was also considered as a non-destructive test method that could have been used. The test is based on the principle that the rebound of an elastic mass depends on the hardness of the surface against which the mass impinges. The spring loaded mass has a fixed amount of energy imparted to it extending the spring to a fixed position; this is achieved by pressing the plunger against a smooth surface of concrete which has to be firmly supported. This is illustrated in figure 4.4.

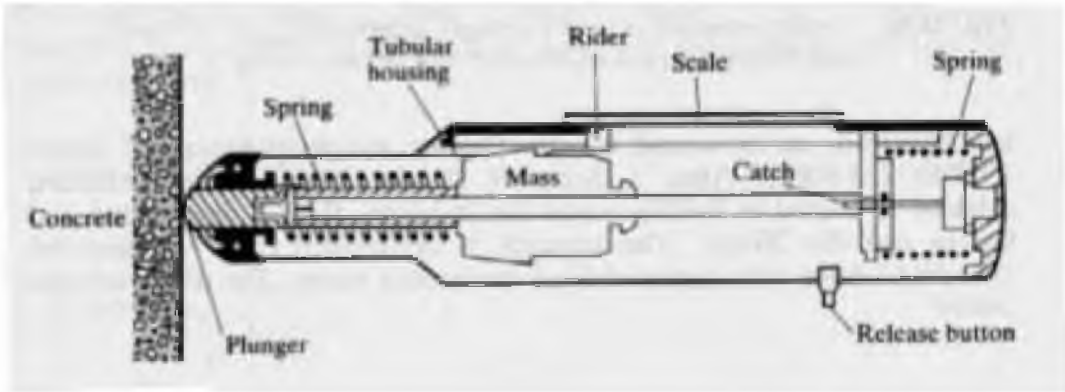


Figure 4.4: Schmidt Hammer

Upon release, the mass rebounds from the plunger, and the distance travelled by the mass, expressed as a percentage of the initial extension of the spring is called the rebound number; it is indicated by a rider moving along a graduated scale. The rebound number is an arbitrary measure since it depends on the energy stores in the given string and on the size of the mass. This test was rejected for this research as the concrete was not hard enough indicated by when the mass rebounded it indented the concrete.

The *penetration resistance test* and the *pull out test* were rejected as they are partially destructive tests. The penetration resistance test estimates the strength of the concrete from the depth of penetration by a metal rod driven into the concrete by a given amount of energy generated by a standard charge of powder. The pull out test measures the force required to pull out a previously cast-in steel rod with an embedded enlarged end. Because of its shape, the steel rod assembly is pulled out with a lump of concrete in the approximate shape of a frustum of a come. The pull out strength is calculated as the ratio of the force to the idealised area of the frustum. As it was decided that all specimens would be crushed using the compression test after intermediate readings had been taken, using any of the partially destructive tests would damage the specimens. Damaging the specimens and the crushing them would result in an unrealistic low compression strength and since the destructive compression test is the most reliable and accurate test available, the compression test method was adopted.

It is expected that the concrete made with sand "Y", the good sand, will give a 7day dry compressive strength of approximately 12 MPa, which is within the range of a 'normal' concrete. The range of concrete strength is usually between 10-70 MPa, which means that the estimated value is nearer the lower value. This is because it is expected that the method for making the specimens is more basic than methods used in normal UK concrete making.

It is expected that concrete made with sand "X", will have a 20% lower dry compressive strength than sand "Y", at approximately 9.6MPa. This is explained in the literature by the extra fines interfering with the chemical reaction in the cement and the water. Concrete is not generally used at strengths lower than 10MPa, so it is expected that sand "X" will not be suitable for use in concrete.

4.2.2 Shrinkage / Swelling

As mentioned previously, due to the high fines component in the composition of the sand of the concrete, shrinkage and swelling were considered to be important factors to be investigated. This is because:

"The high fines fraction in the material being tested means there is a high proportion of small particles having surface electrostatic charges to which a layer of water becomes attached", (Spence and Cook, 1983).

The thickness of this layer depends on the normal pressure to which the soil mass is subjected, and this gives fines, especially clays, their characteristic ability to swell and shrink as they take on and lose moisture.

Shrinkage can be divided into three main categories. They are,

- *Plastic,*
- *Autogenous,*
- *Drying.*

Plastic shrinkage is the shrinkage which occurs before the concrete has set or has attained any significant strength. The principle cause of such shrinkage is the rapid evaporation of water from the concrete surface. The restraint of the mass of concrete will cause tensile strains to be set up in the near surface region, and as the concrete has near zero tensile strength, plastic shrinkage cracking may result. Any tendency to plastic shrinkage cracking will be encouraged by greater evaporation rates of the surface water which occurs.

Autogenous shrinkage results in volume change without the loss of moisture. The contraction is relatively small and of significance only in large structures. This type of shrinkage is not examined in this report.

Drying shrinkage occurs when concrete is hardened and cured and allowed to dry. First, the water saturating the voids in the concrete dries out. This continues until the total moisture content is reduced by about half. Further drying results in water being drawn out of the mass of small capillaries which permeate the cement gel. This process continues on an ever decreasing scale for a long time. The rate at which the shrinkage occurs depends on the speed of water loss and on the moisture movement through the concrete; therefore in hot, dry and windy climates rates of shrinkage will be high.

So, when cement, aggregate and water are mixed together the gross volume decreases as the finer particles arrange themselves in the interstices of the larger particles. The shrinkage continues as the concrete is being worked into place, i.e. when still plastic. Evaporation of water in the mix also decreases the volume of such concrete.

Shrinkage, when the concrete is in a fluid state, does not matter structurally because no internal stresses can be instigated. When the concrete changes from a fluid to a solid state, further shrinkage of the concrete will cause internal stresses and even cause cracks to occur. If a mass of concrete shrinks or expands uniformly and its movement is not restricted by any external forces, then no internal stresses can be induced in the concrete. This seldom happens in practice; usually any movement of the concrete is restricted internally by reinforcement imbedded in the concrete, and often externally by its surroundings. Also, the surface of the concrete will often dry out (and therefore shrink) faster than the internal particles of concrete. It was therefore decided that drying shrinkage would be measured during the curing period of 56 days.

Shrinkage after the concrete has solidified continues as and when further water evaporates. The chemical reaction of cement with water, and thus the shrinkage, continues in the concrete seemingly indefinitely. A gel is formed which contracts upon desiccation and becomes very hard. If the concrete is submerged in water the cement gel expands with considerable force, so that the whole mass of the concrete expands and swells. This expansion, however, can never equal the shrinkage which has already taken place. On drying the concrete in air shrinkage again occurs. Therefore, when concrete is subjected to continual wetting and drying, it experiences corresponding expansions and contractions, (shrinkage and swelling). This is illustrated graphically in figure 4.5. It was decided to measure these expansions and contractions to examine the effect of a high proportion of fines had on these movements as it was expected that the specimens would be susceptible to damage on exposure to wetting and drying cycles.

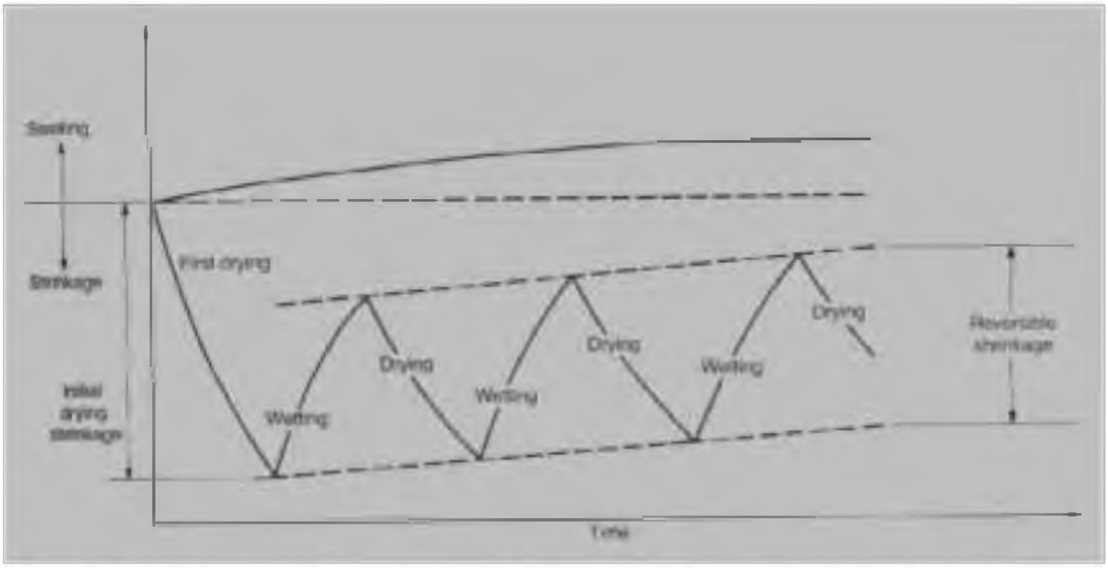


Figure 4.5: Schematic of volume changes in concrete due to alternate cycles of drying and wetting.

Maximum shrinkage occurs on the first drying, and a considerable part of this is irreversible; i.e. it is not recovered on subsequent rewetting. Further drying and wetting cycles result in more or less completely reversible shrinkage. The diagram also shows the continuous but relatively small swelling of the hardened cement paste (hcp) on continuous immersion of water. The water content first increases to make up for the self-desiccation during hydration and to keep the paste saturated. In principle the stronger the hcp structure, the less it will respond to the forces of swelling and shrinkage.

Shrinkage and swelling were measured using dial gauges that were set up on the specimen cubes. This was the only method considered as it is a cheap and effective way of measuring the volume changes with the specimens that had already been made for testing compressive strength. With a 100mm cube, strains could be measured to a resolution of ~ 100 μ-strains. Using the same specimens allowed better time management and enabled a larger range of results. The details of the procedure are described in the research methodology.

According to Illston, (1994), 'normal concrete' usually shrinks between 300-600 μ-strains in one year of drying. Therefore, it is expected that concrete made with the good sand, sand "Y", will have a drying shrinkage of approximately 600μ-strain, the upper limit, allowing for the 5% extra fines than recommended in BS 812. It is expected that concrete made with sand "X", will have a larger drying shrinkage, approximately 20% more shrinkage, of about 720μ-strain, since there is a larger proportion of fines. Cyclic shrinkage is thought to follow the usually pattern described above, but with increased shrinkage and swelling movements. Usually cyclic movements change between $\pm$400μ-strain, so it expected that concrete made with sand "X" will have movements between approximately $\pm$600μ-strain.

5. Research Methodology

5.1 General Approach

This project examined the factors affected by a high fines content when making concrete. This means that several iterations were used, each to change a factor, while holding all others constant to obtain a result. When a new result was obtained, a new understanding was gained and different avenues were explored. This structure led to most of the project being devoted to experimental work in the laboratory and analysing results. Alongside the experimental work, secondary data was collected to support the laboratory work and to justify the findings.

5.2 *Making the specimens*

The compressive strength test that was used uses a concrete cube and is the standard destructive test in the UK set out by the British Standard, BS 1881: 1983. The cube size tested was 100mm, as recommended for maximum aggregate sizes of 20mm or less. Some of these cubes were also used for the PUNDIT test and for the shrinkage and swelling measurements.

The mix components were chosen for experimentation was based on a soil-cement ratio of 4:1 by weight. This ratio was chosen because for high strength concrete a ratio of 3:1 is normally used, and for rendering, a ratio of 5:1 is commonly used. It was decided to use an in-between value in order to keep it constant throughout the duration of the project, so not to have another variable. The water content, vibration time, and mixing time were varied until the optimum, or best operating point was found, (this will be discussed in the analysis). The approximate density of the concrete to be tested was 2000kg/m^3, and the volume of the cube was 0.001m^3. Since:

Density (kg/m^3) = mass (kg) / volume (m^3), the mass needed for each block was approximately 2kg. That is, 1.6kg of aggregate and 0.4kg of cement.

The cubes were cast in lubricated steel moulds, accurately machined to ensure the opposite faces are smooth and parallel. A thin layer of lubricant was applied to the inside surfaces of the mould in order to prevent bond between the concrete and the mould. The concrete was placed and compacted using a vibrating table, as soon as the mix was made. This was so there would be no effect from the delay of moulding.

A vibrating table can be considered as a case of formwork clamped to the vibrator. A rapidly rotating eccentric weight makes the table vibrate with a circular motion but, by having two shafts rotating in opposite directions, the horizontal component of vibration can be neutralised, so that the table transmits a simple harmonic motion in a vertical direction only. The concrete was fully compacted by external vibration and filling the mould in two stages. The details of this procedure are prescribed by BS 1881: Part 108: 1983. When the mould was filled the surface was trowelled smooth.

The moulded specimens were stored for 48 hours at a temperature of 23 °C since curing at a higher temperature would result in a higher initial strength development, but a lower 28 day strength. If a much lower temperature was chosen, the concrete would also have a lower strength. Subsequently, the de-moulded cubes were stored at the same temperature in a polyethylene bag until the prescribed age of testing.

5.3 Compressive strength tests

5.3.1 Destructive compressive test

The cube testing machine has two heavy platens through which a uniformly distributed increasing axial compression load was applied to the concrete.

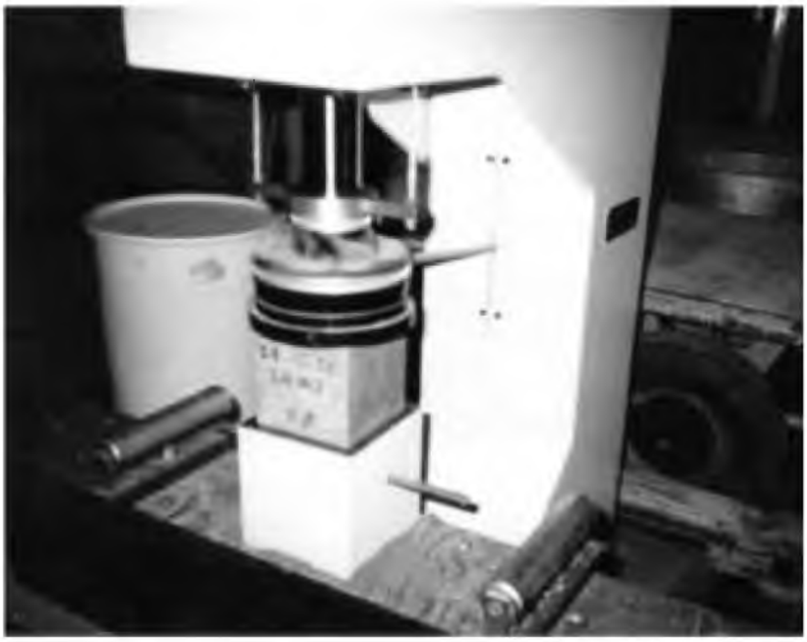


Figure 5.1: Compression Test Machine

The bottom platen is fixed and the upper one has a ball seating which allows rotation to match the top face of the cube at the start of loading. This then locks in position during the test. The load was applied to the pair of faces which were cast against the mould, i.e. with the trowelled face to one side. This ensured that there were no local stress concentrations which would result in a falsely low average failure stress.

The strength of concrete increases as the rate of loading increases so it is necessary to define the rate for testing purposes. McHenry and Shideler (1955) have shown that if the specimen is not subjected to impact or if the load is applied in a reasonable period of time, the rate of loading does not significantly influence strength. Therefore, a rate to reach the ultimate load in a few minutes is recommended, so a loading rate of 50kN/ Min was chosen. When the specimens had failed, the maximum load and type of failure was recorded.

The cracking pattern within the cube produced a double pyramid shape after failure, (see figures 5.2a & b).

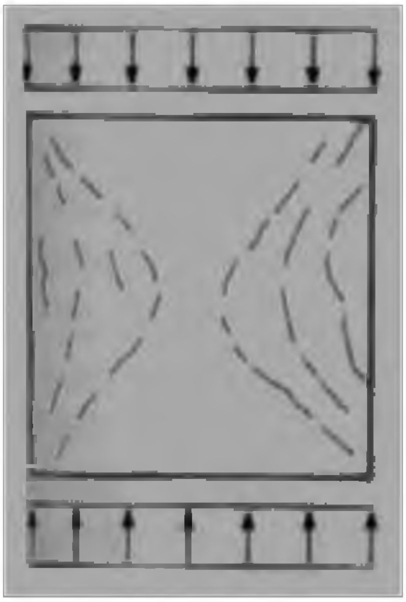


Figure 5.2a: Cracking Pattern Of a Cube

Figure 5.2b: Cube after failure

From this, it was immediately apparent that the stress within the cube was not uniaxial. The compressive load induced lateral tensile strains in both the steel platens and the concrete due to the Poisson effect. This mismatch between the elastic modulus of the steel and the concrete and the friction between the two resulted in lateral restraint forces in the concrete near the platen. The concrete cube was therefore in a triaxial stress state, with consequent higher failure stress than the true, unrestrained strength. This was the major objection to the cube test, but the test was relatively simple to perform and is capable of comparing different concretes.

The strength was expressed as the ultimate compression load per cross sectional area. This reading was converted into MPa for a reading of strength. This type of test is a destructive test and was applied to the cubes after seven days. No other results from the specimens tested could be obtained. Therefore, in order to obtain more results during the project time, it was decided to use an in-situ test which is a type of non-destructive test up until an age of 56 days and then crush them.

5.3.2 PUNDIT test

The PUNDIT test (Portable Ultrasonic Non-destructive Digital Indicating Tester), was used to indirectly test the compressive strength of the cubes made as described above.

Figure 5.3: PUNDIT in laboratory

Three types of waves are generated by an impulse applied to a solid mass. Surface waves having an elliptical particle displacement are the slowest, whereas shear or transverse waves with particle displacement at right angles to the direction of travel are faster. Longitudinal waves with particle displacement in the direction of travel are the most important since these are the fastest and generally provide the most useful information. Electro-acoustical transducers provide waves primarily of this type; other types generally cause little interference because of their lower speed.

The underlying principle of this PUNDIT is the recognition that factors that increase the concrete strength usually increase the velocity of the acoustic wave propagations in the material as well. The PUNDIT generates low frequency ultrasonic pulses and measures the time taken for a wave to propagate through the specimen of length 100mm. This enables the velocity to be determined from:

Pulse Velocity = Path length / Transit time

Path lengths and transit times were measured to an accuracy of ± 1%. This is because the pulse velocities for most concrete mixes lie within a narrow range so it is therefore necessary to measure both the transit time and path length to this accuracy. To acquire a high accuracy of transit time, good acoustic coupling between the transducer face and the concrete is needed. This was achieved by covering the transducer face with medium grease. A zero control is incorporated in the instrument since the zero is likely to change when different transducers are used. The instrument indicated the time taken for the earliest part of the pulse to reach the receiving transducer measured from the time it left the transmitting transducer when these transducers were placed on opposite surfaces of the specimen, (see figure 5.4). It was important that the readings were repeated by complete removal and re-application of transducers to obtain a minimum value for the transit time.

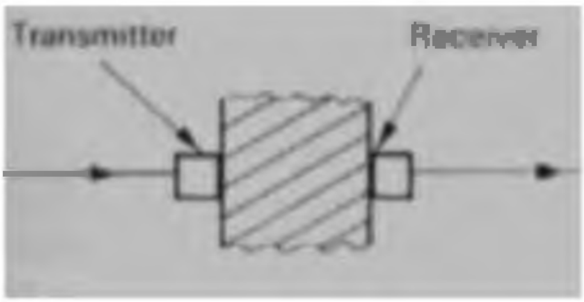


Figure 5.4: Diagram of PUNDIT on specimen

This arrangement for taking readings was the direct transmission arrangement, and is the most satisfactory method of taking readings since the longitudinal pulses leaving the transmitter are propagated mainly in the direction normal to the transducer face. Pulses were not transmitted through large air voids in a material, and if such voids existed directly in the pulse path, the instrument would have indicated the time taken by the pulse which circumvents the void by the quickest route.

It can be shown theoretically that the wave velocity depends upon the elastic properties, and hence if the mass and velocity of wave propagations are known it is possible to assess the elastic properties. The compression wave velocity is given by:

$$V = \sqrt{\frac{K.E}{\rho}}$$

Where V = Compression wave velocity (km/s)

$K = \dfrac{(1-v)}{(1+v)(1-2v)}$, (typically K = 1.3)

E = dynamic modulus of elasticity (kN/mm^2)

ρ = density (Kg/m^3)

v = dynamic Poisson's ratio, (typically 0.3).

In practice the velocity of such pulses travelling in concrete is very closely related to the elastic modulus since changes in the density or Poisson's ratio give rise to proportionately greater changes in elastic modulus.

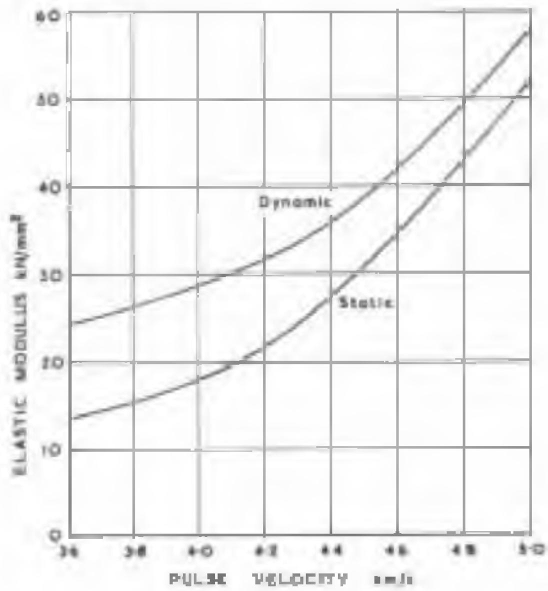


Figure 5.5: Curves relating Pulse Velocity with Static and Dynamic Elastic Modulus

The elastic modulus of concrete increases as its mechanical strength increases so that pulse velocity can be related empirically to cube strength. The exact relationship cannot be defined simply by consideration of the properties and proportions of individual constituents. This is because of the influence of the aggregate particle shape, efficiency of the aggregate, matrix interface and variability of the particle distribution, coupled with changes of matrix properties with age. There is no single curve which relates concrete strength to pulse velocity although, for a given cement content and aggregate type, a reasonable correlation exists between velocity and cube strength when this is varied by changing either the water cement ratio or the age at test. This correlation could not be used in the experimental work since the aggregate type is different to the "norm". In order for the Pundit Test results to be related to compressive strength, experimental collaboration of the specimens was necessary. Strength calibration was attempted as readings were taken between both pairs of opposite faces of cubes, which were then crushed in the usual way. Ideally, at least 10 sets of three specimens should be used, but time constraints did not allow this, so only 3 blocks for each type of specimen were crushed at 7 and 56 days. 6 pulse velocity readings were taken for each cube, and each individual reading was within 5% of the mean for that cube.

Although the precise relationship is affected by many variables, the curve is usually found to be of the general form:

$$f_c = Ae^{B\nu}$$

Where f_c = equivalent cube strength

e = base of natural logarithms

ν = pulse velocity

and A and B are constants.

Hence, a semi-logarithmic plot of pulse velocity against cube strength is linear for a particular concrete. It is therefore possible to use a curve derived from reference specimens to extrapolate from a limited range of results. This process is done in section 6.1.8.

5.4 Shrinkage & swelling measurements

Shrinkage and swelling measurements were done on the cubes made for the PUNDIT test. During the curing process, once the specimens had been removed from the moulds, the blocks were set up as shown in the photograph in figure 5.6.

Figure 5.6: Specimen during the curing process

Point dial gauges were set up on the side face of the specimens, and readings were taken at 7, 14, 28, and 56 days respectively. This allowed measurements of the drying shrinkage to be made. As it can be seen from the photographs, the blocks were covered in a polyethylene bag to keep the curing conditions the same for each specimen. Because of stick and hysterisis in the dial gauges, measurements were always approached from one side (above), so that the friction in the spring could be ignored. At 56 days, the specimens were measured using callipers and were then placed in a drying oven at 105°C and left to dry out for 7 days. After this period of time the cubes were re-measured, then submerged in water with a dial gauge set up on it. The movement in the specimen was recorded after 7 days, and the drying wetting cycle was carried out again.

The point dial gauges measure to an accuracy of ± 0.005mm and the callipers to an accuracy of ± 0.05mm. It was necessary to use the callipers for the drying part of the cycle, since the drying oven was not large enough to set up a dial gauge. This meant that the accuracy of the shrinkage measurements during the cycle were much less accurate than the swelling measurements.

6. Results & Analysis

6.1 Strength

The first property of the concrete that was tested was the dry compressive strength, after a curing period of 7 days. When the strengths of comparable concretes, (i.e. the sand with a high fines content, and the one without), with identical water-cement ratios were made, the strength of the concrete unexpectedly *increased* as the fines content increased. This is shown in table 6.1 below.

		Maximum load (kN)				
	Water-cement Ratio	Block 1	Block 2	Block 3	Mean Maximum Load (kN)	Mean Pressure (MPa)
Sand "X" Cube	0.95	122.2	120.1	121.1	121.1	12.1
Sand "Y" Cube	0.95	79.9	79.3	79.2	79.5	7.9

Table 6.1: Comparable concretes with identical water-cement ratios after 7 days

This was an unexpected result since it was expected that the fines would interfere in the bond between the hardening cement paste and the aggregate. The strength of concrete originates from the strength of the hardening cement paste, which originates from the hydration products in the form of a rigid gel, the cement gel.

"It is reasonable to assume that the bonds of the gel particles to each other, to the sand particles, and to other bodies in the concrete are responsible for strength", (Popovics, 1998).

A likely explanation as to why the compressive strength was higher for the cubes with more fines is that the bond to the hardening cement paste to aggregate particles of larger sizes is weaker than the bond to smaller sizes because of the specific surface of the former. Also, the 'fines' are siliceous substances of high surface area that react with the excess alkali in the cement to form additional 'pozzolanic' gel.

The strength being discussed is this research is the strength of small samples. The strength of these small samples is generally higher than that of core samples taken from a structure and there are several reasons for this:

1. The cube specimens were compacted, cured and tested under controlled conditions; a structure (or core samples taken from it) is not subjected to the same controlled conditions. Compaction and curing in a structure is generally less than the optimum that would be achieved with the test specimens.
2. The concrete is a structure is subjected to restraint stresses due to the reinforcement member geometry and other factors, and this pre-test load will influence strength of a core taken from a structure.

Interpretation of the Strength Readings

The strength of the concrete samples made from the same mix and tested in the same manner are variable and if a population (or large sample) of results are considered, the variation follows a normal distribution. The characteristics of a normal distribution are the arithmetic mean (x), which is the sum of the strength results divided by the number of results, the standard deviation (σ), which is the root mean square deviation of a set of results from their arithmetic mean, and the coefficient of variation ($v_c = (\sigma/x) \times 100$), which is a measure of the relative variability of a set of results. The results are said to follow a normal distribution if they are equally spaced about the mean value and if the largest number of cubes have strength close to the mean value, the number falling off as the results are much greater or less than the mean value. In some cases it may be considered doubtful as to whether actual results lie within a normal distribution, but for the purpose of statistical analysis they are considered to do so. The range of a set of strength results will be reflected in the values of standard deviation and coefficient of variation; the higher these values the greater the range. The variability of concrete is a reflection of the manner in which it was made and tested, i.e. the degree of quality control.

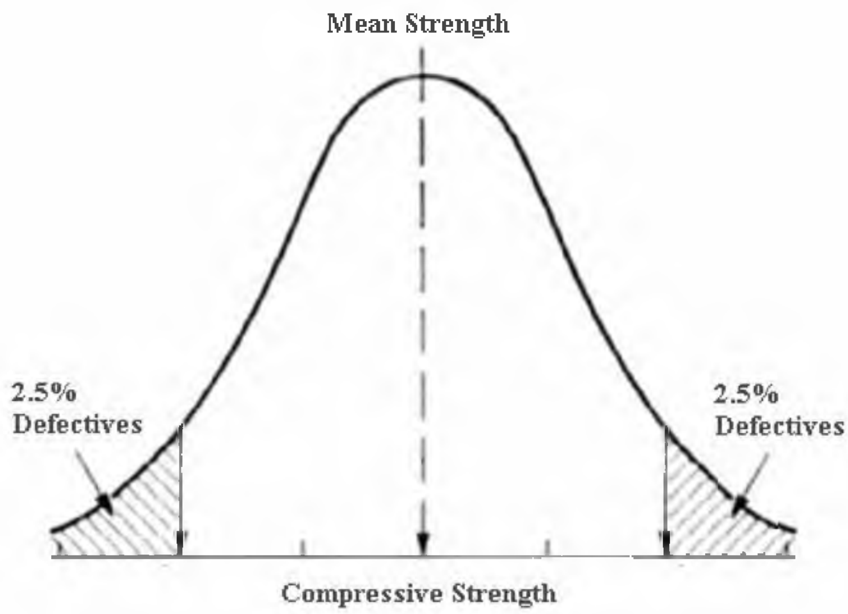


Figure 6.2: Normal distribution for concrete strengths

The mean value of a number of results gives no indication of the extent of variation of strength. The extent of variation can be determined by relating the individual strengths to the mean strength and determining the variation from the mean. The variations which occur in compressive strength test results, even when testing all the concrete from one batch, are not due to faulty workmanship but are inherent in the making and testing of concrete. The amount of scattering is in itself a measure of quality; where the sampling of concrete is not restricted to one batch but is spread over a number.

The variability of the properties of concrete can only be determined by testing, but testing itself can introduce error. Precision is the general term used for the closeness of agreement between replicate test results, and two terms are relevant: repeatability and reproducibility.

BS 5497: Part 1: 1979 defines repeatability as the value below which the absolute difference between two single test results, obtained with the same method on identical test material under the 'same' conditions (i.e. same operator, same apparatus, same laboratory, and a short interval of time), may be expected to lie within a specified probability (usually 95%). On the other hand, reproducibility is defines as the value below which the absolute difference between two single test results, obtained with the same method on identical test material under 'different' conditions, (i.e. different operators, different apparatus, different laboratories, and/or different times), may be expected to lie within a specified probability (usually 95%).

Values of repeatability and reproducibility are applied in a variety of ways, e.g.

- to verify that the experimental technique of a laboratory is up to requirement;
- to compare the results of tests performed on a sample from a batch of material;
- to compare test results obtained by a supplier and by a consumer on the same batch of material.

Using this statistical analysis, it can be shown that the results from comparable concretes with identical water-cement ratios can be proved to be statistically significant:

Testing for a difference between two normal distributions

	Result number	Water-cement Ratio	n	x	σ_{n-1}
Sand "X"	X_1	0.95	3	12.1	0.105
Sand "Y"	X_2	0.95	3	7.9	0.038

Table 6.3: Data on the 2 different batches of concrete for significance testing

If two sets of results, X_1 and X_2, have independent normal distributions with means of x_1 and x_2 and standard deviations σ_1 and σ_2 respectively then the difference Δ between two populations is another normally distributed entity

$$\Delta = X_1 - X_2 \sim N(x_1 - x_2, \sigma_1^2 + \sigma_2^2)$$

Since X_1 and X_2 are based on sample sizes of n_1 and n_2, respectively from the above two normal populations then

$$\bar{X}_1 - \bar{X}_2 \sim N\left(x_1 - x_2, \frac{\sigma_1^{\ 2}}{n_1} + \frac{\sigma_2^{\ 2}}{n_2}\right)$$

and the statistic $\bar{X}_1 - \bar{X}_2$ can be used to test hypothesis about the values of x_1 and x_2.

If $X_1 \sim N(x_1, \sigma_1^{\ 2})$ and the independent random variable $X_2 \sim N(x_2, \sigma_2^{\ 2})$ then a test of the null hypothesis H_0: $x_1 = x_2$ can be carried out using the test statistic

$$Z = \frac{\bar{X}_1 - \bar{X}_2}{\sqrt{\left(\frac{\sigma_1^2}{n_1} + \frac{\sigma_2^2}{n_2}\right)}} \sim N(0,1^2)$$

So, the null hypothesis is

$H_0 : x_1 = x_2$ $\qquad$ $H_1 : x_1 \neq x_2$

In this comparison of strength of cubes made with sand “X” and sand “Y” respectively:

$\sigma_1 = 0.105$, $n_1=3$, $\sigma_1 = 0.0.38$ and $n_2=3$

$x_1 - x_2 = 12.1 - 7.9 = 4.2$, where σ and x are measured in MPa.

Thus, for this experiment, the test statistic Z has the value:

$$Z = \frac{\bar{X}_1 - \bar{X}_2}{\sqrt{\left(\frac{\sigma_1^2}{n_1} + \frac{\sigma_2^2}{n_2}\right)}} = \frac{4.2}{\sqrt{\left(\frac{0.105^2}{3} + \frac{0.038^2}{3}\right)}} = 65.15$$

The 5% (two tailed) critical values for z are $Z_{95} = \pm 1.96$

Hence 65.15 is a statistically significant value (i.e $z >> z_{95}$) and H_0 can be rejected. It can be concluded that there is significant evidence of a difference in means of the two results.

This statistical analysis was applied to all relevant results, and is used in later sections of the analysis. The main factors which affect strength are taken to be quality of water, cement, mixing time, vibration compaction time, curing conditions, workability, water-cement ratio and curing age. Each of these factors will be looked at it in turn.

6.1.1 Water Quality

Water is one of the main constituents of concrete. It has various functions; it reacts with the cement powder, causing it to set and harden, and it is a lubricating liquid which enables the concrete to be placed as a semi-fluid and so facilitates compaction.

"It is generally accepted that water which is fit for drinking is suitable for mixing concrete", (Neville & Brooks, 1987).

In this country it is doubtful whether there is any justification for using anything other than water supplied for domestic purposes. In some situations, however, it is necessary to use sea water. Sea water contains sodium and magnesium chlorides and sulphates. The action of the chloride ions is to accelerate early strength development while the sulphate ions reduce the later age strength. Mindless and Young (1981), indicate that the 28 days strength can be reduced by as much as 20% if sea water is used as mix water. Also, the addition of chloride ions to concrete increases the risk of corrosion of the reinforcing steel. Hence, because of the generally poor standard of concrete construction in the Third World countries, it is considered that sea water should, as far as it is practical, not be used. For the purposes of this study, only tap water was used for mixing.

6.1.2 Cement quality

Cement is the most important and expensive ingredient of concrete, on a price per tonne basis. It was patented by J.Aspdin in the UK in 1824 and he called his product Portland cement because of the 'artificial stone' (concrete) made with it resembled Portland stone.

The chemical composition of Portland cement consists essentially of four main elements; lime, silica, alumina and iron oxide. These elements interact with one another in the kiln to form a series of more complex products. Of these, four of the products are considered to be the major constituents of cement:

- Tricalcium silicate, C_3S
- Dicalcium silicate, C_2S
- Tricalcium aluminate, C_3A
- Tetracalcium aluminoferrite, C_4AF

Each grain of cement consists of an intimate mixture of these compounds.

There are several types of Portland cement available commercially, but the most common type used in concrete construction is Ordinary Portland Cement. Its related British Standard is BS 12:1978. Over the years, there have been changes in the characteristics of ordinary Portland cement: modern cements have a higher C_3S content and a greater fineness than 40 years ago. In consequence, modern cements have a higher 28 day strength than in the past, but the later gain is smaller. For the purposes of this research ordinary Portland cement was used to make the cubes since it is the cheapest and most widely available cement for builders in developing countries to use.

6.1.3 Mixing time

"For onsite mixing or in the laboratory, the essential requirement in mixing concrete is to produce a uniform mixture in as short a time as possible", (Spence and Cook, 1983).

The mixing operation consists of essentially of rotation and stirring, the objective being to coat the surface of all the aggregate particles with cement paste, and to blend all the ingredients of concrete into a uniform mass. Mixing can be achieved in pan or drum type mixers, or it can mixed by hand which is generally less effective in achieving the objective above. Nevertheless, hand mixing was used to make the concrete mix since this is the most likely method to be used in developing countries.

It is essential to mix the concrete well since the strength of concrete is also affected by the efficiency of mixing. Poor mixing results in a bad distribution of the coarse and fine aggregates through the mix, so that some parts are lean and some parts are rich in cement. The lean parts are not fully compacted, whilst the rich parts may be over compacted, resulting in some wet segregation.

Mixing time is considered to have an asymptotic effect, so provided it was 'very high' its actual value would not affect the strength of the concrete. However, in case this assumption was unreliable, some attempt was made to hold the mixing time constant. It was decided to hold the mixing time constant at 5 minutes for a mix of 5 kg. This time was chosen not in relation to the optimum mixing time, but instead was chosen due to the time constraints on a builder in a developing country. Although mixing for a much longer period increases the strength it is not done in practice, so an arbitrary value was chosen.

6.1.4 Vibration compaction time

The compaction of concrete is the process whereby the amount of voids is reduced to a minimum and the particles of aggregate are constrained to pack more closely together so as to achieve the minimum potential density and strength from the concrete.

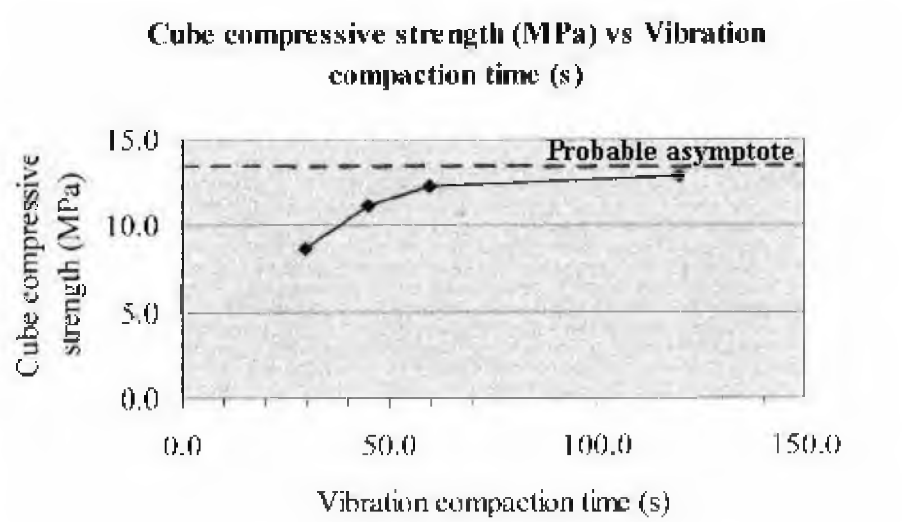
Vibration does not increase the workability of concrete and can be detrimental by causing segregation of the constituents of the concrete, the gravel particles tending to sink to the bottom, and the sand and cement to float to the top of the concrete. The vibration employed should *only just be sufficient* to make the concrete flow into the sharp corners of the mould and around the reinforcement.

Four batches of three specimens were made to test the effect of vibration time on compressive strength when using a high fines content concrete. The batches were vibrated for 30 seconds, 45 seconds, 1 minute and 2 minutes respectively.

It was discovered the compressive strength increases with vibration time in the form of an asymptotic relationship. The difference of 2 minutes vibration time compared to 1 minute gave a 5% increase in strength, whereas between 45 and 60 seconds, there was a 10% increase in strength. Therefore in terms of costs, one minute vibration time was considered sufficient, as this time is nearing its probable asymptote. This is shown in figure 6.5.

	Maximum Load (kN)					
Vibration Time (seconds)	Block 1	Block 2	Block 3	Mean Maximum Load (kN)	Mean Pressure (MPa)	Load Normalised to Max. Load
30	87.5	87.2	87.4	87.4	8.7	0.68
45	111.2	111.4	111.7	111.4	11.1	0.86
60	122.8	122.7	123.4	123	12.3	0.95
120	129.4	129.1	128.9	129.1	12.9	1.00

Table 6.4: Vibration compaction time against Cube compressive strength



Graph 6.5: Vibration compaction time against Cube compressive strength

6.1.5 Curing Conditions

Curing is the name given to procedures used for promoting the hydration of cement, and thus, the development of strength of concrete, the curing conditions being control of the temperature and of the moisture movement from and into the concrete.

Although there are several ways to cure concrete, including curing under water, the project focuses on methods used in developing countries.

In tropical and subtropical countries, the temperature during the day can regularly exceed 30°C and special precautions have to be taken, otherwise the concrete may crack and its strength will be impaired.

"Curing in developing countries means that there is a difficulty with fresh concrete because the temperatures are above 'normal', meaning that the concrete can stiffen before it is compacted and finished", (ACI Committee 305, 1982).

The specific harmful consequences of this on the fresh concrete are:

- Increased water demand
- Increased rate of slump loss
- Increased rate of setting resulting in greater difficulty of handling and curing
- Increased tendency for dry shrinkage
- Decreased durability
- Increased creep.

During placement the concrete temperature should be kept low. The optimum temperature range for the setting and early hardening of concrete is between 10 to 20°C. Control of concrete temperature through the temperature of ingredients can only be done at the point of batching and mixing. According to Popovics (1998), since the greatest portion of concrete is aggregate, reduction in aggregate temperature brings about the greatest reduction in concrete temperature. This can be done by shading the supplies.

These hot climate conditions have not been simulated, since it would be too expensive to do so, but it is assumed that the aggregate is kept well shaded, at a temperature of approximately 18°C.

At present, many builders in developing countries do not understand the consequences of curing concrete at high temperatures, and wrongly assume that leaving the concrete to cure in the sun is acceptable. In fact, a higher temperature maintained throughout the life of a concrete will result in higher short term strengths but also, lower long term strengths. A temperature of fresh concrete higher than normal results in a more rapid hydration of cement and leads therefore to accelerated setting and to a lower long-term strength of hardened concrete since a less uniform framework of gel is established.

Also, as water is relatively scarce, so it is unlikely that artisanal builders would be willing to adopt under-water curing methods. It has therefore been suggested that concrete should be cured in tightly sealed polyethylene bags in developing countries. This method has been adopted for the purposes of this study.

6.1.6 Workability

The strict definition of workability is the amount of useful internal work necessary to produce full compaction. The useful internal work is a physical property of concrete and is the work or energy required to overcome internal friction between the individual particles in the concrete. In practice, however, additional energy is required to overcome the surface friction between the concrete and the formwork. Also, waste energy is consumed by vibrating the form and in vibrating the concrete that has already been compacted.

More usefully, workability means the ease at with which the concrete can be handled from mixing to its final fully compacted position. Workability is influenced by the type of aggregate, its maximum size, shape and grading. Since the sand being tested contained a large number of fines, more water was needed to increase the workability to an acceptable level. A higher workability of concrete means it is easier to place and handle, but if the workability exceeds a certain limit for a particular type of material, a lower strength and durability will result. A balance needed to be made between the workability and water content in order to produce an optimum mix. The water content was optimised to obtain “the highest strength compatible with acceptable workability”, and will be discussed in the next section.

Therefore, the workability chosen was the lowest workability consistent with efficient handling and placing of the concrete into the moulds. The workability was deemed acceptable if the fresh concrete was able to be mixed relatively easily by hand, and as a general rule the workability was considered optimised when the mix was just about to reach its plastic limit, as judged by eye.

6.1.7 Water-cement ratio

"The water cement ratio is defined as the ratio of the mass of free water to the mass of cement in the fresh concrete", (Newman, 1959)

The quantity of water and cement are expressed in kilograms, which provides the water-cement ratio by mass. The total water in a concrete mix divides between the water absorbed by the aggregate to bring it to a "saturated surface dry" condition, and the free water available for the hydration of the cement and for the workability of fresh concrete.

The basis of the concrete strength versus water-cement ratio relation is that an increase in the water-cement ratio produces more capillary pores in the matrix portion of concrete. The higher the water cement ratio, the more diluted the cement paste becomes; therefore the weaker it will be at any stage of hydration. But, this relationship between concrete strength and water-cement ratio is only approximate because it may be affected by secondary factors.

A high strength concrete requires to be as free from voids as possible. If water in excess of the amount required for the chemical reaction with the cement is present in the mix, the water remains in a free state and the concrete sets around the drops of water. Such particles of water form pores and voids in the concrete, resulting in weakness and permeability. Dependent on curing conditions, they may freeze and expand, causing corrosion and / or eventually evaporating into the atmosphere.

The water cement ratio was altered in the experiments by changing the water content while keeping the cement content constant. When the cement content is kept constant and the water content is changed, the concrete consistency changes dramatically. When the consistency of a concrete is already so stiff that its workability is just barely suitable for the available method of compaction, any reduction in the water cement ratio caused by a decrease in the water content will lead to an overly dry consistency. The consequence of this is a reduction in the concrete strength because the impaired workability will result in incomplete compaction. By contrast, if the consistency is fluid, it is difficult to achieve a homogeneous, cohesive concrete without significant segregation.

Therefore, experiments were carried out in the laboratory to find the optimum water-cement ratio in relation to dry compressive strength using sufficient workability as discussed previously. The results are shown in figure 6.6 and 6.7.

Before the water was added to the mix, a soil moisture test was carried out on the sand with the extra fines to see how much water is already within the material. If the amount of water already in the material was considered significant it would be added onto the water content when calculating the water-cement ratios.

80g of sand "Y" & 20g of Kaolin was placed in a heat proof dish. (This is the composition of sand "X"). The combined dish and material was weighed to an accuracy of ± 0.005g and then placed in a drying oven which maintained a temperature of 105°C. After 7 days the combined dish and material was re-weighed. The difference in weights between these two readings is the amount of water, in grams, that the material contained. This amount excludes those water molecules bonded so tightly to solid surfaces that even a temperature of 105 °C will not detach them.

Soil Moisture Test Results

Sand "Y" and Kaolin = 100g
Dish = 157.78g
Combined weight before drying = 257.78g
Combined weight after 7 days in drying oven = 257.14g
Amount of water loss = 257.78g – 257.14g = 0.64g = 0.64%

Kaolin was also tested for moisture on its own, and showed no decline in weight, hence it contained no water. It can therefore be concluded the sand contained 0.64% of water. It was decided to take this value into account when working out the water-cement ratios, so 0.64% of water was deducted from every amount of water added to the concrete.

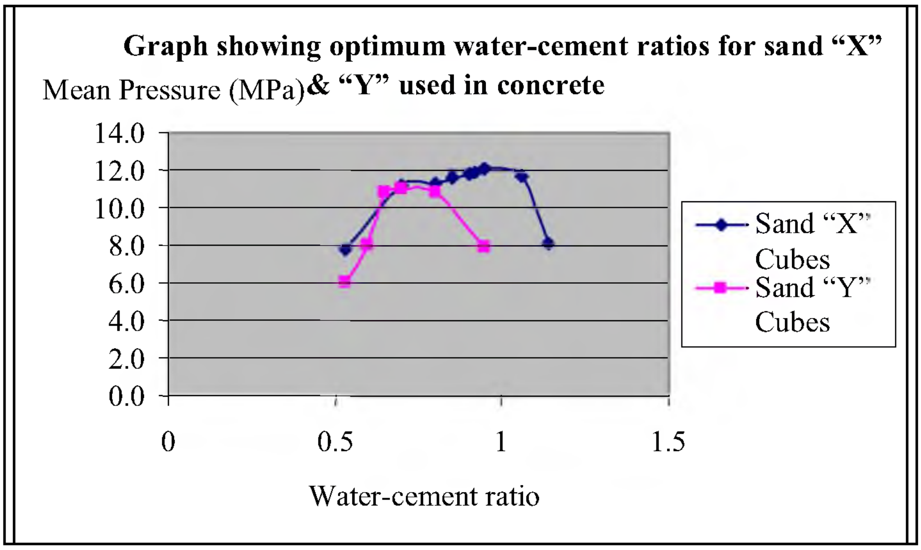
Optimum Water-cement Ratio Results

Sand "X"		Maximum load (kN)					
Water normalised to optimum	Water-cement Ratio	Block 1	Block 2	Block 3	Mean Maximum Load (kN)	Mean Pressure (MPa)	Load normalised to max. load
0.56	0.53	78.2	78.1	78.4	78.2	7.8	0.64
0.74	0.7	112.1	112.6	112.4	112.4	11.2	0.92
0.85	0.8	113.4	113.5	113.2	113.4	11.3	0.93
0.9	0.85	116	115.8	116.2	116	11.6	0.95
0.95	0.9	117.8	117.5	117.4	117.6	11.8	0.97
0.97	0.92	119.1	119.5	119.3	119.3	11.9	0.98
1	0.95	122.2	120.1	121.1	121.1	12.1	1
1.11	1.06	117.5	117.3	117.2	117.3	11.7	0.96
1.2	1.14	80.9	80.5	80.7	80.7	8.1	0.68

Table 6.6: Water-cement ratios versus compressive strength of Sand "X" cubes, (75% sand, 4.8% silt and 20.2% clay).

Sand "Y"		Maximum load (kN)					
Water normalised to optimum	Water-cement Ratio	Block 1	Block 2	Block 3	Mean Maximum Load (kN)	Mean Pressure (MPa)	Load normalised to max. load
0.76	0.53	62.1	61.1	57.1	60.1	6	0.55
0.86	0.6	80.2	80.5	80.4	80.4	8	0.73
0.93	0.65	107.7	107.3	107.9	107.6	10.8	0.98
1.00	0.7	110.1	109.9	110.5	110.2	11	1.00
1.14	0.8	108	107.8	108.1	108	10.8	0.98
1.36	0.95	79.9	79.3	79.2	79.5	7.9	0.72

Table 6.7: Water-cement ratios versus compressive strength of Sand "Y" cubes,(95% sand, 4.8% silt and 0.2% clay).



Graph 6.8: Graph showing optimum water-cement ratios for sand "X" and sand "Y" used in concrete.

Statistical analysis, similar to the method used in 6.1, was carried out on these results and is shown in Appendix A3.

As it can be seen by the figures shown in red, it was found that the concrete made from sand "Y" had an optimum water-cement ratio of 0.7, compared to a water-cement ratio of 0.95 for the sand "X" concrete. These optimum water-cement ratios were determined using compressive strength after seven days of curing.

"The ratio of water to cement for concrete made from 'good quality' concreting sand usually varies between 0.36 and 1.0 depending on the richness, and is usually approximately 0.4", (Akroyd, 1962).

The water-cement ratios for the materials used in this project are higher than the water-cement ratio recommended in the literature. This is because the concreting sand that was used had a poor grain distributions and a large amount of fines. Fines cause an increase in the water demand by the concrete to produce a specific workability. As a result, the strength of the concrete *should* decrease, as a consequence of the water-cement ratio rule. The reasoning associated with this is described below.

The important effect of the water-cement ratio, by weights, on the strength of concrete was published in 1918 by D. Abrams of Chicago, who stated that the strength of any workable concrete, of constant materials other than water, was dependent on the water-cement ratio alone, assuming the same cement, and degree of compaction are used and the conditions of curing and age at comparison of strengths are constants. The concretes compressive strength is inversely proportional to the water-cement ratio. This means that as the amount of water increases above that necessary for complete hydration of the cement it produces a more porous structure and results in a decrease in strength.

But, as can be seen from the results, *a higher water-cement ratio produced higher compressive strengths*. This was probably due to the fines "stealing" water from the cement and hence more water was needed to allow the full chemical reaction to occur with the cement. Once the chemical reaction had occurred with a sufficient amount of water, the bond of the hardened cement paste to the aggregate particles of larger sizes was weaker than the bond to the smaller sizes because of the smaller specific surface of the larger particles, hence increasing the compressive strength. But, the results also show that if the water cement ratio increased to 12% over the ideal, the decrease in dry compressive strength is large, approximately 68% of the strength for the optimum water cement ratio. This implies that the fines have "stolen" the water they need and too much is available for the chemical reaction, diluting the cement water paste and hence decreasing the compressive strength.

6.1.8 Curing age

It is experienced that the strength of a concrete increase with the progress of the hydration, that it, with age. When the hydration process stops, the strength development also stops. Hydration stops when any of the following three conditions occur:

- No more un-hydrated cement is available in the concrete.
- There is not enough free water.
- Diffusion can no longer take place.

Regardless of the hydration process, no retrogression in strength is permitted at any age. Such retrogression is always the warning sign of deterioration of the concrete due to some attack from the outside, faulty cement, reactive aggregate, or other cause.

Experiments were undertaken in the laboratory to verify this relationship. The two different concreting sands were used to make 12 cubes of each type. 7 day dry compressive strengths had previously been noted, so PUNDIT readings were taken at 7, 35 and 56 days. Ideally, readings should have been taken at 14 and 28 days but the timing of the Christmas vacation did not allow this. The full PUNDIT results tables for the two different types of specimens are shown in Appendix A4. The blocks were then crushed, and a calibration curve was drawn. The calibration results table is shown in Appendix A5 and graphs are shown in figures 6.9a and 6.9b below.

As mentioned previously, the precise relationship between the Pulse velocity and Compressive strength if affected by many variables, but the curve usually takes the general form:

$$f_c = Ae^{Bv}$$

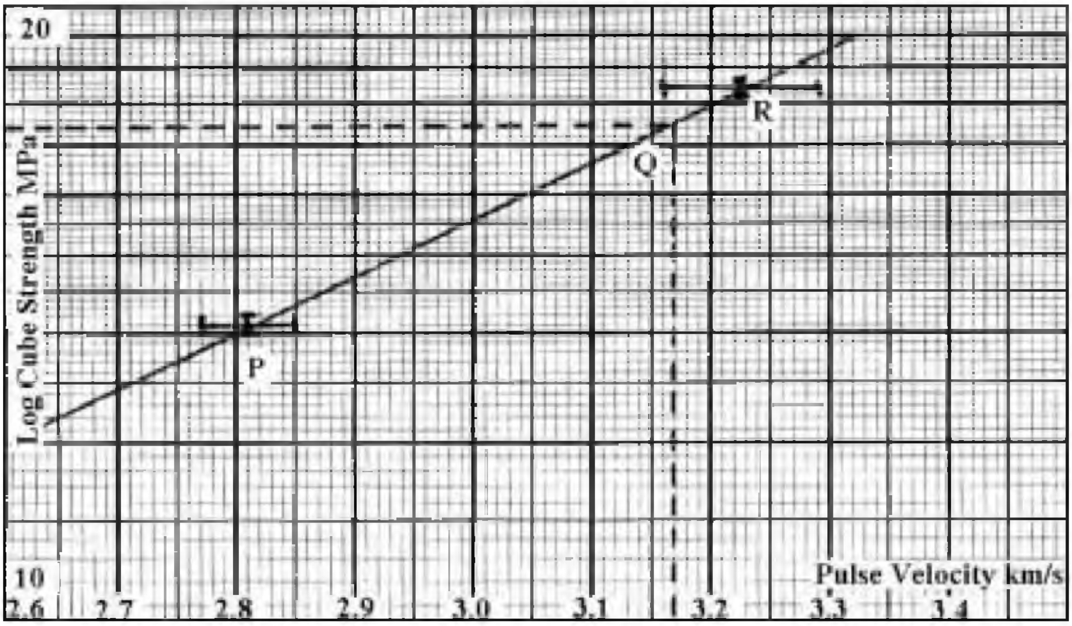
Where f_c = equivalent cube strength

e = base of natural logarithms

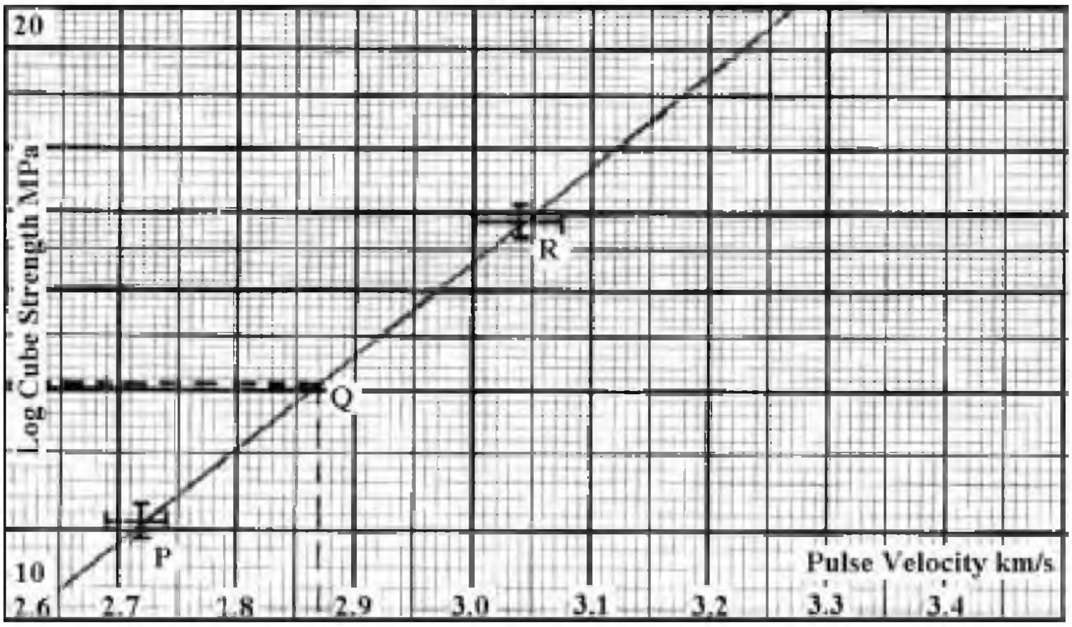
v = pulse velocity

and A and B are constants.

Hence, a semi-logarithmic plot of pulse velocity against cube strength was plotted, and a straight line was achieved.



Graph 6.9a: Calibration graph of Cube Strength against Pulse Velocity for Sand "X" Cubes.



Graph 6.9b: Calibration graph of Cube Strength against Pulse Velocity for Sand "Y" Cubes.

In graphs 6.9 a & b;

Point P represents strength-velocity calibration with 7 day specimens.

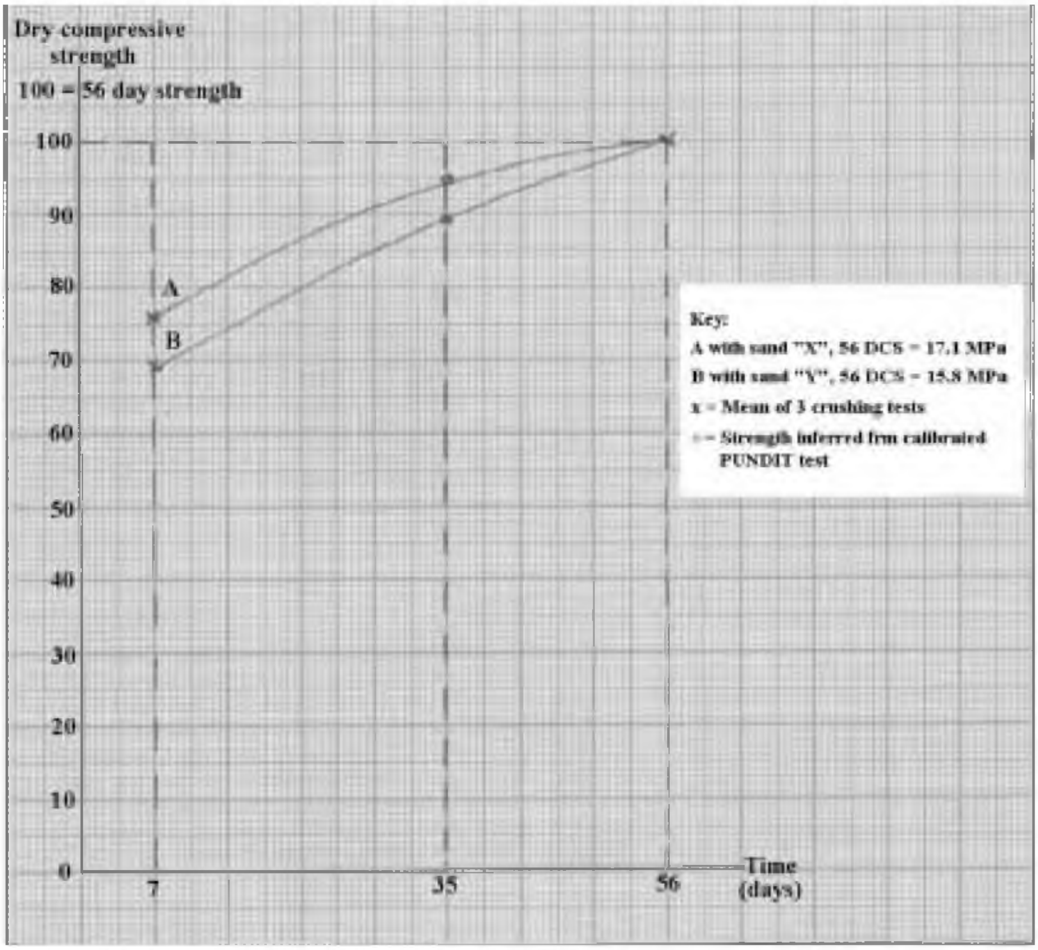
Point R represents strength velocity calibration with 56 day specimens.

Point Q is the inferred strength from the velocity measured at 35 days.

The graphs show the range of results so the line could have been drawn at any angle through the plotted lines. This shows there is some variation in results, especially in the pulse velocities, so the line of best fit has been drawn. The dotted lines on the graphs show the pulse velocity readings at 35 days, where no cube strength was measured. The predicted cube strength at this age is 16.4 MPa for the Sand “X” cubes and 13.1 MPa for the Sand “Y” cubes.

As it can be seen from both sets of results the degree of hydration increases with age, leading to the effect of age on strength. But, the hydration reactions are never complete, and in the presence of moisture, concrete will continue to gain strength for many years, although the rate of increase after such times will be very small.

The following graph, figure 6.10, shows the dry compressive strength development over a period of 56 days. Curve A shows the strength development of the concrete made with the poor sand “X”, and curve B is plotted for the good sand “Y”.



Graph 6.10: Graph showing the development of dry compressive strength over time

This graph shows that concrete made with sand “X” has a higher initial strength at 7 days, but has a slower development of strength to 56 days than sand “Y”. Concrete made with sand “Y” has a greater development of strength over a period of 56 days. This implies that if both types of concrete were left for many years, it could be possible that concrete made with sand “Y” could eventually develop a higher dry compressive strength than sand “X”.

This graph can also be used to obtain a factor by which to multiply the 7 day dry compressive strength results to obtain ‘standard’ 28 day characteristics. In concrete literature, it is more usual to specify the 28 day strength rather than 35 days, but as stated previously vacation timing did not allow the 28 day readings to be taken. The graph implies that for concrete made with the good sand “Y”, the 28 day strength is a 1.16 times more than the 7 day strength, implying that the 28 day strength is 12.64

MPa. For the concrete made with sand "X", the 28 day strength is 1.06 times more than the 7 day strength, implying that the 28 day strength is 13.78 MPa.

6.2 Shrinkage / Swelling

6.2.1 Drying Shrinkage

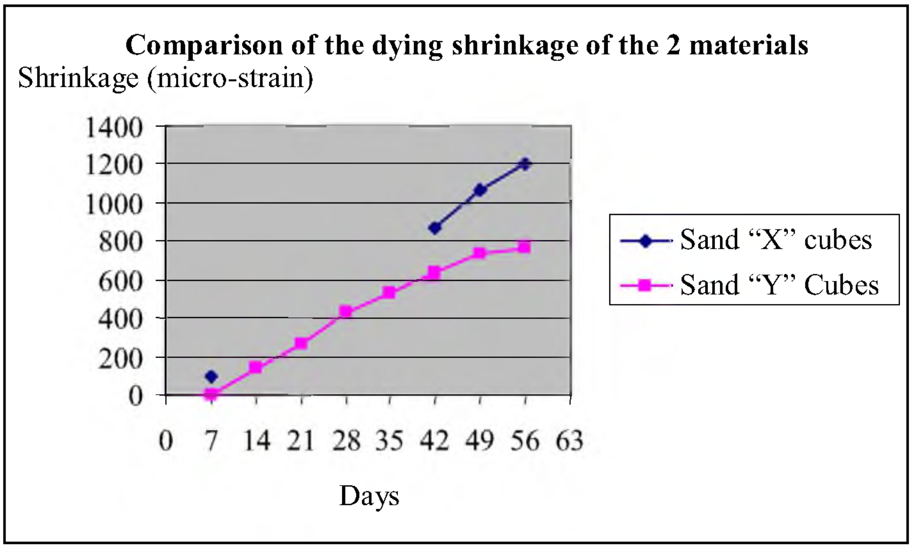
During the curing period of 56 days the shrinkage was measured. The specimens that were tested were ones that were made using the optimum water-cement ratio and using all other suitable conditions as described earlier. These specimens produced the following results:

		Total Curing Shrinkage (in μ-strains) after								
Sand "X" Cubes	Block Number	7 days	14 days	21 days	28 days	35 days	42 days	49 days	56 days	Direction of Movement
Dry cured	3	100	holiday	holiday	holiday	900	1100	1200	1300	Shrinkage
Dry cured	6	100	holiday	holiday	holiday	600	800	900	1000	Shrinkage
Dry cured	9	100	holiday	holiday	holiday	600	700	1100	1300	Shrinkage
Wet cured	16	-100	holiday	holiday	holiday	-500	-600	-700	-800	Swelling

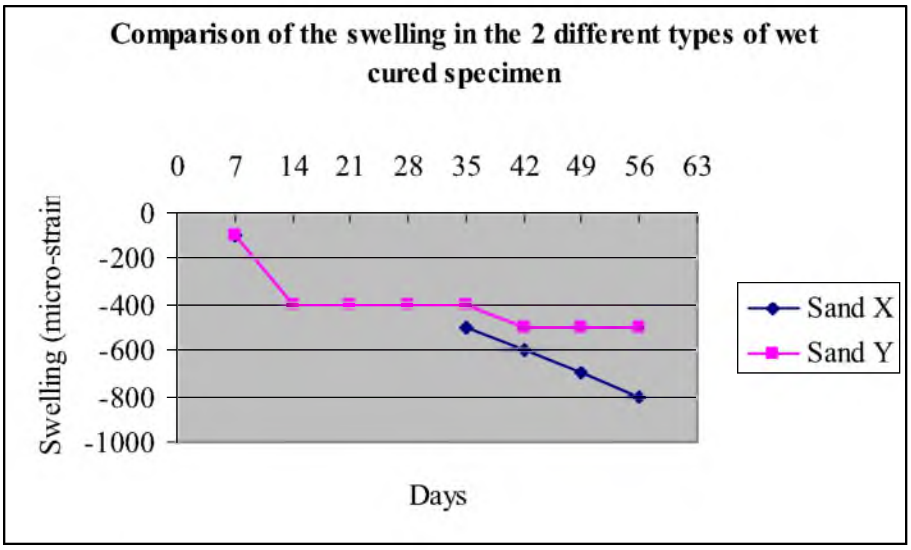
Table 6.11a: Drying Shrinkage during Curing in Sand "X" Cubes

		Total Curing Shrinkage (in μ-strains) after								
Sand "Y" Cubes	Block Number	7 days	14 days	21 days	28 days	35 days	42 days	49 days	56 days	Direction of Movement
Dry cured	3	0	100	200	400	500	600	700	700	Shrinkage
Dry cured	6	0	200	300	500	600	700	800	800	Shrinkage
Dry cured	9	0	100	300	400	500	600	700	800	Shrinkage
Wet cured	16	-100	-400	-400	-400	-400	- 500	-500	-500	Swelling

Figure 6.11b: Drying Shrinkage during Curing in Sand "Y" Cubes



Graph 6.11c: Graph showing the drying shrinkage of the dry cured specimens for the two materials.



Graph 6.11d: Graph showing the comparison of the swelling for the two different wet cured specimens.

Measurements have been converted into micro-strain using:

μ-strain = $\frac{\delta l}{l}$ x 10^6, l is in millimetres

Measurements to the nearest 10 microns

Only four dial gauges could be set up for each type of concrete, since the department did not have any others that could have been used over a long period of time. This means that there could be some error in the wet cured specimens, as there were no other results to compare them with. Some measurements for the Sand "X" cubes were missed due to the Christmas vacation, and unfortunately, this could not have been avoided.

Analysing these results shows that the dry cured concrete made with sand "X" has shrunk 56.7 % more than the concrete made with sand "Y". The wet cured specimens *unexpectedly* swelled upon curing, visibly absorbing the water that it was curing in. The concrete made with sand "X", the poor quality sand, swelled 300μ-strain more than material "Y".

Shrinkage is influenced by the aggregate, which restraints the amount of shrinkage of the cement paste that can actually be realised in the concrete. The maximum size of grading of aggregate does not influence the magnitude of shrinkage of the concrete with a given volume of aggregate and a given water-cement ratio. However, larger aggregate permits the use of a leaner mix at a constant water-cement ratio, so that larger aggregate leads to lower shrinkage. This is a likely explanation of the results, since the Sand "X" cubes consisted of a larger amount of smaller particles and more water, hence shrunk more upon dry curing than the Sand "Y" cubes.

The likely mechanism that could explain these results involves complicated soil mechanics theory, and more detailed information can be found in Mitchell, 1993, "The Fundamentals of Soil Behaviour". A basic explanation of the results is provided below.

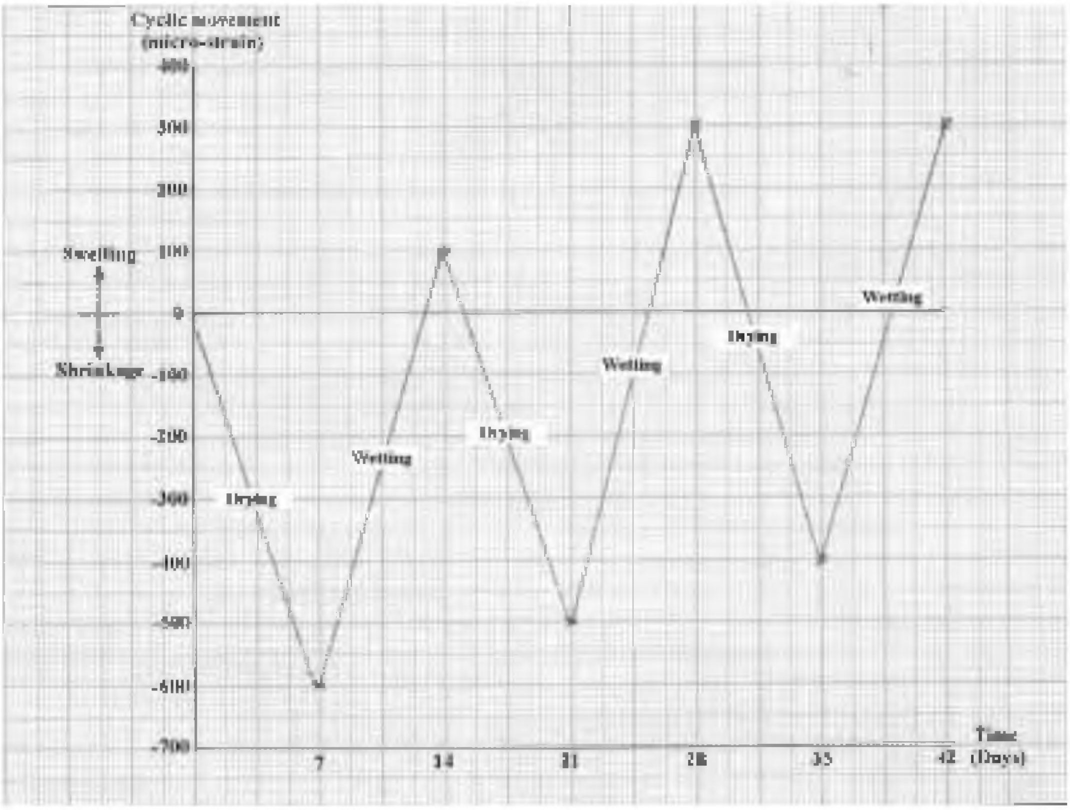
"The high clay fraction in the material being tested means there is a high proportion of small particles having surface electrostatic charges to which a layer of water becomes attached", (Spence and Cook, 1983).

As the saturated clay in the concrete is dried it shrinks, with capillary tension in the pore water balancing the pressure between the clay particles, and the friction caused by the pressure gives the clay a degree of cohesion. The difference in pressure between the pore water and the surrounding atmosphere is balanced by surface tension which can be considerable if the diameter of the pores is very small. Beyond a certain point, further drying causes no reduction in volume because the bulk of the water is removed, but the particles are held tightly together by small water droplets at their points of contact. When such a dried concrete is then wetted, water is absorbed into empty pores in the body of the clay and the capillary suctions are reduced, allowing the clay to soften rapidly as the pressure between the soil grains is reduced, and also to swell.

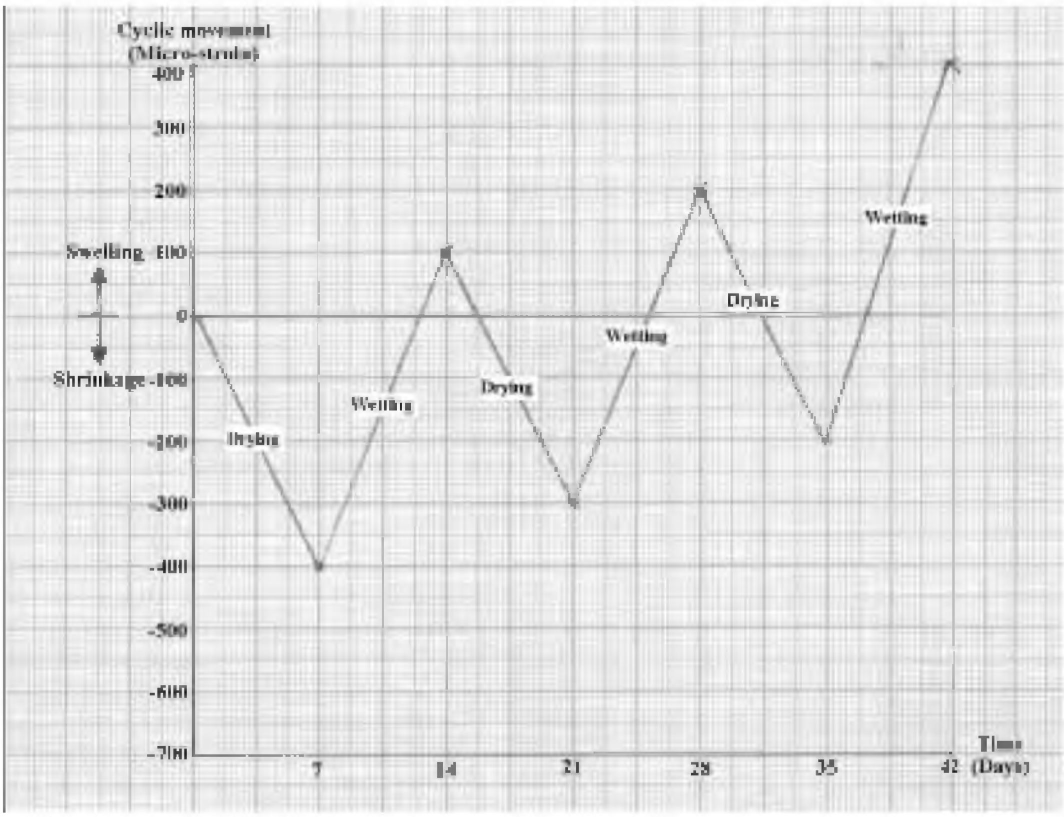
These results are not conclusive for this type of concrete since there is a considerable complication in interpreting and comparing drying shrinkage measurements. Although all the specimens tested were of the same size, the actual size of the specimen will affect the result. Water can only be lost from the surface and therefore the inner core of the specimen will act as a restraint against overall movement; the amount of restraint and hence the measured shrinkage will therefore vary with specimen size.

6.2.2 Cyclic Movements

After 56 days, the cubes were wetted for 7 days, then placed in a drying oven for 7 days, and underwent this cyclic process for three cycles. This was done on both types of concrete, and the results are shown in figures 6.12a and 6.12b.



Graph 6.12a: Cyclic shrinkage of concrete made with sand "X".



Graph 6.12b: Cyclic shrinkage of concrete made with sand "Y".

The results show that drying and wetting cycles result in more or less completely reversible shrinkage. Shrinkage after the concrete has solidified continues as and when further water evaporates. The chemical reaction of cement with water, and thus the shrinkage, continues in the concrete seemingly indefinitely. A gel is formed which contracts upon desiccation and becomes very hard. If the concrete is submerged in water the cement gel expands with considerable force, so that the whole mass of the concrete expands. This expansion, however, can never equal the shrinkage which has already taken place. On drying the concrete in air shrinkage again occurs. Therefore, when concrete is subjected to continual wetting and drying, it experiences corresponding expansions and contractions.

The concrete made from the good sand, sand "Y" has smaller cyclic movements than that of the concrete made with sand "X". Sand "X" has movements of ± 800 μ-strains, whereas sand "Y" has movements of ± 500 μ-strains. These results were expected since the extra fines in sand “X” were expected to absorb water every time the concrete was submerged in water, as well as the cement gel expanding.

7 Discussion

The high fines content in the concrete produced with sand “X” *enhanced* the dry compressive strength implying that fines in sand for fabricating concrete is not problematic. A higher strength of 12.1 MPa indicates that this type of concrete would be suitable for many applications of building in developing countries. Typically the range of concrete strengths is between 10-70MPa, so although the concrete made with sand “X” has a strength of 12.1 MPa, it is at the lower end of the range. Care should be taken when using this material for high strength applications, such as reinforced concrete columns in tall buildings.

Although this type of material seems advantageous as a building material due to its strength, shrinkage experiments produced contradictive results. ‘Normal concrete’ usually shrinks between 300-600 μ-strains in one year of drying, but the concrete made from sand “X” shrunk 1300 μ-strain in 56 days. This drying shrinkage was larger than an estimated value of 720 μ-strains, probably due to a bigger effect of the fines. This is not necessary a problem in itself since concrete in compression can withstand 2000 μ-strains. The major problem occurs when the concrete is in tension as it can only withstand 200μ-strains and material “X” expanded by 800μ-strains, four times greater than the tolerable amount.

So, when determining whether this material is suitable for applications of concrete, these shrinkage movements need to be considered. For pre-made concrete blocks, providing the drying shrinkage has already occurred, it is expected that these concrete blocks can be used. Care needs to be taken if they are expected to experience continual wetting and drying since the cyclic movement is ± 800μ-strains, and cracks are likely to occur.

Brickwork in compression is expected to withstand about 600 μ-strains and in tension 200 μ-strains. It is therefore not advisable to use this material for brickwork as the movements are too great despite the good strength value. If it is necessary to use this material "X" for bricklaying, then the wall should be built in small sections, maybe one metre per day, without expansion joints.

This material is suitable for non water-proof renders since the drying shrinkage of 1200 μ-strains can be tolerated as concrete in compression can withstand up to 2000 μ-strains. If the cracks are about one metre apart and unrelieved the rendering will look acceptable. It is not advisable to use this material as a water-proof render since when the material is in tension it will fail. The material can only withstand 200μ-strains in tension whereas the cyclic movement is $\pm$ 800μ-strains.

Shrinkage in steel reinforcement concrete can be a maximum of 600μ-strains which indicates that it would not be advisable to use this material. Large cracking would occur, causing a dangerous structure. Fortunately, this material "X" can be used for lintels since it is common practice to leave a gap in the mortar at both ends to allow it to expand and contract with cracking. Therefore, the gap can be made large enough to accommodate the cyclic movement of $\pm$ 800μ-strains in material "X".

8 Future Work

This report concentrates on two main factors only, strength and shrinkage. There are many other properties of concrete that can be specified, but the duration of this project limited the amount of factors that could be investigated. Although the large shrinkage and swelling movements of the concrete tested limits its use in the building industry, it does have a better compressive strength, so other research could be carried out on this 'low quality' concrete to identify other properties that can be improved. It is expected that the examination of 'poor quality' sand in concrete and how to improve the quality will be carried out by other third year project students in the future.

9 Conclusions

The purpose of this project was to investigate the affect fines have on the properties of concrete. Initial results showed that making concrete with sand with a ‘high fines’ content *unexpectedly* increases the strength of concrete. Material “X”, made with poor quality sand, was tested for its dry compressive strength and its optimum water-cement ratio of 0.95, mixing time of 5 minutes and vibration compaction time of one minute. This material exceeded the expectation that its seven day dry compressive strength would be approximately 9.6MPa, and was instead 12.1MPa. Since concrete strength used for building purposes lies within 10-70MPa, material “X” has a suitable dry compressive strength.

Despite the suitable dry compressive strength, it has been discovered that the use of this material will be hindered due to its large cyclic movements and drying shrinkage of approximately $\pm 800\mu$-strains and 1300μ-strains respectively. Due to the restricted sample sizes the accuracy of the results from the material testing may be limited. The purpose of testing was to get approximate values for the strength and the shrinkage, since testing under conditions in the developing world may produce different results.

This project supports the statement published in British Standard, BS 812, that sand with a ‘high fines’ content is unsuitable for making good quality concrete, but sand “X” is suitable for some applications in developing countries where operations are basic and money is short. It can be used for concrete and mortars in unconstrained applications such as block making and lintels and in situations where the material does not undergo cyclic movements, such as non water-proof rendering. If this material is used for applications such as bricklaying then the wall should be built in small sections, maybe one metre per day, without expansion joints.

There are many other properties of concrete, other than strength, drying shrinkage and cyclic movements that can be specified, but the duration of this project limited the amount of factors that could be examined. It is expected that other research in this field will be carried out to improve the quality of concrete made with ‘high fines’ sand, possibly by washing it, as the original project suggested.

10 List of References

Attwood, G. & Dyer, G., *Statistics 2*, 1st Edition, Heinemann, 1995, pp. 47.

Akroyd, T. N. W., *Concrete: Properties and Manufacture*, 1st Edition, Pergamon Press Ltd., 1962, pp. 100 & 177.

BS 12: 1978: *Ordinary and Rapid Hardening Portland Cement.*

BS 812: Part 1:1975: *Sampling, shape, size and classification.*

BS 1881: Part 3: 1970: *Methods of Making and Curing Test Specimens.*

BS 1881: Part 115: 1983: *Specification for Compression Testing Machines of Concrete.*

BS 1881: Part 116: 1983: *Method for the Determination of Compressive Strength of Concrete Cubes.*

BS 1881: Part 202: 1986: *Measurement of the Velocity of Ultrasonic Pulses in Concrete.*

BS 5497: Part1: 1979: *Guide for the Determination and Repeatability and Reproducibility for a Standard Test Method.*

Bungey, J. H. & Millard, S.G., *Testing of Concrete Structures*, 3rd Edition, Blackie Academic and Professional, 1996, pp. 49.

Elvery, R.H., *Ultrasonic Testing of Concrete – Te Use of the PUNDIT,* 1972, pp. 13.

Illston, J.M., *Construction Materials*, 2nd Edition, Chapman and Hall, 1994, pp. 155-157, 129-132.

Kesler, C. E., *Significance of Tests and Properties of Concrete and Concrete Making Materials*, ASTM STP No. 169-A, Philadelphia, 1996, pp. 1044-1052.

Lydon, F. D., *Developments in Concrete Technology*, 1st Edition, Applied Science Publishers Ltd., 1979.

Mitchell, J. K., *Fundamentals of Soil Behaviour*, 2nd Edition, John Wiley and Sons, 1993, pp. 177, 302-303.

Neville, A. M. & Brooks, J. J., *Concrete Technology*, 2nd Edition, Longman Group Ltd., 1990, pp. 74, 179.

Popovics, S., *Strength and Related Properties of Concrete: A Quantitative Approach*, 1st Edition, John Wiley and Sons, 1998, pp. 2, 221.

Spence, R. J. & Cook, D. J., *Building Materials in Developing Countries*, 1st Edition, John Wiley and Sons, 1983, pp. 36, 191-206.

Stulz, R. & Mukerji, K., *Appropriate Building Materials: A Catalogue of Potential Solutions*, 3rd Edition, Skatt Publications & IT Publications, 1988, pp. 6-8.

Teychenne, D. C, Franklin, R. E., & Erntroy, H. C., *Design of Normal Concrete Mixes*, 1st Edition, Building Research Establishment Report, 1988, pp. 1-11.

Young, J. F. & Gray, R. J., *The Science and Technology of Civil Engineering Materials*, 2nd Edition, Prentice Hall, 1998, pp. 194-197.

11 Glossary

Many terms have different meanings for different applications. The definitions used in this report are:

Aggregates: A granular material obtained by processing natural material.

Clay: It is the fraction of a soil composed of particles smaller than 0.002 mm in size.

Concrete: The essential ingredients of concrete are cement, aggregate (sand and gravel), and water. When mixed in carefully prescribed proportions, they produce a workable mass, which can take the shape of any formwork into which it is placed and allowed to harden in.

Fines: Any solid material passing a 75μm sieve.

Grading: Particle size distribution.

Sand: The fraction of soil composed of particles between the sizes of 2mm and 0.06mm, i.e. the smallest grain size than can be discerned by the naked eye.

Silt: The fraction of a soil composed of particles between the sizes of 0.06mm and 0.002mm.

Appendix A1
Letters of Correspondence

62 Kingsway,
Leamington Spa,
Warwickshire,
CV31 3LE.

Smith and Sons (Bletchington) Ltd.,
Enslow,
Kialington,
Oxon.
OX5 3AY

29.9.01

Dear Sir / Madam,

I am a penultimate year student at Warwick University studying Design Engineering. This year of study involves me undertaking a large project in which I will be developing an effective method of, or a machine for improving sand, possibly by washing it, to make it more fit for use in making concrete or mortar.

Although the design task I face is looking at a method that will be compatible for operations, finances and normal sand sources of a builder in Africa, it would be useful to find out about the current methods in the UK as research for this project. It could also be interesting to find out whether the current methods in the UK could be improved in terms of cost and efficiency. I was hoping that your company would be able to assist me in this process. I would be very grateful if I could visit your company for part of a day in the near future for a brief discussion with somebody concerning the methods that are used.

Due to this project only running over the course of one academic year, the time scale for me to develop my ideas is short. I would therefore appreciate it if you could contact me as soon as possible if you are able to help. If this is not possible, it would be very much appreciated if you could send me some names and addresses of other companies that may be able to assist me.

My email address is vicky.fernandes@vizzavi.net and my telephone number is 07833 970966.

Thank you very much for your time in reading this letter, and I hope to hear from you in the near future,

Yours Faithfully,

MISS VICKY FERNANDES
3rd Year Engineering Design Student, Warwick University.

SMITHS
BLETCHINGTON

Smith and Sons

Miss Vicky Fernandes,
62 Kingsway,
Leamington Spa.
Warwickshire,
CV31 3LE

Our Ref: rsb/gm/sitevisit001

Tuesday 2nd. October 2001

Dear Miss Fernandes,

Re: <u>Site Visit to Gill Mill Quarry, Witney.</u>
<u>Study of Sand Plant Operations.</u>

I am in receipt of your letter to our Head Office of 29th. September and it has been passed to me to see if I can assist you. This site has a 200 tonnes per hour sand and gravel washing plant and has recently installed a new sand recovery plant which is highly efficient in addition to having a sand cyclone tower for fine sand production.

However, I have considerable experience with which I may be able to advise you. I am an Engineer with HNC's in Marine and Mechanical Engineering, I am a Fellow of the Institute of Quarrying and I have spent a considerable period of my life working overseas in Africa, the Middle East and the Pacific and am very familiar with Third World conditions and materials.

I am enclosing an Indemnity Form which I would be obliged if you could fill in and return to me as soon as possible. Perhaps you would call me on 01993 779514, Fax 01993 779383, to discuss a timetable. The site address is on the top of the Indemnity Form.

I am looking forward to hearing from you.

Yours Sincerely,
Smith and Sons (Bletchington) Ltd..

Robert S. Bacon
Production Manager

Sands and Aggregates

Site Excavation and Groundworks

62 Kingsway,
Leamington Spa,
Warwickshire,
CV31 3LE.

Smith and Sons (Bletchington) Ltd,
Mr. R. Bacon,
Gill Mill Quarry,
Standlake Road,
Ducklington,
Nr. Witney,
Oxon. OX8 7PP.

17.10.01

Dear Mr. Bacon,

Thank you very much for your knowledge and time spent helping me on my research project. It really helped my understanding of the processes you use to improve the quality of your sand, and your added experiences of developing countries has given me a better idea of what type of cleaning methods I will need to develop.

Thank you again for all your help and advice, it was very much appreciated.

Yours Sincerely,

MISS VICKY FERNANDES.

Appendix A2
Sedimentation Test (Syphon)

Sedimentation Test (Syphon)

A material from Gill Mill Quarry in Witney in Oxfordshire was tested for its silt content. This was done to see whether the material contained enough silt, to represent a silt content for the sand chosen to be tested.

The test used was adapted from a similar test described in the University of Warwick's working papers. The working paper used was number 38, written by Mr. D.E. Gooding called "Soil Testing for Soil-Cement Block Preparation, 1993".

Equipment: 1 Flat bottomed glass jar, approximately 1 litre capacity.

1 Flat circular disk on a stem such that it may be lowered into the cylinder.

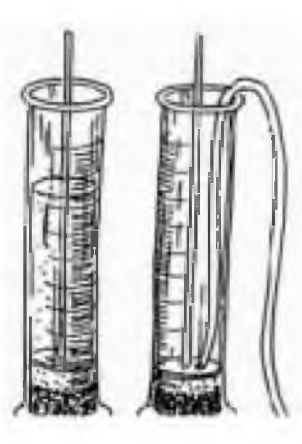
1 Flexible rubber siphon tube to remove suspended material from the cylinder.

1 Stopwatch or clock.

1 Weighing balance accurate to at least 0.1g, preferably 0.01g.

1 Heat proof container to receive the syphoned suspension.

Method: Weigh out a representative 60g sample of dry soil and place it in the cylinder. Add clean water to 200mm. Close the cylinder with a rubber bung and shake vigorously end over end to produce a uniform suspension of soil. Once a uniform suspension has been formed place the cylinder on a flat steady level surface and begin to time 20 minutes.



At the end of 20 minutes slowly lower the disk to cover the settled material, taking care not to disturb it. The top layer of material is silt. (It the disk is allowed to rest on the surface then some silt will be forced up around the edge of the disk. Any silt forced back into suspension will give a misleading low value for the silt fraction). The remaining suspended material can be siphoned off with the rubber siphon tube. The siphon operation is simpler to perform if the tube is tied to the stem just above the upper face of the disk. This stops the tube from floating or curling.

The material remaining in the jar is the silt fraction, so is put in a heat proof dish, and is dried. It is then weighed and recorded as the silt and sand fraction. These two materials are further separated by dry sieving. The material passing the 0.063mm sieve is the silt fraction.

Results: Mass = 60g

Clay fraction = (weight of container + clay) – weight of container

= (313.03 - 275.07) + (268.25 – 257.86)

= 49.12g

Sand and Silt fraction = (weight of container + silt) – weight of container

= 324.60 + 314.49

= 10.88g

After dry sieving:

Sand fraction = 4.90g

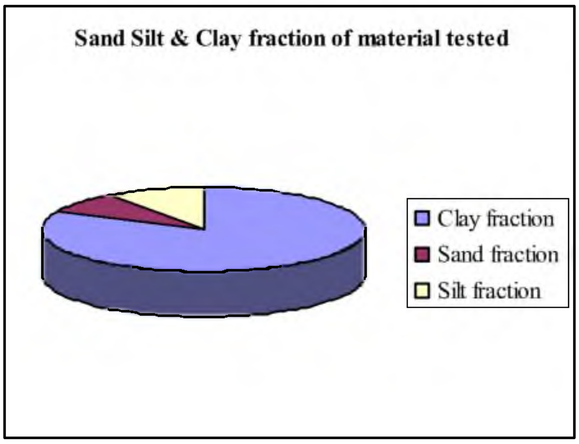
Silt fraction = 5.98g

Results as a percentage:

Clay fraction = 81.9%

Sand fraction = 8.2%

Silt fraction = 9.9%



These results showed that there was only 9.9% of silt present in the material, and hence not a good source of silt to use for my project. This material was rejected, and it was decided that Kaolin, a type of clay, would be used to represent fines, and no other silt would be added.

Appendix A3

Statistical Analysis on Water-Cement Ratios

Statistical Analysis on Water-cement Ratios

Sand "X" Mix	Maximum load (kN)								
Water-cement Ratio	Block 1	Block 2	Block 3	Mean Maximum Load (kN)	Mean Pressure, x (MPa)	n	σ	x_1-x_2	z
0.53	78.2	78.1	78.4	78.2	7.8	3	0.0152	-3.4	-200.69
0.7	112.1	112.6	112.4	112.4	11.2	3	0.0251		
0.8	113.4	113.5	113.2	113.4	11.3	3	0.0153	-0.3	-20.64
0.85	116	115.8	116.2	116.0	11.6	3	0.02		
0.9	117.8	117.5	117.4	117.6	11.8	3	0.0208	-0.1	-6
0.92	119.1	119.5	119.3	119.3	11.9	3	0.02		
0.95	122.2	120.1	121.1	121.1	12.1	3	0.105	0.4	6.53
1.06	117.5	117.3	117.2	117.3	11.7	3	0.0153		
1.14	80.9	80.5	80.7	80.7	8.1	3	0.02		

Sand "Y" Mix	Maximum load (kN)								
Water-cement Ratio	Block 1	Block 2	Block 3	Mean Maximum Load (kN)	Mean Pressure, x (MPa)	n	σ_{n-1}	x_1-x_2	z
0.53	62.1	61.1	57.1	60.1	6.0	3	0.2646	-2	-13.07
0.6	80.2	80.5	80.4	80.4	8.0	3	0.0152		
0.65	107.7	107.3	107.9	107.6	10.8	3	0.0306	-0.2	-8
0.7	110.1	109.9	110.5	110.2	11.0	3	0.0306		
0.8	108	107.8	108.1	108.0	10.8	3	0.0153	2.9	122.9
0.95	79.9	79.3	79.2	79.5	7.9	3	0.0379		

Statistical Analysis can be performed on any two sets of results. This appendix only compares results next to each other in the table.

The null hypothesis is

$H_0 : x_1 = x_2$ $\qquad$ $H_1 : x_1 \neq x_2$

$$Z = \frac{\overline{X}_1 - \overline{X}_2}{\sqrt{\left(\frac{\sigma_1^2}{n_1} + \frac{\sigma_2^2}{n_2}\right)}}$$

The 5% (two tailed) critical values for z are $Z_{95} = \pm 1.96$.

It can be seen that all results are statistically significant since they are larger than 1.96, so H_0 can be rejected. It can be concluded that there is significant evidence of a difference in means between each set of results.

Appendix A4

PUNDIT Test Readings

Appendix A5

Cube Strength versus Pulse Velocity

Cube Strength (MPa) versus Pulse Velocity (km/s)

Type of concrete	Cube Strength MPa		Pulse Velocity km/s	
	Range	Mean	Range	Mean
Sand “X” Cubes	12.01-12.22	12.12	2.77-2.88	2.83
	17.52-17.6	17.07	3.17-3.27	3.22
Sand “Y” Cubes	10.99-11.05	11.02	2.69-2.74	2.72
	14.92-15.04	14.92	3.00-3.09	3.05

COUNTERING SHRINKAGE CRACKING IN RENDERS

S. MOEED

23RD APRIL 2002

3rd Year Undergraduate Project, 2001-2002,
School of Engineering, University of Warwick

Summary

An investigation was performed into ways of reducing shrinkage cracking in cementitious renders used to line rainwater harvesting tanks. Crack reduction was measured via both leakage rate through the renders and direct measurement of the cracks propagated. Emphasis was placed on crack distribution and how this affected leakage rate. Methods of reinforcing mortar were used, the most successful being wire mesh reinforcement which reduced the leakage rate by a factor of ten. Mesh reinforcement was also the most successful in reducing shrinkage. Other renders tested included fibre reinforcement and an expansive additive to compensate for shrinkage. This investigation was a refinement of previous work carried out by Tom Constantine in 2001 but looking into different methods of waterproofing renders.

Contents

1. Introduction

1.1. Rainwater Harvesting

River flows together with the annual turnover of groundwater account for less than 40% of the rain and snow, which falls on the world's land surfaces[1]. For people in many developing countries rainwater harvesting is a viable and relatively inexpensive option to overcome water shortages. In many countries there is the problem of irregular rainfall throughout the year with heavy precipitation over certain periods but drought at other times. Therefore a system of collection and storage needs to be implemented, such as collection of roofwater, which is channelled through drainage pipes into a tank (see figure 1 below).

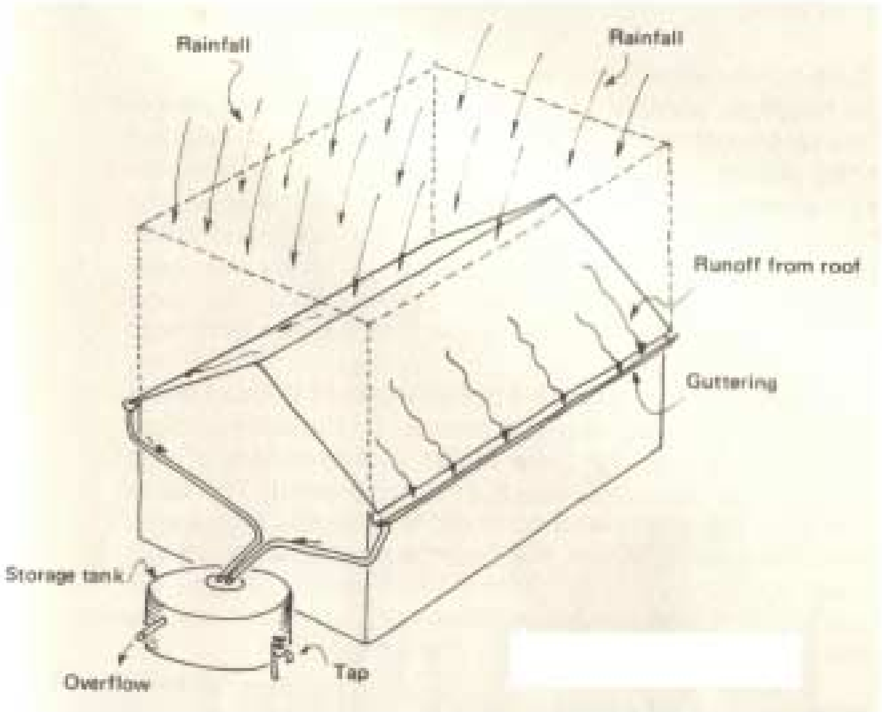


Figure 1: Arrangement of roof catchment tank (from Watt 1978)

This project deals with the render used to line the tanks to ensure they are waterproof. Tanks of up to 15m³ can be made from either bricks, rammed earth, or be dug in sitú. Earth and bricks are permeable and therefore a mortar is applied to the inside of the tank as a render. As this mortar dries it tends to shrink and the lining will therefore crack as the mortar is constrained on the tank walls. This cracking leads to leakage in

[1] Pacey pg.1

the tank, which is obviously undesirable. The leakage of rainwater harvesting tanks can be reduced if measures are taken to reduce the cracking in renders. It is important that the tanks are cheap to produce and that any materials used are available locally.

1.2. Methods to reduce cracking and leakage

A number of different methods can be used to counteract shrinkage cracking in mortar or just reduce the leakage rate through the mortar:

1. Crack distribution (by the use of a reinforcement such as chicken mesh or fibres). So that a few large cracks are replaced by many small ones.
2. The use of a non-shrinking mortar or a substance, which swells to counter the shrinkage, for example ettringite cement.
3. Filling the cracks once formed or using a waterproofing paint on the surface once the cracks have formed.
4. Applying the mortar in two layers possibly with a cement-water wash in between.
5. The use of super-plasticisers, which reduce the amount of water required for workability and hence the shrinkage during curing.
6. The use of other chemical admixtures such as strength increasing admixtures or waterproofing admixtures.

For the purposes of this study not all of these methods could be tested, as there were a limited number of test rigs and a limited amount of time. Previous projects had looked into the use of admixtures but there had been less research into reinforcing the mortar with mesh or fibres. It was decided to investigate the effects of crack distribution on the leakage rate, this was therefore the main focus of the study. It was also decided to look at applying two layers of mortar and the effectiveness of a swelling agent to counteract shrinkage.

1.3. Previous studies

Tom Constantine conducted research into this topic in the academic year 2000 – 2001. He found there was a problem with obtaining accurate results, particularly with the leakage tests in the second study. This was therefore the initial focus in the project as it is necessary to have a method of testing which is reliable and therefore gives credible results on which a conclusion can be reached.

1.4. Project Objectives

- To measure leakage flow through renders in order to recommend methods of effective waterproofing for water tank construction.
- To improve on the experimental method used in the previous year to yield more credible results.

2. Background Theory

2.1. Tank Construction

Tanks can be made of brick, rammed earth or built in sitú. The Development Technology Unit (DTU) at Warwick University is involved in the research, design and building of roofwater harvesting tanks. Probably the strongest of tanks would be those dug in sitú as the surrounding earth provides a strong tank wall and good base for the render.

2.2. Ferro-cement tanks

Water tanks made from wire-reinforced cement-mortar can be used as an alternative to earth or brick tanks. They are built by hand trowelling mortar onto a mesh of wire reinforcement, which forms the walls and shapes the tank (see figure 3 below). This forms cylindrical tanks with thin walls, which vary in thickness from 3 to 10cm depending on the size of the tank[2].

[2] Watt pg.11

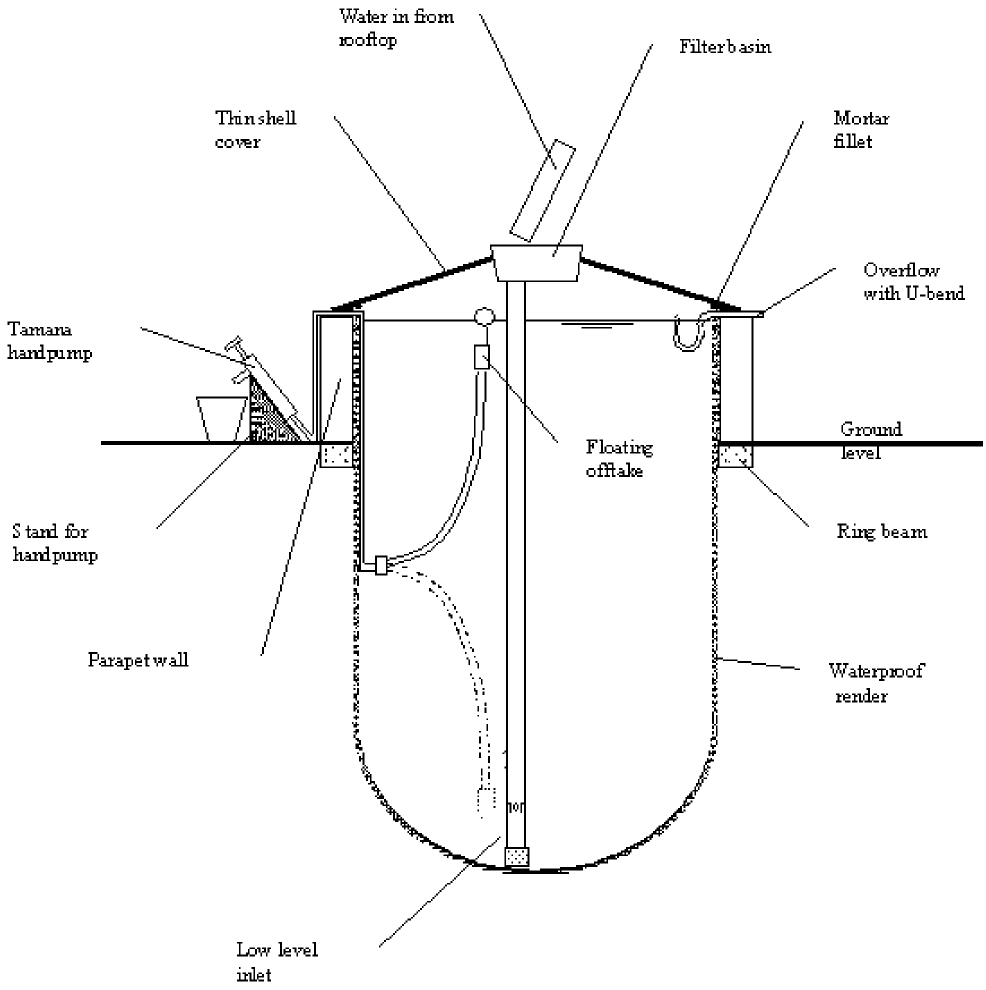


Figure 2: Diagram of partially below ground rainwater harvesting tank (DTU Website)

2.3. Mortar

Mortar is made from mixing building sand cement and water in certain proportions depending on the application. The material used for lining RWH tanks is a cement rich mortar, with a cement to sand ratio of 1:3, for this application it is beneficial to keep the water content down to a minimum as this improves the quality of the mortar once cured. The cement used is Ordinary Portland Cement.

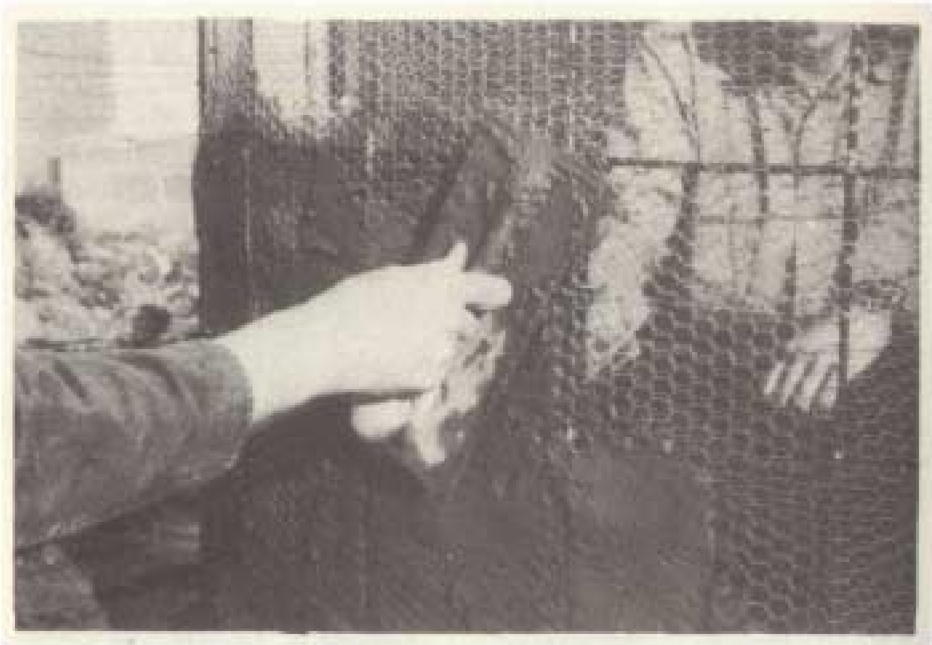


Figure 3: Construction of ferrocement tank wall (from Watt 1978)

2.4. Hydration of cement

The main compounds of Portland cement are Tricalcium silicate, Dicalcium silicate, Tricalcium aluminate and Tetracalcium aluminoferrite. In the presence of water, the silicate and aluminates listed above form products of hydration, which in time produce a firm and hard mass – the hydrated cement paste. Figure 4 below details hydration product development over time for ordinary Portland cement.

When cement is mixed with water, for an initial period the consistency of the cement-water paste remains relatively constant. Initial set occurs between two and four hours after mixing at normal temperatures, at this point the mix begins to harden at a much faster rate. Strength gain does not start until after the final set which occurs some hours later. The rate of strength gain is rapid for the next one or two days, and continues, but at a steadily decreasing rate, for at least a few a months. In order to increase the strength of cement or mortar, there needs to be ample water supplied during hardening in order to maintain the ongoing hydration reactions. For this reason mortar is cured in a humid environment. Curing is discussed in greater detail in section 3.3.1.

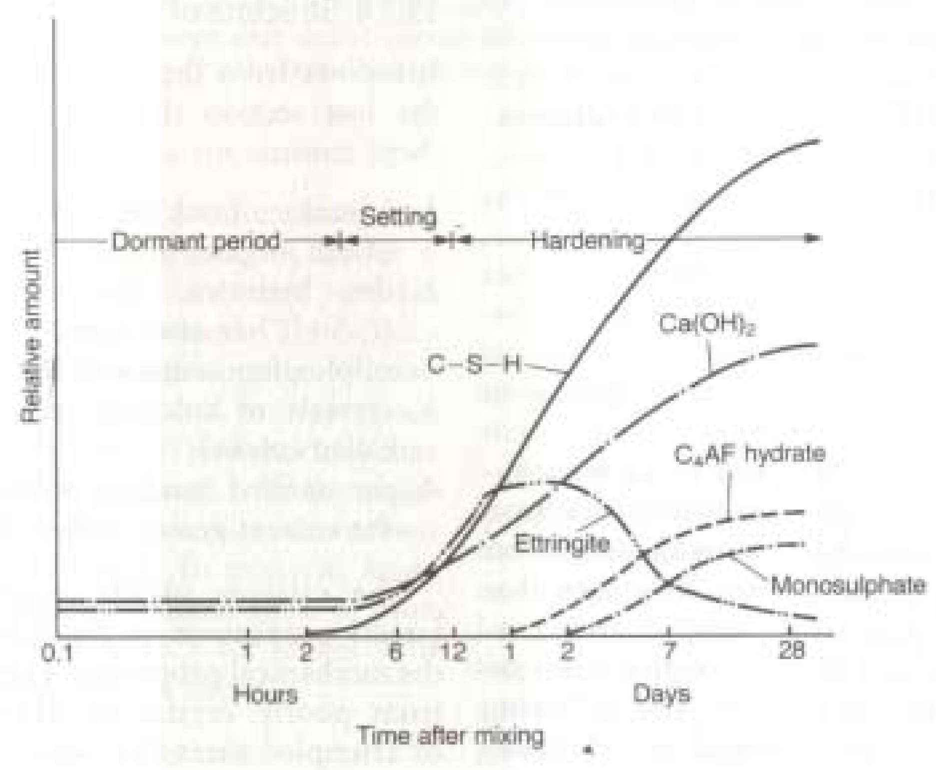


Figure 4: Typical Hydration product development in Portland cement paste (Illston 1994)

3. Literature Review

3.1. Previous studies

Constantine T, 2001

In 2001 Tom Constantine, a Warwick student, conducted an investigation into the shrinkage and cracking of mortar. Constantine experimented with various admixtures for concrete :–

- Silica fume, which reduces the porosity of concrete and therefore increases strength, which can lead to a reduction in cracking.
- Superplasticiser, which increases the workability of concrete therefore requiring less water, which also leads to less porosity as well as a reduction in shrinkage cracking.
- Harilal leak seal, an Indian admixture for waterproofing concrete.

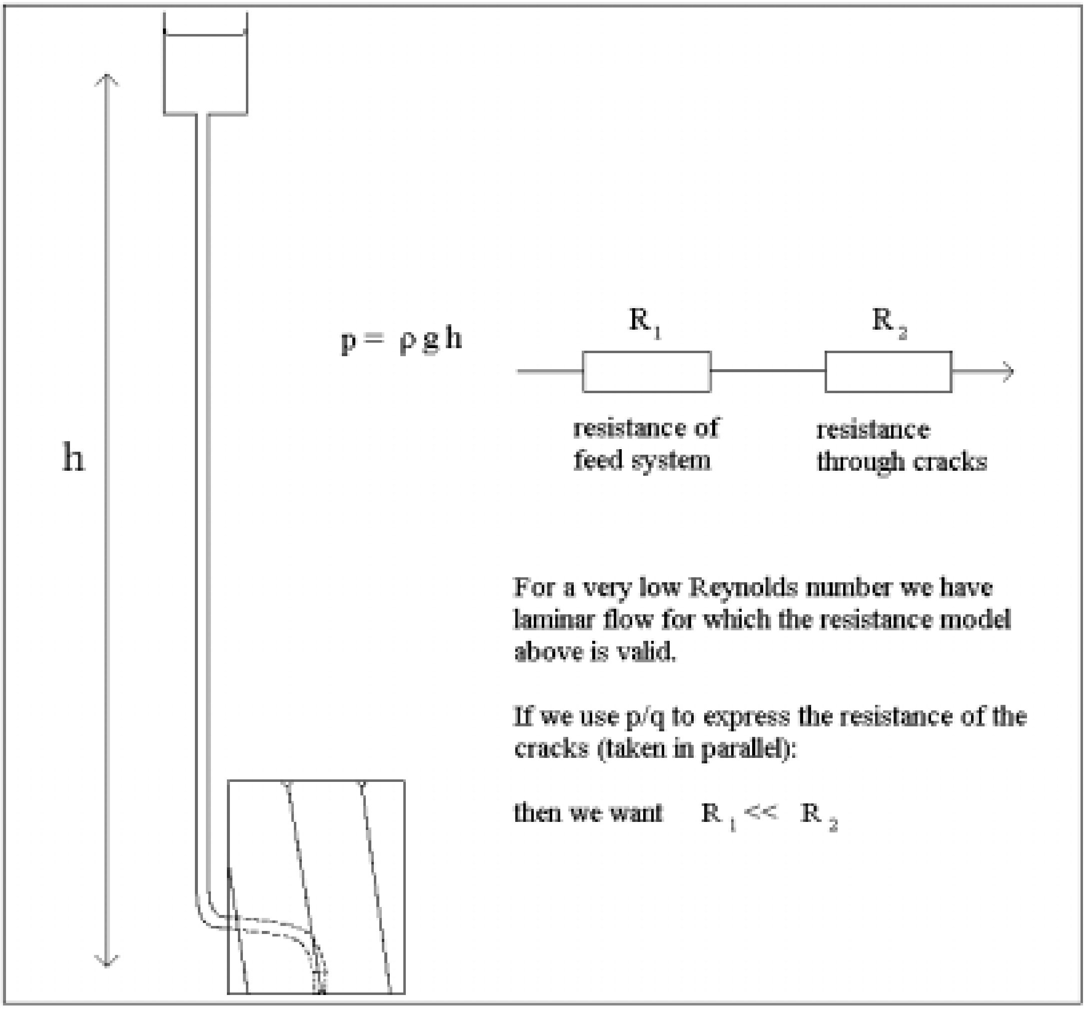
He also tested a sample of plain mortar. In the leakage tests, over a period of 55 minutes the total water lost was as follows:

Sample	Water lost in 55 minutes (ml)
Plain mortar	77.6
Mortar containing Silica Fume	39.5
Mortar containing Harilal Leak Seal	40.7
Mortar containing Superplasticiser	32.5

Table 1: Summary of Constantine's leakage test results with respect to total water loss

He found the mix with Superplasticiser to give the lowest leakage rates.

However, as can be seen from the results above, the rate of water loss is very small. Tom experienced problems with his test rigs and due to this low leakage rate it was believed that the resistance in the rig was too great (see figure 5 below), and therefore there was no guarantee that all the cracks were being fed.



3.2. Shrinkage and cracking of concrete

Neville 1995:

When water moves out of a porous body, which is not fully rigid – contraction takes place. During hydration, while the cement paste is plastic, it undergoes a volumetric contraction (of the order of 1% of the absolute volume of dry cement). However the extent of hydration prior to setting is small and as the hydrating cement becomes rigid, contraction induced by loss of water is restrained.

Water can also be lost by evaporation from the surface of the concrete while it is still plastic. Similar loss can occur by suction by the underlying dry concrete or soil. This is called Plastic Shrinkage as it occurs while the concrete is still in a plastic state.

Magnitude of plastic shrinkage is affected by amount of water lost from the surface, which is influenced by temperature, ambient relative humidity and wind velocity. The rate of water lost does not necessarily predict shrinkage, a lot depends on the rigidity of the mix.

Water can be bought to the surface of concrete by bleeding. Water in the mix tends to rise to the surface of freshly placed concrete, this is caused by the inability of the solid constituents of the mix to hold all of the mixing water (water having the lowest specific gravity). If the amount of water lost per unit area by evaporation exceeds the amount bought to surface by bleeding, surface cracking can occur, this is called plastic shrinkage cracking.

3.2.1. Plastic shrinkage

Plastic shrinkage is greater the greater the cement content of the mix. Retardation of setting allows more bleeding and leads to increased plastic shrinkage, on the other hand greater bleeding capacity prevents too rapid a drying out of the surface of the concrete reducing plastic shrinkage cracking. In practice it is cracking that matters, however, in this case, due to the mortar being constrained, cracking occurs due to shrinkage.

3.2.2. Thermal cracking

This occurs only in large volumes of unreinforced concrete. The heat of hydration (hydration is an exothermic reaction) causes expansion. Cooling from the temperature peak of this reaction results in cracking, due to a temperature gradient and internal stresses. This form of cracking is not applicable to the mortar lining situation as the layer of mortar is too thin for a substantial temperature gradient to form.

3.2.3. Carbonation shrinkage

This occurs due to the reaction of Carbon Dioxide present in the atmosphere with the hydrated cement. This is not a major concern in this application as drying shrinkage leads to the vast majority of shrinkage and cracking in the constrained mortar.

3.2.4. Concreting in hot weather

The deformation of mortar also depends on the surrounding environment. In practice the tanks will be built in hot conditions and therefore the effect of heat needs to be taken into consideration. Plastic shrinkage can be prevented by keeping down the rate of evaporation of water from the surface of the concrete, $1kg/m^2$ per hour should not be exceeded. Evaporation is increased when the temperature of the concrete is much higher than ambient temperature, then plastic shrinkage can occur even if the relative humidity of the air is high. It is therefore best to protect concrete from the sun and wind, place and finish fast and start curing quickly. Avoid placing concrete on a dry subgrade. Another type of cracking is caused by differential settlement of fresh concrete due to obstruction to settlement e.g. large particles of aggregate or reinforcing bars, this is plastic settlement cracking.

3.2.5. Drying shrinkage

As the name implies, this shrinkage occurs as water is lost from the cement paste.

Illston 1994 :

Hydrated cement paste has a considerable affinity for water and therefore its overall dimensions are water sensitive i.e. loss of water results in shrinkage. The water content also has an effect on porosity, to illustrate this it is useful to look at the way in which water behaves in the paste (see fig.6 below).

- *Water vapour* – largest voids may only be partially filled with water, and remaining space will contain water vapour.
- *Capillary water* – located at the capillary and larger gel pores, it is the bulk of water not influenced by the attractive forces of solid surfaces. Water in voids larger than 50nm is considered free in that it’s removal does not lead to any overall volume change. However, water in pores smaller than about 50nm is subject to capillary tension forces, and its removal may lead to some shrinkage.

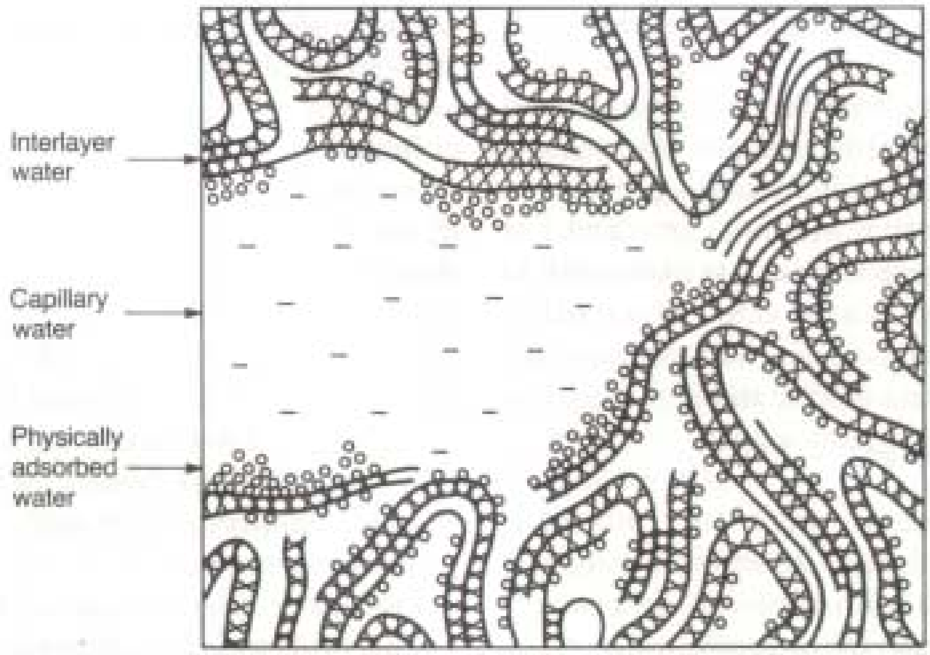


Figure 6: schematic of types of water within hydrated cement paste (from Illston 1994)

- *Adsorbed water* – Close to solid surfaces and under the influence of surface attractive forces. A large proportion of this water can be lost on drying to 30% relative humidity and this loss is the main contributing factor to drying shrinkage.
- *Interlayer water* – This is the water in gel pores narrower than about 2.6nm, this is under the influence of two surfaces and therefore more strongly held. It can only be removed by strong drying – i.e. elevated temperatures and/or relative humidities less than 10%. Its loss results in considerable shrinkage as strong forces pull the solid surfaces closer together.
- *Chemically combined water* – Water that is combined with the fresh cement in hydration reactions. This is not lost on drying.

Figure 7 below illustrates the relationship between water/cement ratio and shrinkage.

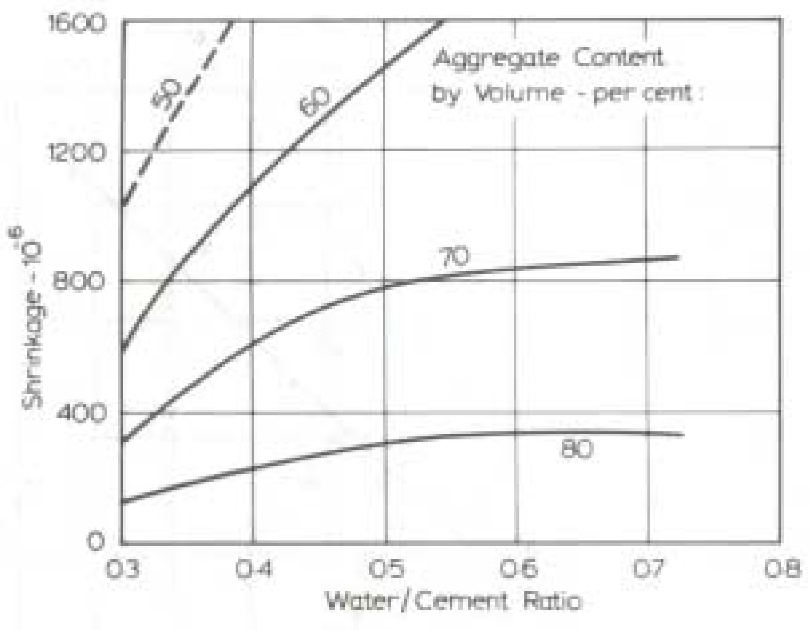


Figure 7: Influence of water/cement ratio and aggregate content on shrinkage (from Neville 1995)

The shrinkage of cement or mortar is larger the higher the water/cement ratio as this determines the amount of evaporable water in the cement paste and the rate at which it can move to the surface. The amount of shrinkage is not equal to the volume of water removed as it is also influenced by other factors such as gel particle size. The affect of aggregate leads to less shrinkage in concrete in comparison with mortar, which does not contain aggregate. Also the type of water removed has an affect as discussed earlier, emptying of the capillaries results in water loss with no shrinkage, but once capillary water is lost, the removal of adsorbed water takes place and causes shrinkage. When no moisture movement to or from the cement paste is permitted, shrinkage occurs due to withdrawal of water from the capillary pores by the hydration of the hitherto unhydrated cement called self desiccation. This shrinkage is restrained by the rigid skeleton of the already hydrated cement paste and also by the aggregate particles.

From a graph of early shrinkage (Figure 1.1., Appendix 1), it can be seen that after 24 hours mortar has shrunk by 0.0045 of its original volume i.e. 0.45%. At this point the graph becomes level, it can therefore be concluded that the majority of shrinkage occurs within the first 24 hours and so the figure of 0.0045 has been taken as the value for shrinkage strain for the plain mortar used in testing.

3.3. Preventing shrinkage and cracking

For a sample of constrained mortar, i.e. in this application mortar is held by the tank walls, improving the mortar strength can reduce cracking. Increasing tensile strength will increase the shrinkage strain that can be tolerated before cracking occurs:

as $\varepsilon_{yield} = \sigma_{yield} / E$

3.3.1. Factors affecting mortar strength

3.3.1.1. Water Content

Illston 1994:

The strength of cement paste is governed by its porosity, which depends on the water cement ratio and degree of hydration. The higher the water content the greater the porosity and volume of voids, which leads to a weak mortar. Figure 8 below shows the effect of age and water content on the strength of the mortar.

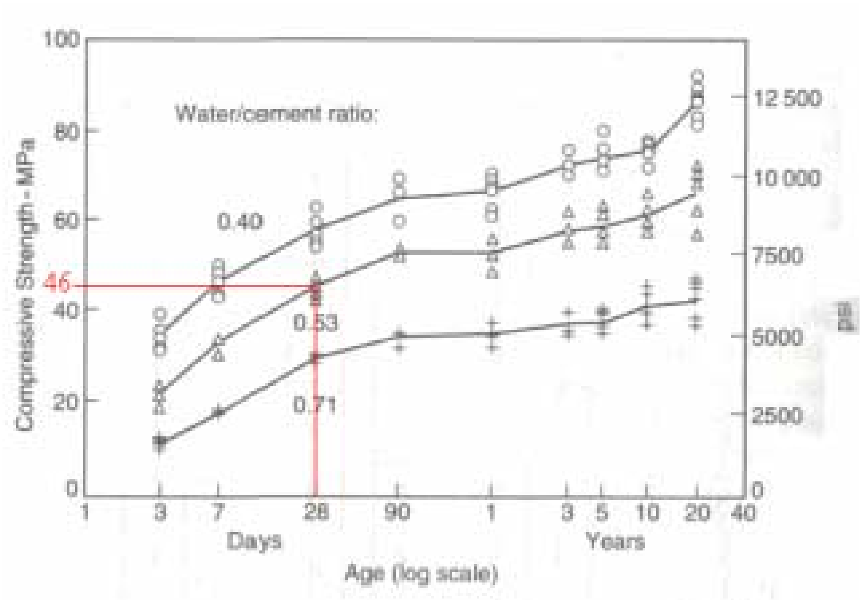


Figure 8: Effect of age and water/cement ratio on concrete strength (Neville 1995)

However, too low a water content will reduce the workability of the mortar and therefore a compromise between strength and workability is required.

3.3.1.2. Effect of age

The degree of hydration increases with age. This leads to an effect on the strength as can be seen in figure 8 above. Hydration reactions are never complete, and in the presence of moisture, concrete will continue to gain strength for many years. The rate of increase, however, will be very small in such situations.

The water/cement ratio used for the mix was 0.5, and each sample was tested once it was 28 days old. The graph in figure 8 indicates a figure of around 46Mpa for the compressive strength of the samples, (in SI units 46×10^{-6} N/m^2). According to British Code of Practice BS 8007:1987, the relationship between the compressive and tensile strength of concrete can be defined by the formula:

$\boldsymbol{f_t = 0.12\,(f_c)^{0.7}}$ (eq. 1)

Where f_t is the tensile strength and f_c is the compressive strength[3].

However, this usually gives an underestimate of tensile strength, presumably to ensure a safety margin in concrete and mortar used for building. A graph showing the relationship between compressive and tensile strength can be seen in figure 1.2. appendix 1, from this graph the best fit overall is given by the expression:

$\boldsymbol{f_t = 0.3(f_c)^{2/3}}$ (eq. 2)[4]

Using this formula with a value of 46 Mpa for compressive strength taken from the graph in figure 8 the tensile strength of the plain mortar samples can be calculated to be 3.85 Mpa (3.85×10^{-6} N/m^2). Values for tensile strength will be used in section 5 in order to calculate an expected relationship between crack width and number of cracks for the purposes of comparing the theoretical predictions with actual results.

3.3.1.3. Effect of curing

Neville 1995 and Illston 1994:

The object of curing is to keep concrete saturated until the originally filled water space in the fresh cement paste has been filled to the desired extent by the products of hydration of cement. Hydration is greatly reduced when the relative humidity in the capillary pores drops below 80%[5]. It therefore follows that for hydration to continue, the relative humidity inside the concrete has to be maintained at a minimum of 80%. Once the concrete or mortar surface is no longer liable to damage, curing can take place. This can be done by covering the surface with wet material, submerging it in

[3] Neville pg.310
[4] Neville pg.310

water completely or leaving it in a sealed environment to reduce evaporation and maintain high humidity in the air surrounding the specimen. Some form of curing i.e. preventing water loss from the concrete surface is not only important to maintain hydration and therefore strength gain, water loss also leads to plastic shrinkage and increased permeability (due to increased porosity) – which is a problem when the material is required for waterproofing. Figure 9 below illustrates the influence of curing on concrete strength.

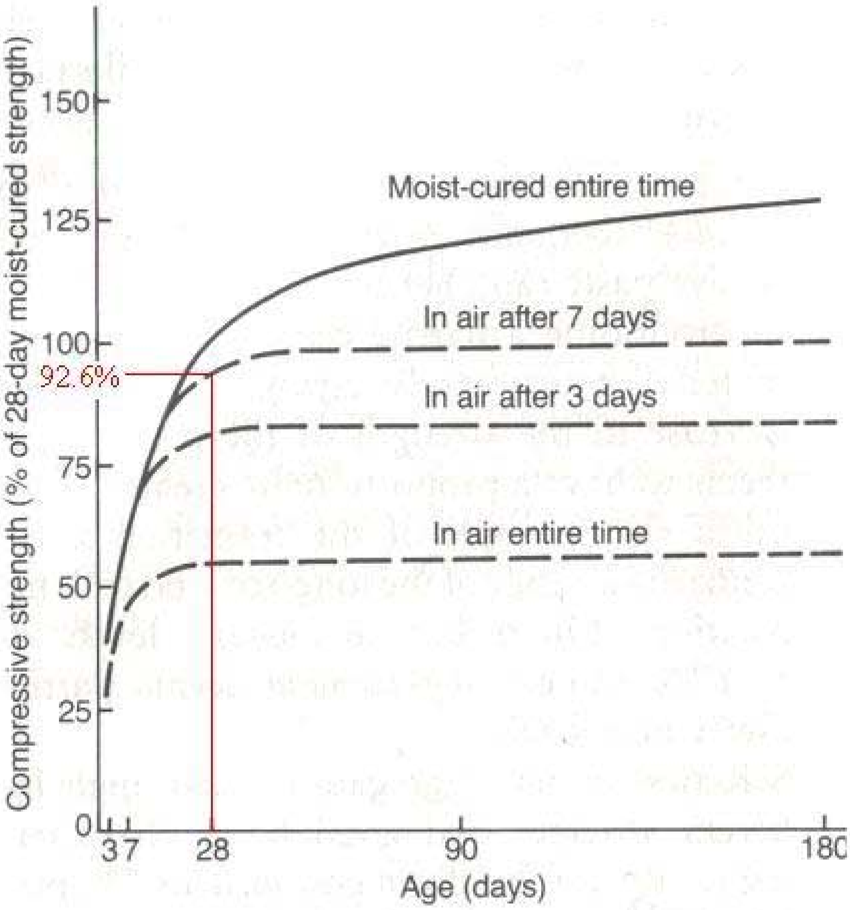


Figure 9: Influence of curing conditions on concrete strength (From Illston 1994)

The data from this graph can be used to make the figures for tensile and compressive strength attained in section 3.3.1.2. more accurate. The graph in figure 8 shows data for samples, which have been moist, cured the entire time. Due to time restrictions the samples used were only cured for seven days and then left to dry in air for another 21

---

[5] Neville pg.318

days before testing was carried out. From the graph above it can be seen that the actual compressive strength is therefore 92.6% of the previous figure, giving a new value of $42.6x10^{-6}$ N/m^2. Using equation 2 defined in section 3.3.1.2. the tensile strength becomes $3.66x10^{-6}$ N/m^2.

3.3.2. Admixtures

Rixom 1978:

These are chemicals that are added to the concrete immediately before or during mixing. They significantly alter its fresh early age or hardened state, to advantage or gain in properties. There are different types of admixtures:

- Water reducing
- Accelerating
- Air entraining
- Retarding
- Superplasticising
- Water-proofing

For the purposes of this report only those that may help reduce shrinkage cracking or leakage are discussed.

- Waterproofing – concrete absorbs water by capillary action. These admixtures aim to prevent penetration of water into concrete, for example vegetable or animal fats.
- Water reducing – in order to reduce the water: cement ratio whilst retaining workability. This will enable the use of a low amount of water thus reducing drying shrinkage and increasing strength. The most effective of water reducing agents are superplasticisers:
- Superplasticisers – Long molecules wrap themselves around cement particles giving them a negative charge so they repel each other, resulting in a dispersion of cement particles improving workability. These can also be used to produce concrete of normal workability but high strength due to reduction in water/cement

ratio. Superplasticisers can reduce water content for a given workability by 25 to 35%.

If the initial water/cement ratio is 0.5, a reduction in water content by 30% would mean a new ratio of 0.35. This corresponds to a compressive strength of 61.3 MPa, (see graph fig.1.3 appendix 1), if 92.6% of this value is used (due to each sample being cured for 7 days) the compressive strength is 56.8 MPa. Using equation 2, as defined in section 3.3.1.2. the tensile strength can be calculated to be 4.43 MPa ($4.43x10^{-6}$ N/m^3).

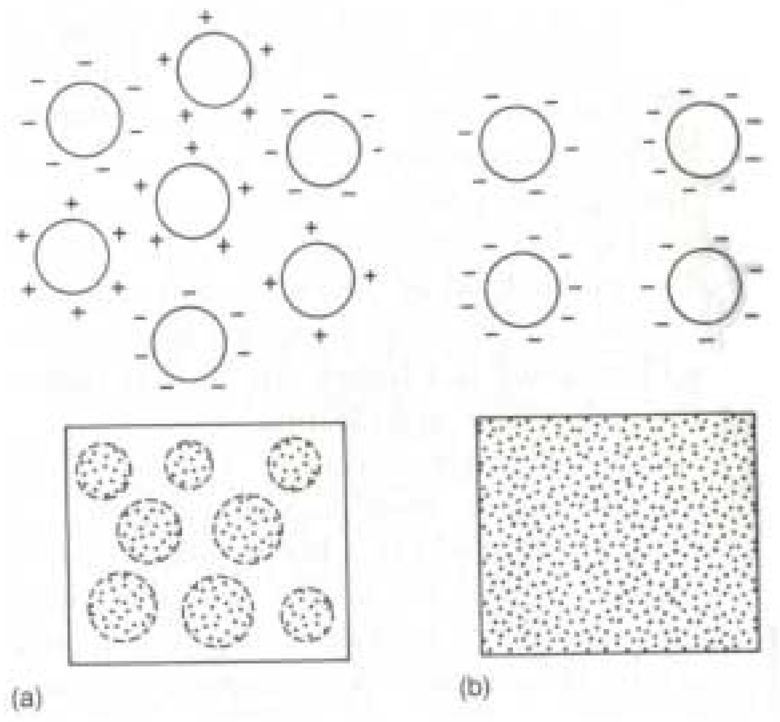


Figure 10: Dispersing action of plasticising admixtures (From Illston 1994)

(a) flocculated particles
(b) dispersed particles after admixture addition

There are also strength increasing admixtures which can have the effect of reducing cracks by producing a mortar with greater tensile strength so there is greater opposition to crack propagation.

3.3.3. Expansive cements

Odler 2000:

Expansive cements are inorganic binders that generate expansive (compressive) stresses in the hardened paste in the course of hydration, counteracting the tensile stresses generated by chemical shrinkage and drying shrinkage. Thus the generation of these expansive stresses may prevent the formation of cracks in concrete in the course of drying. Cements that meet these requirements are called *shrinkage-compensated cements*. The setting and hardening properties of concrete mixes made with shrinkage-compensated cements differ little from those made with ordinary cements, but the impermeability of the hardened concrete increases significantly

3.3.3.1. Expansive cements based on ettringite formation

Odler 2000 and Illston 1995

During hydration reactions, prior to setting, ettringite is regularly formed in ordinary Portland cement. It is the result of the reaction of gypsum with C_3A to form calcium sulphoaluminate (ettringite):

$$C_3A + 3CSH_2 + 26H \rightarrow C_3A \,.\, 3CS \,.\, H_{32}\,.$$

However, this formation of ettringite does not cause expansion as it will crystallise out, this can be seen in figure 4 section 2.4., which details the products of hydration over time. The formation of ettringite will lead to shrinkage compensation if the ettringite is formed after setting, at a stage when the paste has already attained certain rigidity. There are shrinkage-compensated cements available, which will reduce shrinkage in concrete through ettringite formation.

3.3.4. Reinforcing mortar

The reinforcement of mortar aids in increasing mortar strength, as the material used for reinforcement has a greater tensile strength than the mortar itself. Here two types of reinforcement are discussed:

- Fibre reinforcement
- Wire mesh reinforcement

3.3.4.1. Fibre reinforcement

Bentur 1990:

Fibre reinforcement is used in cementitious materials in:

a) Thin sheet materials, in which conventional reinforcing bars cannot be used. In these applications fibres act to increase strength and toughness.

b) Components, which must withstand locally high loads.

c) Components in which fibres are added to control cracking induced by humidity and temperature variations.

In application (b) and (c), the main role of the fibres is to control the cracking of the composite. The effect of fibres in fibre reinforced concrete is illustrated in the schematic (figure 11) below. The fibres improve the 'ductility' of the material, i.e. its energy absorption capacity.

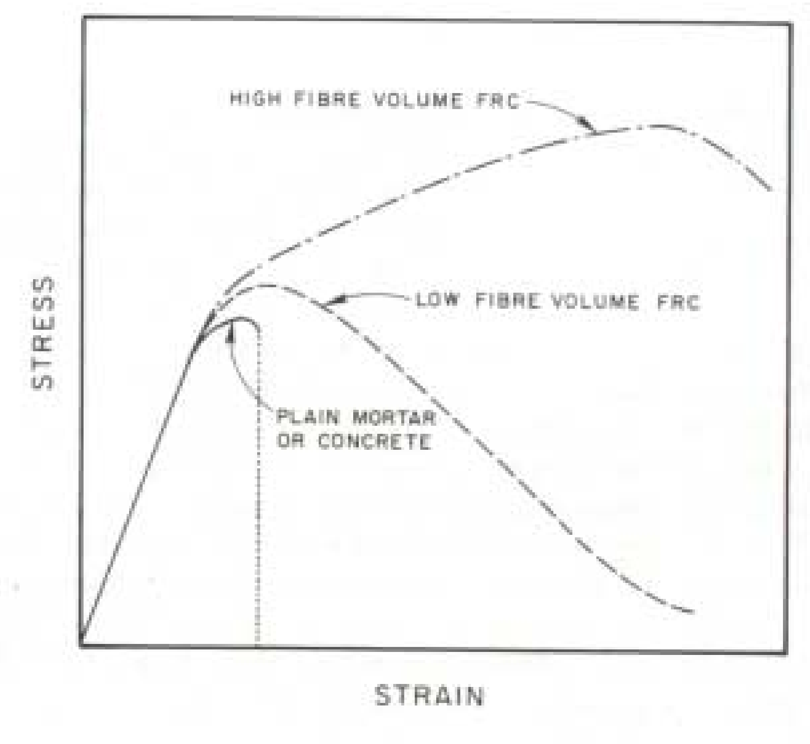


Figure 11: Typical stress-strain curves for low fibre volume and high fibre volume FRC
(From Bentur 1990)

3.3.4.2. Mesh reinforcement

Watt 1978:

The weakness of mortar in tension occurs due to planes of weakness between the edges of discrete lumps that make up the mortar. These are exaggerated by shrinkage during curing and the imperfect bond between each layer of mortar that is trowelled on. In compression these planes of weakness are held together by the load, but under tensile loading they will open up beyond their elastic limit, and cause the mortar to fail. In reinforced mortar, the mortar is assumed to contribute greatly to the tensile strength of the composite layer. This is due to the wire mesh, distributed relatively densely through the mortar, will allow the load to be taken throughout the complete layer and will prevent the early concentration of critical stresses in planes of weakness. Any cracks that do form under moderate loading will not be wide enough to allow water to reach the reinforcing wires and start corrosion.

According to Watt (1978), the maximum tensile stress in a thin walled cylindrical tank, constructed using wire mesh reinforced mortar, is 1.26 Mpa. This figure is given for a tank of wall thickness 0.03m, it can therefore only be used as an approximation as the tank lining of interest should have a thickness of 0.01m.

Reinforcement has the benefit of not only increasing the strength of the mortar but can also enable better crack distribution and the mathematical analysis in section 4 shows that this can lead to a reduced leakage rate.

4. Crack Distribution theory

Flow through a crack can be modelled as laminar flow driven by a pressure gradient between two infinitely wide plates (Plane Poiseuille Flow). This is assuming that the crack length is much greater than the crack width. Figure 12 below shows flow between plates a distance 2R apart, a fluid element of width 2y has been highlighted for analysis.

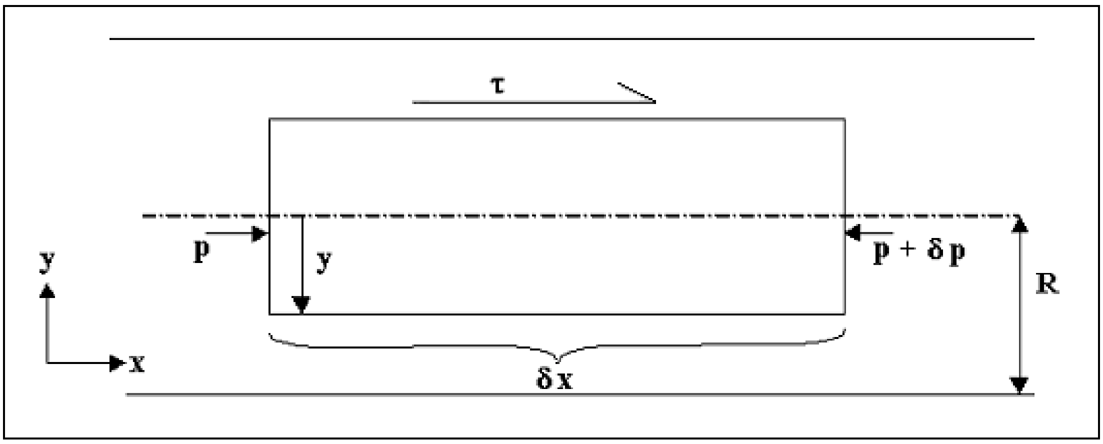


Figure 12: model for crack of width 2R, mortar thickness w (where w is parallel to x axis) and infinite length

y : distance from axis
p : pressure
R : ½ crack width

$$\tau = -\mu \frac{du}{dy}$$

τ : viscous shear stress for a Newtonian fluid

u : velocity parallel to axis shown

μ : viscosity of fluid

Force balance for a unit length into the paper:

$$p2y = -2\tau\delta x + (p + \delta p)2y$$
$$2\tau\delta x = (p + \delta p)2y - p2y$$
$$\tau = \left(\frac{\delta p}{\delta x}\right)y$$

where:

$$\frac{dp}{dx} = \frac{\Delta p}{w}$$

where Δp is pressure drop across a crack through mortar thickness of w

$$\tau = -\mu \frac{du}{dy} = -\frac{\Delta p}{w} \cdot y$$
$$\frac{du}{dy} = -\frac{\tau}{\mu} = -\frac{\Delta p}{w} \cdot y$$

Integrate to get velocity profile:

$$\int du = -\int \frac{1}{u} \cdot \frac{\Delta p}{w} \cdot ydy$$

$$u + c = -\frac{\Delta p}{2\mu w} y^2$$

at

$$u = 0, y = \pm R$$

$$\therefore c = -\frac{\Delta p}{2\mu w} y^2$$

$$u = \frac{\Delta p}{2\mu w}(R^2 - y^2)$$

Volumetric flow rate:

By symmetry:

$$Q = 2\int_0^R u(y)dy$$

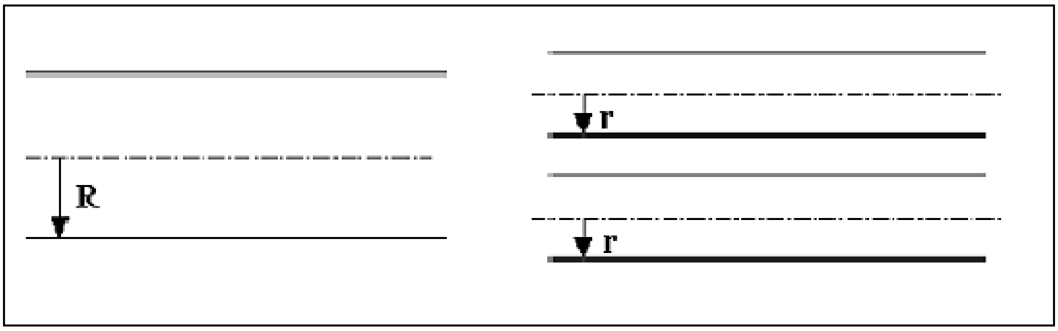
$$Q = \frac{\Delta p}{\mu w}\int_0^R (R^2 - y^2)dy$$

$$Q = \frac{\Delta p}{\mu w}\left[R^2 y - \frac{y^3}{3}\right]_0^R$$

$$Q = \frac{\Delta p}{\mu w}\left[R^3 - \frac{R^3}{3}\right]$$

$$Q = \frac{2\Delta p R^3}{3\mu w} \qquad \text{(eq. 3)}$$

Therefore Q α R^3



If the number of cracks is doubled but surface area remains the same (R=r/2), and if $Q_1 = KR^3$ is the flow through a single crack, then for the two half-sized cracks taken together the flow will be:

$$Q_2 = 2 \times K\left(\frac{R}{2}\right)^3 \qquad Q_2 = K\,\frac{R^3}{4}$$

It can therefore be concluded that distributing cracks in order to achieve twice as many cracks for the same surface are will quarter the flow through the cracks.

This analysis will only apply if the flow through the crack is laminar. The crack has the smallest surface area in comparison with the rest of the rig through which water will flow, therefore flow is least likely to be laminar in the crack.

For an estimated flow of 0.1m/s, and a crack width of 0.001m (a crack width of 1mm is likely to be the largest crack size possible), Reynolds number can be calculated:

$$\text{Re} = \frac{U2R}{\mu}$$

where U: mean velocity = 0.1   2R: Crack width = 0.001 μ : viscosity = 10^{-3}

therefore Re = 0.01

A Reynolds number of less than 1 indicates that the flow is laminar. Therefore the Plane Poiseuille Flow analysis holds.

5. Reducing cracking in constrained mortar

In this section the methods that were tested are discussed in more detail.

5.1 Plain Mortar

Initially four samples of plain mortar were tested. All samples had a sand/mortar ratio of 3 and a water/cement ratio of 0.5, they were cured in a humid environment for seven days and left for cracks to form for a further twenty-one days. Therefore each sample tested was twenty-eight days old. The expected tensile stress for these samples was calculated to be 3.66×10^{-6}N/m^2.

An expression for calculating crack width can be derived using stress, strain and Youngs Modulus for mortar in tension:

For mortar in tension:

$\sigma = \sigma_{UTS}$ where σ_{uts} is the ultimate tensile stress.

Youngs Modulus = stress/strain

$$\therefore \sigma = -E\varepsilon$$

$$\therefore \varepsilon = \frac{\sigma_{UTS}}{E}$$

Crack Width = circumference (-ε) / number of cracks, where –ε is the shrinkage strain less strain accommodated by tension. The shrinkage strain is 0.0045 (shrinkage of plain mortar – see appendix 1). Strain accommodated by tension is σ_{uts}/E.

Therefore:

$$2R = \frac{\pi d}{n} \times \left[0.0045 - \frac{\sigma_{UTS}}{E} \right] \qquad \text{(eq. 4)}$$

Substituting values for plain mortar:

Diameter of rig = 0.165m	$E = 28x10^{-3} N/m^2$ (Illston 1994)
$\sigma_{uts} = 3.66x10^{-6} N/m^2$	*R n = 1.13x10*$^{-3}$

Where *R* is half crack width and *n* is number of cracks. For example if we could expect ten cracks, then the average crack width would be: *0.00023m*, for plain mortar.

Referring to equation 3 in section 5:

$$Q = \frac{2\Delta p R^3}{3\mu w}$$

Substituting values:

In this case *2R = 0.00023*	R = 0.00012 m
$\Delta p = \rho g h$	(in testing a head of 2m is required)
$\rho = 1000\ kgm^{-3+}$	$g = 9.81\ ms^{-2}$
h = 2m	therefore Δp = 19620 Pa
$\mu = 10^{-3}$	*w* (mortar thickness) = 0.01m

Therefore the flow rate through one crack of width 0.00023m is:

$Q = 0.0023\ m^3 s^{-1}$

This is a leakage rate of 2.3 litres/s, in one day a crack of this size will leak 198720 litres, i.e. almost 200,000 litres leakage per day. This is a phenomenal amount, and it must be kept in mind that any measured leakage rate must eventually be projected onto a full size tank with a capacity of $10m^3$ (1000 litres): diameter 2.5m, height 2m. *[Note: at such high flow rates the analytic neglect of the pressure drop (= velocity head) to get the flow into the crack becomes invalid.]*

5.2. Rockfast

Brown 2001:

Rockfast cements are shrinkage-compensated cements based on the incorporation of calcium sulphoaluminate into a Portland cement system to give a composite cement which exhibits rapid setting, high early strength and dependent on proportions either shrinkage compensation or positive expansion. According to the literature by Blue Circle Cements, a 12% Rockfast replacement of Portland cement will lead to an expansion of 0.6% and a compressive strength of 45.2 N/mm² in 28 days. As the shrinkage of mortar is known to be 0.45%, an expansion of this much is required and therefore 9% replacement is required. According to this theory, therefore a 9% Rockfast replacement of Portland cement should lead to no cracking in the mortar.

5.3. Mesh reinforcement

The method of mesh reinforcement is currently being used in DTU rainwater harvesting tanks. For the purposes of testing a thin layer of mortar was plastered onto the rig. The mesh was placed over the first layer and a second thin layer was applied in order to incorporate the mesh into the mortar. In practice the mesh is held by wooden blocks a certain distance away from the tank wall and the mortar pushed through, but this was not seen as convenient or possible for the small scale on which it was being applied.

5.4. Fibre reinforcement

In tank construction fibres commonly used in fibre reinforced concrete (FRC) manufacture such as steel and glass would need to be purchased from a manufacturer, it was therefore decided to use vegetable fibres as these would be cheap, available and possible to process on site.

Property	Jute[3]	Sisal[3]	Coconut[3]	Sugarcane bagasse	
				Ref. 2	Ref. 3
Tensile strength (MPa)	250–350	280–750	120–200	170–290	20
Modulus of elasticity, (GPa)	26–32	13–26	19–26	15–29	1·7
Elongation at break, (%)	1·5–1·9	3–5	10–25	—	—
Fibre diameter, (mm)	0·1–0·2	—	0·1–0·4	0·2–0·4	0·24
Fibre length, (mm)	1800–3000	—	50–350	50–300	—
Water absorption (%)	—	60–70	130–180	70–75	78·5

Table 2: Properties of natural fibres (from Bentur 1990)

Sisal fibres were used as these were readily available, the fibre length was 5-10 mm and its diameter less than 0.2mm. The stress – strain curves of fibres show an ultimate strain in the range of 1 – 5%, which is much greater than that of the matrix (i.e. mortar). Therefore the mixing of 0.5% fibre (by weight to the mix) should have a positive effect on the strength of the mortar and reduce cracking, or distribute cracks.

In mixing technology, the increase in fibre content and length is associated with reduced workability. A fibre length of around 0.01m was used.

5.5. Double layer with nil coat in-between

Plain mortar mixed in the same proportions as before was used for this render. A thin layer of plain mortar was applied. This was cured for seven days and then left for a further week to dry and crack, then a cement-water wash (nil coat) was applied as the next layer. This layer was allowed to dry and crack for two days before the final layer of plain mortar was applied, this was cured for a further seven days and allowed to dry and for cracks to form for 21 days, as with the other renders. The expectation is that the first layer will crack, these cracks are filled by waterproof nil, then the second layer is applied and also shrinks and cracks, but hopefully in different places so the cracks do not overlap.

6. Leakage Testing

6.1. Experimental method

6.1.1. Previous problems

The experimental method used last year was not devised to simulate a water tank, but to create a condition in which the mortar would be constrained to induce cracking. The mortar was therefore set around a mild steel cylindrical pipe (length 140mm, diameter 165mm, wall thickness 5mm), this would ensure maximum constraint and cracking, and therefore give measurable flow rates. The test rig was required to be such that water could flow behind the mortar, i.e. between the outside of the pipe wall and the mortar. This was the area, which had given the most trouble in the previous year. A helical groove was machined into the outside of the pipe with a pitch of 50mm, and a depth of 3mm (this depth was not constant as the cross-section of the pipe was not a perfect circle). The groove was intended to be the channel through which the water would flow, therefore a way of keeping the channel clear was required so that when the mortar was applied it did not block the groove. Two different methods where tried - laying string in the groove and laying wire in the groove. Both were considered unsatisfactory, not allowing the water to flow freely through the channel, and therefore not ensuring all the cracks were fed.

6.1.2. Finding a solution

In order to find the best way of keeping the channel open it was decided to conduct a short test using three different methods.

- Method 1: using a thin length of string in the channel to act like a wick for the water through the channel:

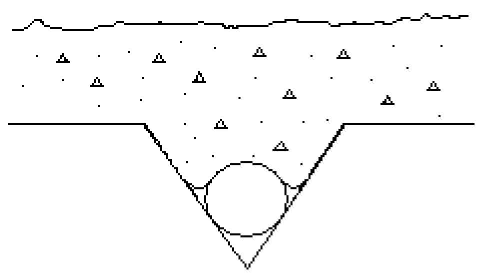


Figure 13: diagram showing string in v-shaped groove, with a layer of mortar on top.

- Method 2: Using a thick length of string to keep mortar from blocking the channel and allow water to flow through it:

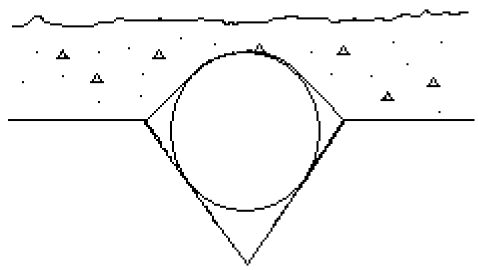


Figure 14: same as figure 13 but with bigger cross-sectional area of string so it acts as a barrier to the mortar entering the groove rather than a wick.

- Method 3: Using a crimped wire on order to keep the channel open and allow water to reach the mortar:

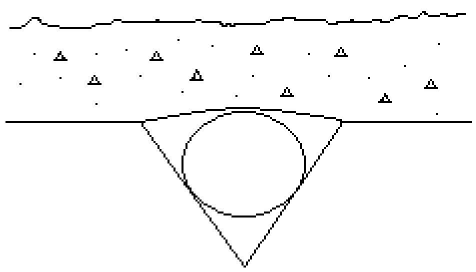


Figure 15: A wire is used instead of the string, the wire cross-sectional area is chosen so that it sits roughly flush with the pipe outside wall.

In order to test these methods each was applied to a rig with a piece of material wrapped tightly round the whole rig to keep the string and wire in place in the channel. It was hoped that the most effective method of keeping the channel open and allowing the water to flow through to the mortar would become apparent by measuring the leakage rate through the material. However, the leakage rate was faster than could be measured and flowed straight through the wrapping material bypassing the channel.

By doing this test it became apparent that a layer of material on the rig in between the mortar and the pipe wall would be very effective in keeping the channel open and would not have the drawbacks of stopping the water reaching the whole channel or slowing flow down significantly as the string did.

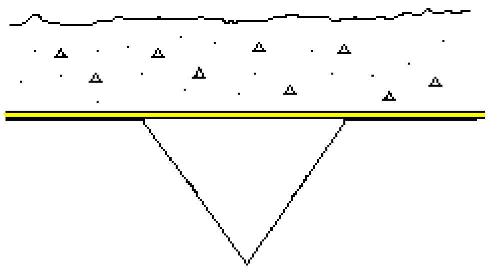


Figure 16: the channel is kept open by a piece of material between the mortar and the pipe.

As the material had to be wrapped tightly around the pipe, calico was chosen, being pure un-shrunk cotton it could therefore be shrunk onto the pipe. The calico was sewn to fit the circumference of the pipe, then shrunk onto the pipe in water.

6.2. Modifying the test rig

There were four existing rigs, which would need to be modified to ensure that the water flowed through the entire channel therefore feeding all the cracks. A manifold to better connect the channel was included, this had to be sawn by hand and filed to a v-shaped groove. A representation of the rig seen from front on (outside pipe wall) is shown in figure five below, originally the rig had only a helical groove (channel) and a hole for feeding water through from the inside at one end:

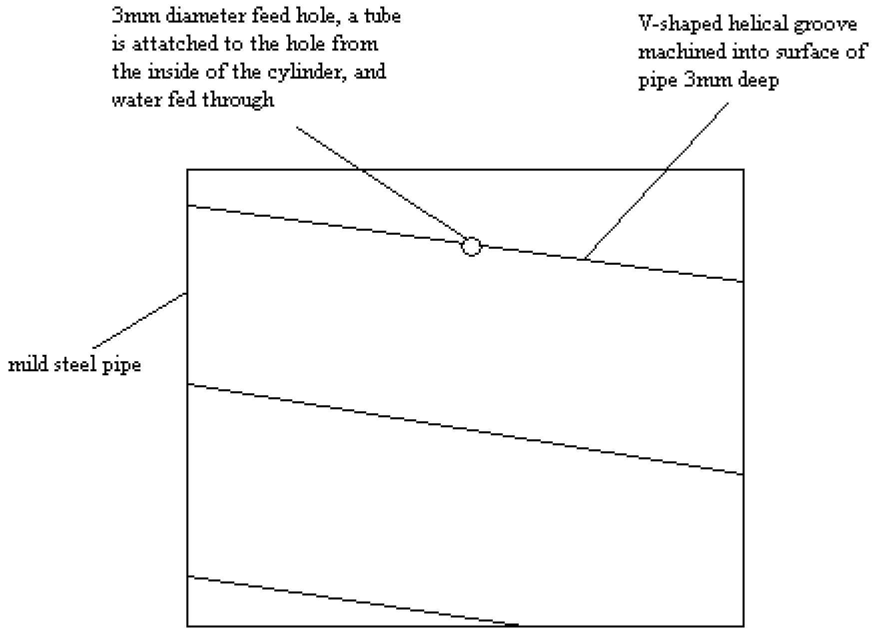


Figure 17: a representation of the front of the test rig (looking at the outside wall) – the original rig used in last year's tests (2001).

Two manifolds diametrically opposite eachother were included to run the whole length of the pipe, the original feed hole was blocked and a new hole drilled in the middle at a point where a manifold intersected the groove. The point at which this manifold intersected the last section of the channel was too near the end of the pipe. This was a problem as the ends were to be sealed due to the likelihood of water leaking from the ends, therefore a connecting channel was also included in the same manner the manifolds had been. Two bleed holes where then drilled into the pipe wall diametrically opposite the feed-hole intersecting the channel (therefore intersecting the second manifold). These were to ensure that air was pushed out of the channel as the water flowed around it, and would be immediately blocked as water came through.

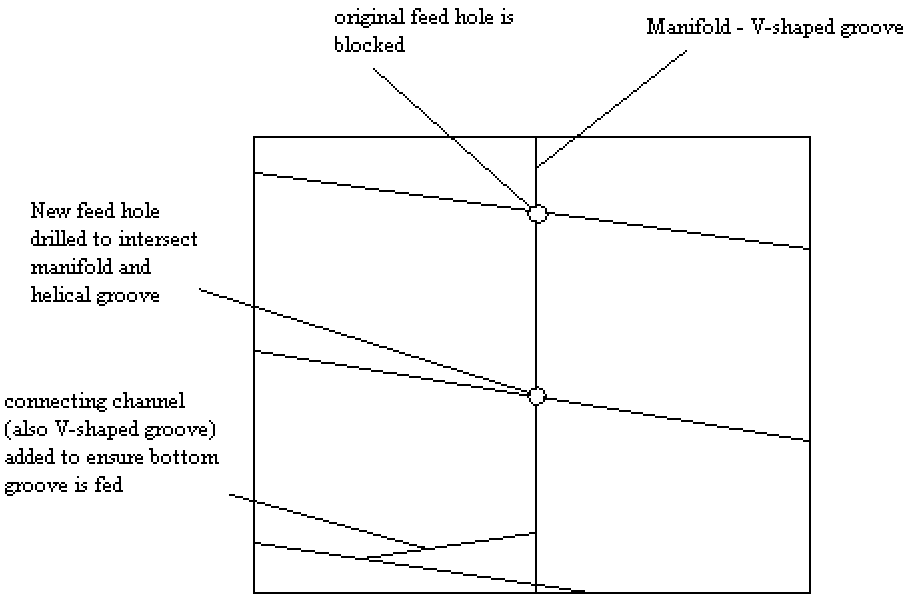


Figure 18: The test rig once modified to improve flow, the two bleed holes are not visible as they are on the other side, they also intersect with the channel.

The test rig was therefore set up as shown below, with a pipe to the feed hole, fed from the bottom to ensure that air was pushed up and out of the channel.

A two meter head was used in order to ensure the water was pushed up the groove and also to be able to compare results with a real tank, which has a height of 2m.

As well as modifying the existing rig, a new rig design was submitted using the same size and material for the pipe but with much closer grooves to ensure that the maximum crack area was fed (see appendix 2 section 2.3). The new design incorporated grooves at 1cm pitch, 2 manifolds diametrically opposite intersecting 2 feed holes at one end of the pipe (one on each manifold), and 2 bleed holes at the other end. This rig was designed so it could be tested upright rather than on its side.

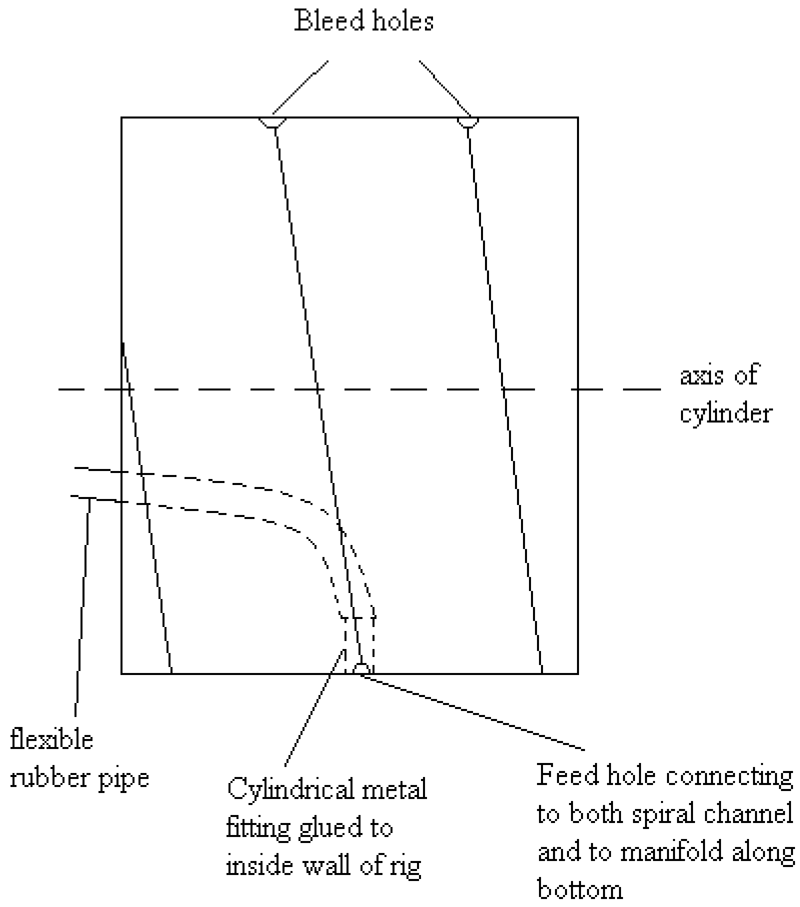


Figure 19: The test rig, as used for testing (looking at the pipe wall but with the rig on its side).

6.3. Constructing render samples

A 0.01m thick layer of mortar was initially applied to two rings, the cement : sand ratio used was 1:3. The water to cement ratio was 0.5. This formed a very thick paste, which was difficult to apply to the pipe wall (now covered in calico), as due to the dryness of the mix it had a tendency not to stick. However, it was decided to keep with a dry mix as more water in the mix the compromises on strength in the mortar. The rigs were left to cure in a bag for seven days before drying outside the bag for a further 21 days. This was done as in the last project it had been noted that it took 28 days for the cracks to form.

The mortar was then applied to the two remaining rings with the same ratios as used before of cement, sand and water. Due to concerns of leakage from the edges it was decided to modify the rings slightly by applying the calico so that roughly 2cm of the rings were left exposed at either end, these ends were roughened using a file. This was to ensure the mortar would stick to the pipe ends and reduce any leakage or seepage through the material if it protruded from the ends. Silicone sealant was also applied to the ends of all four rigs.

The last two of the four rigs were expected to yield better results as the plastering was of a better quality, with a constant thickness throughout. Also there had been more measures taken to prevent end leakage.

After these rigs had been tested the mortar was chipped off, at this point the newly designed rigs had also become available. With eight rigs, it was possible to test four variations of render with two rigs to each render. The renders were applied as described in section 5, the most difficult to apply being the wire mesh as it had to be held onto the outside of wet mortar ensuring no gaps between mesh and mortar:

Figure 20: making a wire mesh reinforced render, (on right – applying mesh into wet mortar, on left – mortar chipped away after testing to reveal mesh).

7. Testing

The rigs were set up so that a 2m head could be obtained. A pipette was attached to the top end of the pipe and water poured through a funnel into the pipette. Two people were required to conduct the testing, as a person was needed at the bottom end to block the bleed holes the moment water poured through them.

Initially testing went to plan as the water first emerged from the bleed holes before the cracks so it could be assumed that there was no air in the channel interfering with the flow. However, once the bleed holes were blocked it soon became apparent that water was flowing through the cracks faster than it could be poured through the pipette, therefore it was not possible to measure the leakage rate.

This proved that the testing method had been improved in the sense that flow was most definitely not inhibited in the channel. However, the leakage rate needed to be measured and so the set up of the experiment was altered. A wider pipette and pipe were used, initially the pipette had a diameter of 7mm, it was now 11mm. Also the pipette was moved down to the bottom so that the water level was not falling immediately at the point it needed to be measured, i.e. the person at the top could stop pouring when the person at the bottom had blocked off the bleed holes and ensured that the water was flowing through the cracks, and then after a few seconds the top of the water level would reach the pipette so the person at the bottom could time the drop. This worked well and a leakage rate was recorded.

Figure 21: set up of the rig showing pipe connecting to channel and pipette

Figure 22: Leakage through a crack

For later tests a reservoir was introduced so that testing only required one person. Water was held in a container 2m high, with a tap, which was attached to the rubber pipe. There was also a valve on the rubber pipe, positioned just above the pipette so one person could control the flow from the bottom (see figure 5 section 3.1 for a rough schematic of how the test was set up).

Crack measurement was done using a microscope with a graticule that measured to 1/20 of a millimetre.

7.1. Problems encountered during testing

As the leakage rate was high due to low resistance in the rig (see appendix 2 section 2.2.), ensuring accurate results was difficult as the water level in the pipette tended to drop very quickly. This was a problem, which could only be overcome by repeating tests and obtaining an average.

A major problem encountered was leakage from the ends, as can be seen below in figure 23. The mortar is wet along the edge below the silicone sealant, indicating water is leaking from the sealed edges:

This had to be overcome by observing leaks, marking them and applying greater quantities of sealant. In the case of the wire mesh samples this was done twice as the end leakage was excessive in these samples. Therefore, in the results there are three different flow rates for the mesh samples, the smallest being that with all end effects removed. This was also done once for the Rockfast sample, giving two flow rates for Rockfast.

Figure 23: A sample with end leakage

8. Results

8.1 Crack measurement results

Two different types of rig were used, the *original rig* used in last years experiments by Tom Constantine and the *new design* of rig detailed in section 6.2.

The plain mortar samples were both set on original rigs, as these were the first samples to be made and the rigs made to the new specification were not available at that time. There are no crack measurement results for the samples made with two layers of mortar with a cement-water wash, as no cracks were visible on these samples. The results are detailed in appendix 3 tables 3.1. to 3.8., and summarised in section 9.1 to enable analysis.

8.2. Leakage results

For the Rockfast tests there is only one leakage result, due to the failure of one rig (the original design), there was a blockage at some point in the groove and not all cracks were being fed also one of the bleed holes did not leak which implies air was trapped inside the channel.

As the results are different for the new rig and old rig, there are two tables – one showing the raw data as measured (appendix 3 table 3.9.) and table 3 below with the results for the new rig normalised so they can be compared with the old rig results.

Table 3: Normalised data for leakage rates

Rig	Type	Test number	Flow rate (m^3s^{-1} E-06)
Original	1) Plain	1	6.2
Original	2) Plain	1	7.6
Original	3) Plain	1	4.6
Original	4) Plain	1	3.7
Original	Mesh	1	0.51
New design	Mesh	1	0.43
		2	0.27
		3	0.23
Original	Rockfast	nil test	
New design	Rockfast	1	10.6
		2	5.6
Original	Fibre	1	4.8
New design	Fibre	1	4.7
Original	Double layer	1	2.2
New design	Double layer	1	2.7

9. Analysis

9.1. Analysis of cracking results

Table 4: Crack measurement results and projected expectations of flow

Sample	Number of cracks	Average crack width (E-03m)	Average crack surface area (E-03m^2)	Total crack surface area (E-03m^2)	Shrinkage	Expected flow rate (E-03m^3s^{-1}
Plain mortar 1	9	0.089	0.0075	0.067	0.00089	1.037
Plain mortar 2	8	0.093	0.0069	0.056	0.00075	1.04
Mesh 1	9	0.036	0.0012	0.01	0.00013	0.08
Mesh 2	9	0.034	0.001	0.0092	0.00012	0.06
Rockfast 1	7	0.100	0.0093	0.065	0.00087	1.31
Rockfast 2	9	0.088	0.0077	0.07	0.00093	1.01
Fibre 1	9	0.130	0.010	0.09	0.0012	3.16
Fibre 2	5	0.120	0.013	0.065	0.00087	1.56

9.1.1. Shrinkage

The shrinkage of the samples was calculated by dividing the total crack surface area by the actual surface area of the mortar on the rig. Analysis of these results reveals that the only effective method of reducing shrinkage was the wire mesh reinforced mortar. The addition of Rockfast made no difference to the shrinkage or crack distribution as the figures for shrinkage are similar to the figures for the shrinkage of plain mortar. Fibre reinforcement also made no difference, having one of the highest shrinkage values. The expected shrinkage for plain mortar, *0.0045*[6], is not reflected in the results, the highest shrinkage being 0.0012, almost a quarter of the expected value. The expected average crack width calculated in section 5.1., for an estimated 10 cracks around the set diameter of the test rig was 0.00023m, in reality the highest average crack width found in the fibre reinforced samples was 0.00013m, almost half the expected width.

9.1.2. Expected flow rate

The expected flow rate through each mortar lining was calculated using the value for average crack width substituted into equation 3 defined in section 5:

$$Q = \frac{2\Delta p R^3}{3\mu w} \qquad \text{eq. 3}$$

Where R is half the crack width. The figure obtained from this calculation was then multiplied by the number of cracks in each rig in order to give a total leakage value for the whole section of mortar. These flow rates will be used in the leakage results analysis below.

9.2. Analysis of leakage results

Table 5: Comparison of expected flow rate with actual

	Theoretical flow rate ($E\text{-}03 m^3s^{-1}$)	**Measured flow rate ($E\text{-}03 m^3s^{-1}$)**
Plain mortar 1	1.037	0.00463
Plain mortar 2	1.04	0.00371
Mesh 1	0.08	0.00051
Mesh 2	0.06	0.00043
		0.00027
		0.00023
Rockfast 1	1.31	
Rockfast 2	1.01	0.01056
		0.00560
Fibre 1	3.16	0.00480
Fibre 2	1.56	0.00474
Double layer 1		0.0022
Double layer 2		0.0027

The table above shows a comparison of the expected flow results according to the theory modelling a crack as laminar flow between parallel planes, and the actual flow rates obtained through measurement. The theory predicts a much higher flow through the cracks measured, in comparison with the actual flow, which in most cases is more

[6] Neville 1995

than 100 times smaller. As well as comparison with the theory it is important to realise findings must be projected onto a much larger tank, which will hold $10m^3$ of water (10,000 litres). The leakage rate for the area of mortar on the test rig was projected onto the larger area of a tank with a height of 2m and a diameter of 2.5m (see table 3.10. appendix 3). The calculation was done for the expected leakage rates and the actual leakage rates measured.

Table 6: results projected for leakage in one day in a tank of volume$10m^3$

	Expected leakage in full sized tank in one day (litres)	Projected leakage (from experimental results) in full sized tank in one day (litres)
Plain mortar 1	18755597	83758
Plain mortar 2	18809856	67010
Mesh 1	1446912	9188
Mesh 2	1085184	7808
		4909
		4091
Rockfast 1	23693184	
Rockfast 2	18267264	190979
		101202
Fibre 1	57153024	86869
Fibre 2	28214784	85806
Double layer 1		39718
Double layer 2		48869

In a 10,000 litre tank, ideally a leakage of more than 10litres a day is unacceptable. Therefore projecting the figures to a full size tank has produced leakage rates, which are far too high and seem unrealistic. Even using the values for mortar reinforced with chicken mesh, which give the lowest leakage, a full tank would be empty in two days.

10. Discussion

In section 1.4 two objectives were outlined:

- To improve on the experimental method used in the previous year to yield more credible results.
- To measure leakage flow through renders in order to recommend methods of effective waterproofing for water tank construction.

10.1. Experimental procedure

The problem encountered by Tom Constantine last year was too large a resistance in the channel, which therefore bought into question whether water was allowed to flow freely and feed all the cracks. The resistance in the rig was greatly reduced by removing all obstructions in the channel, however this then led to the problem of a leakage rate so fast it was difficult to measure. With such low resistance in the channel end leakage also became a problem and some results should have been repeated but could not be due to lack of time.

The rig produced a totally constrained state for the mortar and leakage was also made higher by the large pressure difference between inside the channel and the outside wall. In practice the mortar is backed by earth or bricks, which would serve to reduce the leakage rate. Therefore, with all these factors taken into account the rig is not an ideal approximation of real life. Projecting the leakage rates onto a tank with a 10,000 litre capacity does not yield accurate results for leakage in large tanks, as such tanks are in use and leakage rates are not as high as those calculated in this report. However, the test rigs give a good method of measuring leakage rate through mortar, if all unwanted leaks can be overcome. The rig can provide a controlled system, which allows water to flow freely behind mortar and a reliable method of testing flow rate through cracks. Therefore it is a good method for comparison between different measures to reduce cracking. From the results obtained in this report, wire mesh had the best performance in comparison with the other techniques tested.

10.2. Methods of waterproofing

Rockfast

Although expected to eliminate cracking in the mortar due to shrinkage compensation, the addition of Rockfast produced the same results as plain mortar, if not fairing slightly worse. The shrinkage of plain mortar was in fact less than that predicted by the theory, according to Neville plain mortar shrinks by 0.45%, in tests it was found to average at 0.082%. It is therefore possible that the amount of Rockfast used was too much and instead of shrinkage compensation, encouraged further expansion. Due to the constrained nature of the samples this may have led to internal stresses developing which would have caused cracking.

Fibre

The fibre-reinforced mortar also performed very badly, again giving no real difference from plain mortar. As fibres make mixing difficult, it is possible that this led to a poor quality mortar. During plastering, it was necessary to plaster from the bottom of the rig to the top (along the length of the cylinder) as any other way resulted in the mortar falling off. It is therefore also possible that this plastering action aligned many of the fibres in the wrong direction, as cracks propagated along the length of the cylinder and therefore the fibres would be required to be perpendicular to the cracks, rather than along the same line.

Double layer

The rigs coated with two layers of mortar and a cement-water wash between the layers, performed fairly well. It was harder to seal the ends of these rigs as the layer of render was much thicker than on the other rigs. This may explain why the leakage rate is higher than expected, seeing as no cracks could be identified on the mortar surface. Despite this, the leakage rate with this method of rendering was half that of the three previously discussed. In the building of a real tank using this method is inconvenient as the site will need to be visited at least three times for the application of each layer, and both mortar layers have to be allowed to cure and dry. This also increases expense.

Wire mesh reinforcement

This performed the best of all four methods, reducing the leakage rate from plain mortar by a factor of 10. However, there is a possibility that carbonation will occur with chicken mesh in mortar, particularly in hot countries. Carbon dioxide reacts with alkalis stripping the protective layer on steel, especially in hot countries with poorly made mortar. The mesh will rust in the mortar, leaving the mortar weak and prone to cracking. However, on the basis of the tests performed, wire mesh is most effective in preventing cracking.

10.3. Mortar Quality

Mortar quality is an important factor affecting mortar strength and cracking, that has not been addressed in this investigation. Implementing a rainwater harvesting project in a developing country involves teaching local workers the necessary skills required to build tanks.

The mortar needs to be as dry as possible, however, this makes the plastering job slightly harder and therefore, if emphasis in not placed on the correct water content plasterers may add more water to increase workability. Watt describes the problem of too much mortar being mixed in one go, which leads to a large amount becoming stiff as it is left out in the sun. In this situation it is not uncommon for the person plastering to add a little more water to make it workable again – this will also compromise mortar strength and quality. There also needs to be a great emphasis placed on proper curing, as this increases mortar strength. Without these measures being taken cracking will occur in mortar.

11. Conclusions

The use of test rigs with the channel kept open by a layer of material between the channel and the mortar worked well in reducing resistance in the water-feed path. The test rigs provide a method of comparing renders, however they do not provide a good approximation to a real size tank.

A better method of sealing the ends is required, as in some cases silicone sealant was not sufficient to stop end leakage. For this reason the tests in this report need, ideally, to be repeated.

The best render was the wire mesh reinforced mortar. Although it produced the same number of cracks, the crack sizes were greatly reduced. In plain mortar the average crack width was almost 0.1mm, for mesh the average crack width was 0.035mm, a reduction of almost two thirds. The leakage was reduced by more than a factor of 10.

11.1. Further work

The method of testing needs to be further improved so it can be used to better approximate real size tanks. The use of better seals at the ends of the rigs should reduce the leakage rate to reflect only the leakage from the cracks. One possibility is the combined use of silicone sealant and fix clips (used by Tom Constantine in 2001). If the flow can be reduced, it would make the leakage rate easier to measure and leave fewer margins for error in the timing of the drop in water level through the pipette.

There were many methods of waterproofing renders that were not tested in this report, for example the use of a waterproofing paint, the use of superplasticisers and other chemical admixtures. Also the methods tested need further development,

- testing with different amounts of Rockfast to obtain the right shrinkage compensation
- testing with different types of fibres
- longer term tests with wire mesh to study the effects of carbonation
- repeating tests with an improved end sealing technique

The tests also need to be repeated as two samples of each render are not enough to give conclusive results. The main recommendation that can be made is greater research into wire mesh reinforcement.

Appendix 1 – Concrete and Mortar Data

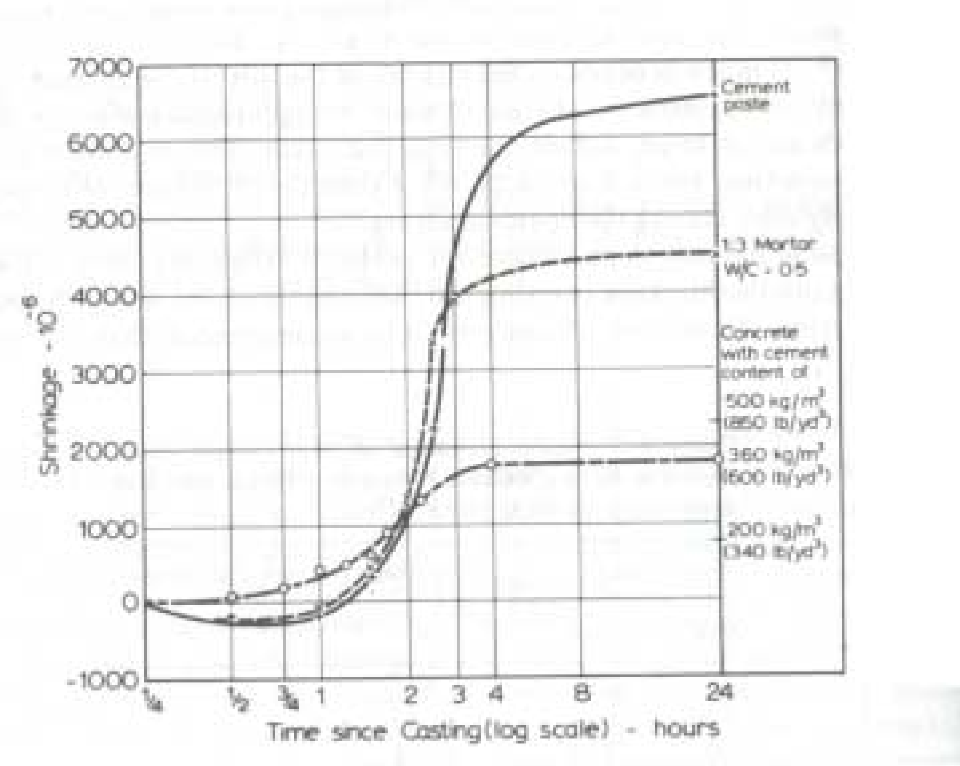


Figure A1.1.: Influence of cement content of the mix on early shrinkage (Neville 1995)

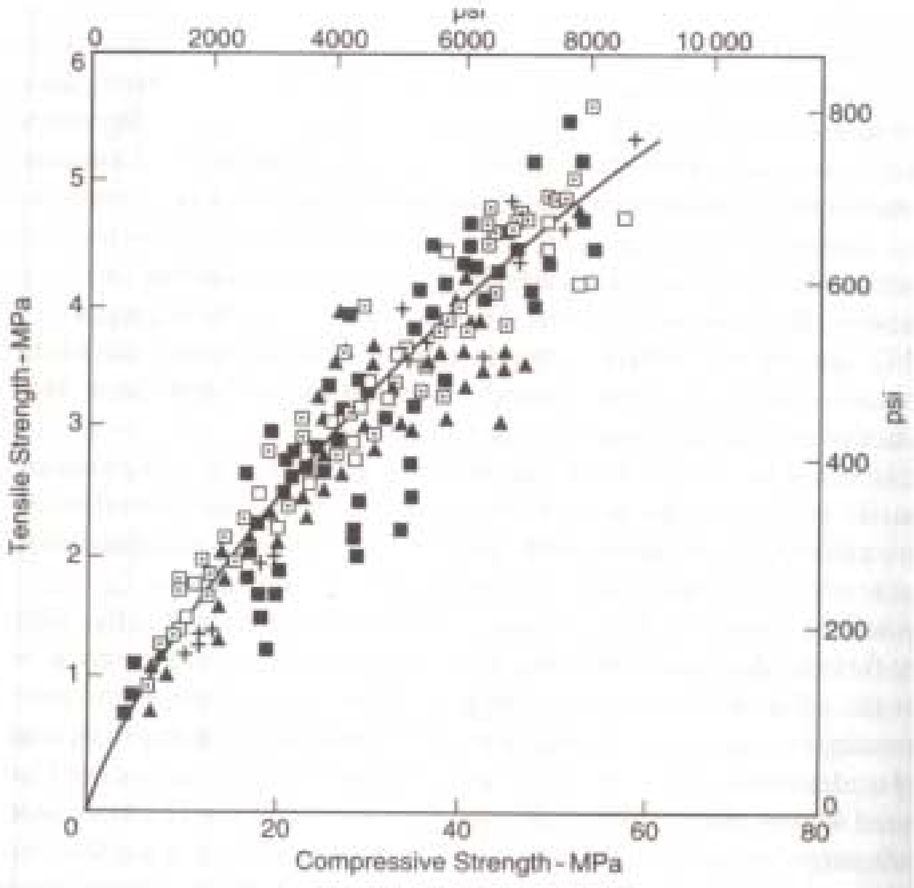


Figure A1.2.: The relationship between tensile and compressive strength (Neville 1995)

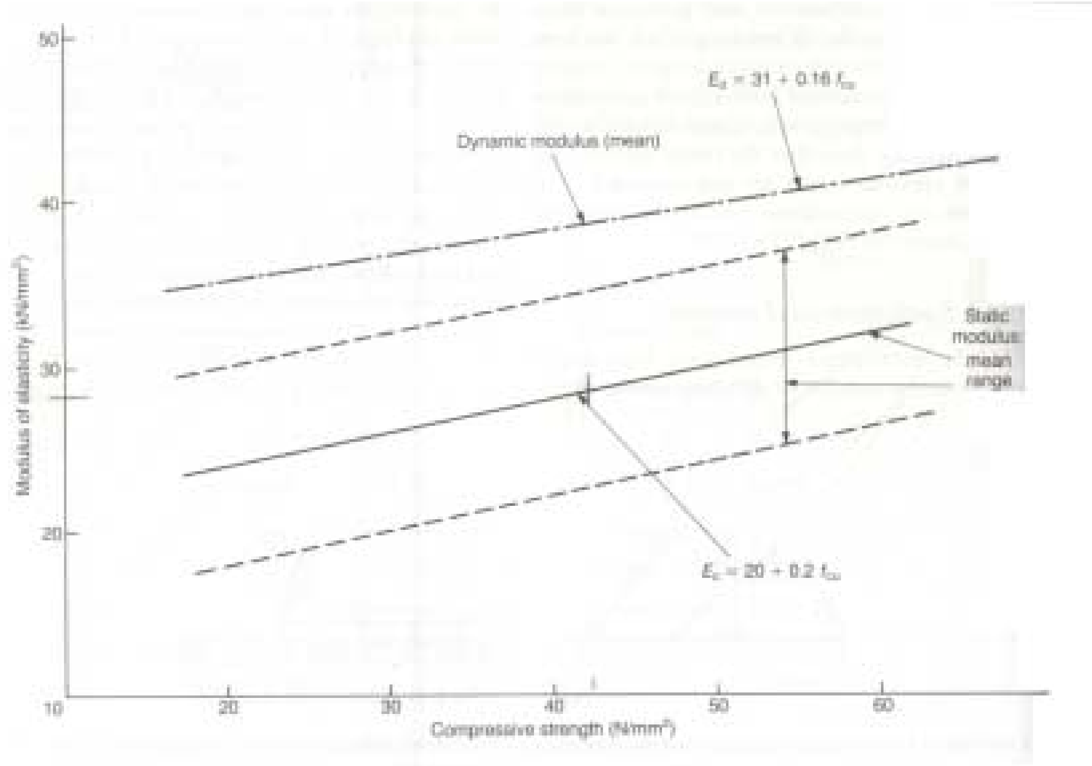


Figure A1.3.: The relationship between compressive strength and Modulus of elasticity (Illston 1994)

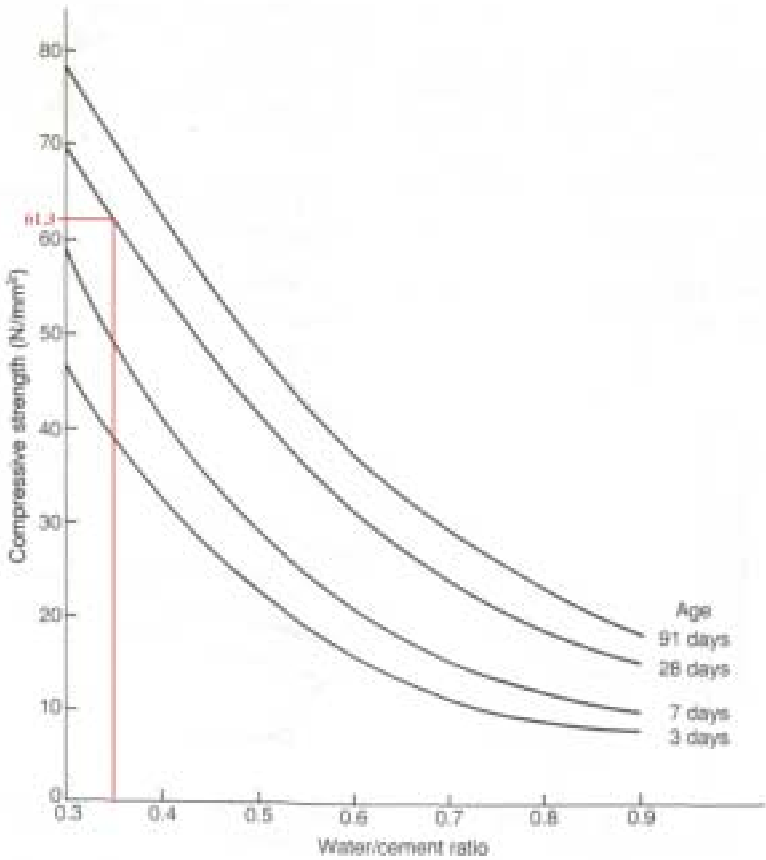


Figure A1.4 Relationship between water/cement ratio and strength (Illston 1994)

Appendix 2 – experimental procedure

A2.1. Calculations carried out before testing began

To ensure a measurable leakage rate through the rig:

Suppose threshold of interest is 1litre/day

Suppose tank is 1m diameter x 2m high

Mean pressure head 1m – (similar to experiment)

1litre/day loss = q Area secs/day

q = 0.001/6.3*86000 = $1.84 \times 10^{-9} m^3/s$ per m^2

Test rig:- Mortar area = $\pi \times 0.165 \times 0.14 = 0.0725 m^2$

Therefore leakage rate per hour = $0.0725 \times 1.84 \times 10^{-9} \times 3600$

= $0.48 \times 10^{-6} m^3/hr$

= 0.48 ml/h ← just about observable

Check that surface tension of water does not prevent flow:

Surface tension of water at 20°C = 72×10^{-3} N/m

Force per metre on film acting downwards is: $72 \times 10^{-3} \times 2 \times \cos\theta$

Assume $\cos \theta = 1$

$F = 1pw$, so $pw = 1.44 \times 10^{-3}$, and $p = 1.44 \times 10^{-3}/w$

If crack width w = 0.0001m (0.1mm), then p = 1440 Pa, i.e. 14 cm of water

A2.2. Resistance in rig

The flow rate through the bleed holes was measured i.e. water flowing straight through rig, bypassing cracks. This rate was 6 litres/second ($0.006 m^3 s^{-1}$ → 10^3 times faster than the flow rate through plain mortar), therefore resistance is negligible in the channel.

A2.3. Comparison of feeding cracks through channel and feeding cracks in an actual tank

In the test rig cracks are fed primarily at points where they are intersected by the channel, figure 2.1. below shows a schematic of the water distribution in a crack as fed by the channel:

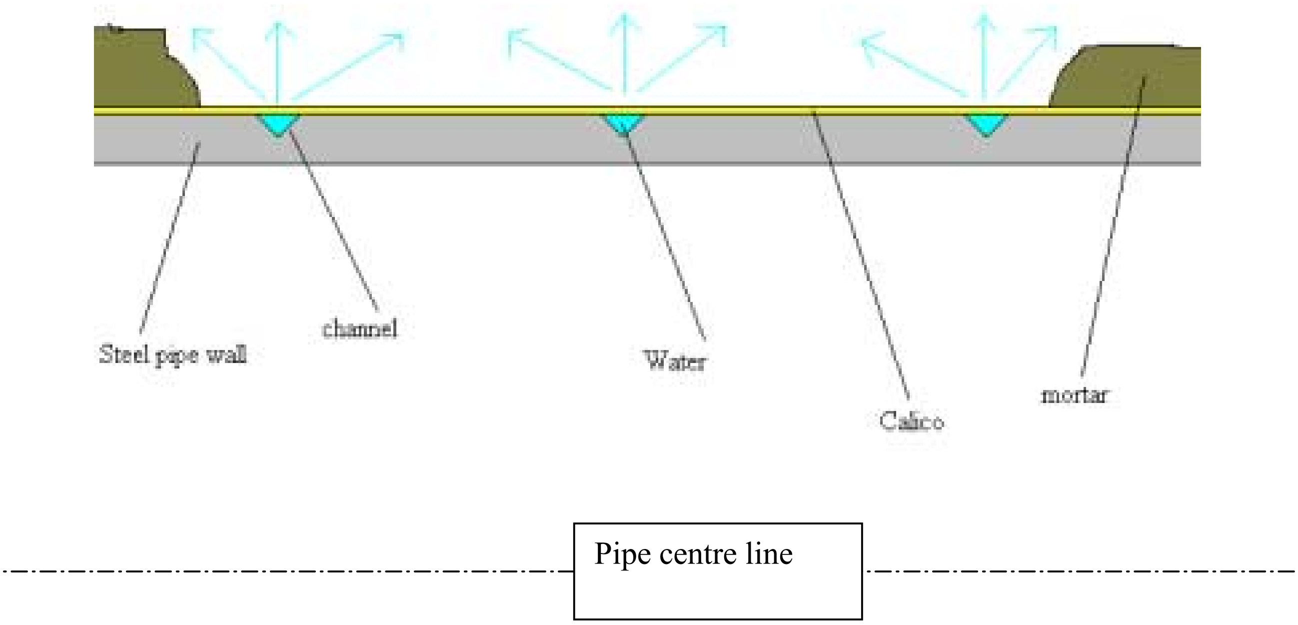


Figure 2.1.: Cross section of mortar and rig, at a point where there is a crack. Water is not evenly distributed through crack

If this is compared with the same situation in a real tank:

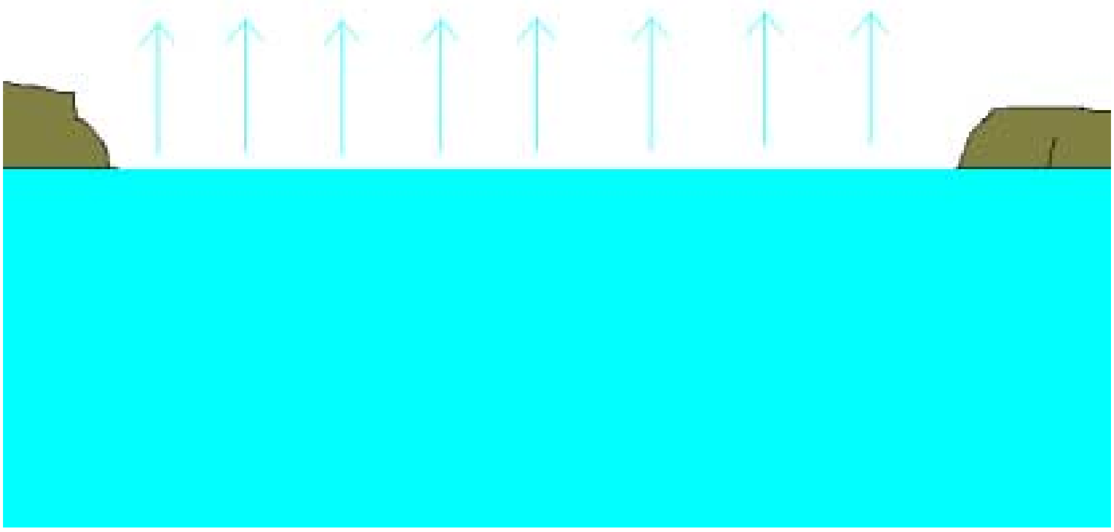


Figure 2.2: in a real tank water distribution is constant

This implies the measured leakage rate from the rig is an underestimate. However, it should also be taken into account that the rig provides the most severely constrained state to ensure maximum cracking. Also mortar lining in real tanks is backed by earth or bricks, whereas in the tests there is only air on the other side of the mortar, so the pressure difference is far greater which will lead to increased flow. Therefore it can be assumed that these factors will have a cancelling effect.

Appendix 3 - Results

***Table 3.1 Plain mortar 1* (original rig)**

Crack number	Length (m)	Average width (E-03 m)	Crack surface area (E-03 m²)
1	0.14	0.2	0.028
2	0.065	0.11	0.00715
3	0.038	0.07	0.00266
4	0.1	0.12	0.012
5	0.023	0.05	0.00115
6	0.081	0.15	0.01215
7	0.047	0.04	0.00188
8	0.056	0.03	0.00168
9	0.025	0.03	0.00075
Average crack width		0.089	
Total crack surface area			0.067
Average crack surface area			0.0075

***Table 3.2 Plain mortar 2* (original rig)**

Crack number	Length (m)	Average width (E-03 m)	Crack surface area (E-03 m²)
1	0.137	0.17	0.0233
2	0.02	0.05	0.001
3	0.04	0.06	0.0024
4	0.024	0.09	0.00216
5	0.095	0.11	0.0105
6	0.075	0.12	0.009
7	0.06	0.08	0.0048
8	0.043	0.06	0.00258
Average crack width		0.093	
Total crack surface area			0.056
Average crack surface area			0.00696

***Table 3.3. Mesh 1* (original rig)**

Crack number	Length (m)	Average width (E-03 m)	Crack surface area (E-03 m²)
1	0.03	0.04	0.0012
2	0.01	0.04	0.0004
3	0.025	0.02	0.0005
4	0.023	0.06	0.00138
5	0.038	0.04	0.00152
6	0.025	0.03	0.00075
7	0.035	0.04	0.0014
8	0.06	0.04	0.0024
9	0.03	0.03	0.0009
Average crack width		0.038	
Total crack surface area			0.01045
Average crack surface area			0.00116

***Table 3.4. Mesh 2* (new design)**

Crack number	Length (m)	Average width (E-03 m)	Crack surface area (E-03 m²)
1	0.01	0.03	0.0003
2	0.014	0.04	0.00056
3	0.026	0.02	0.00052
4	0.032	0.03	0.00096
5	0.034	0.03	0.00102
6	0.029	0.04	0.00116
7	0.056	0.06	0.00336
8	0.044	0.02	0.00088
9	0.01	0.04	0.0004
Average crack width		0.034	
Total crack surface area			0.00916
Average crack surface area			0.001017778

Table 3.5. Rockfast 1 **(original rig)**

Crack number	Length (m)	Average width (E-03 m)	Crack surface area (E-03 m²)
1	0.021	0.05	0.00105
2	0.034	0.07	0.00238
3	0.11	0.19	0.0209
4	0.018	0.03	0.00054
5	0.016	0.05	0.0008
6	0.07	0.06	0.0042
7	0.14	0.25	0.035
Average crack width		0.1	
Total crack surface area			0.06487
Average crack surface area			0.0093

Table 3.6. Rockfast 2 **(new design)**

Crack number	Length (m)	Average width (E-03 m)	Crack surface area (E-03 m²)
1	0.115	0.27	0.03105
2	0.068	0.04	0.00272
3	0.055	0.04	0.0022
4	0.025	0.05	0.00125
5	0.026	0.05	0.0013
6	0.13	0.14	0.0182
7	0.062	0.09	0.00558
8	0.055	0.06	0.0033
9	0.08	0.05	0.004
Average crack width		0.088	
Total crack surface area			0.0696
Average crack surface area			0.0077

***Table 3.7. Fibre 1* (original rig)**

Crack number	Length (m)	Average width (E-03 m)	Crack surface area (E-03 m^2)
1	0.125	0.37	0.04625
2	0.08	0.08	0.0064
3	0.038	0.18	0.00684
4	0.035	0.05	0.00175
5	0.117	0.15	0.01755
6	0.056	0.06	0.00336
7	0.035	0.17	0.00595
8	0.031	0.04	0.00124
9	0.024	0.06	0.00144
Average crack width		0.129	
Total crack surface area			0.091
Average crack surface area			0.0101

***Table 3.8 Fibre 2* (new design)**

Crack number	Length (m)	Average width (E-03 m)	Crack surface area (E-03 m^2)
1	0.054	0.09	0.00486
2	0.018	0.05	0.0009
3	0.131	0.22	0.0288
4	0.022	0.04	0.00088
5	0.131	0.22	0.0288
Average crack width		0.124	
Total crack surface area			0.064
Average crack surface area			0.0129

Table 3.9. Raw leakage data as measured

Rig	Type	Test number	Time taken (s)	Volume (m^3)	Flow rate (m^3s^{-1})	Flow rate (m^3s^{-1} E-06)
Original	1) Plain	1	9.6	0.000059	0.000006175	6.175
Original	2) Plain	1	7.8	0.000059	0.0000076	7.600
Original	3) Plain	1	2.4	0.0000111	4.63E-06	4.631
Original	4) Plain	1	16.0	0.000059	0.0000045	3.705
Original	Mesh	1	121.6	0.000062	5.078E-07	0.508
New design	Mesh	1	79.3	0.000054	6.8546E-07	0.685
		2	86.0	0.000037	4.3114E-07	0.431
		3	172.0	0.0000618	3.5912E-07	0.359
Original	Rockfast	nil test				
New design	Rockfast	1	2.8	0.000047	1.67E-05	16.761
		2	1.8	0.000016	8.88167E-06	8.882
Original	Fibre	1	3.6	0.0000173	4.80278E-06	4.803
New design	Fibre	1	8.2	0.00006175	7.53049E-06	7.530
Original	Double layer	1	4.5	0.00000988	2.19556E-06	2.196
New design	Double layer	1	2.0	0.00000858	4.2885E-06	4.289

Table 3.10. Comparison of expected leakage and actual leakage

	Expected flow rate (E-03m^3s^{-1})	Flow rate for 1m^2 of mortar (l/s)	Flow rate for full sized tank (s.a. 15.7m^2) (l/s)	Leakage in full sized tank in one day (litres)
Plain mortar 1	1.037	13.827	217.1	18755597
Plain mortar 2	1.04	13.867	217.7	18809856
Mesh 1	0.08	1.067	16.74	1446912
Mesh 2	0.06	0.800	12.56	1085184
Rockfast 1	1.31	17.46	274.2	23693184
Rockfast 2	1.01	13.47	211.4	18267264
Fibre 1	3.16	42.13	661.5	57153024
Fibre 2	1.56	20.80	326.6	28214784
	Measured flow rate (E-03m^3s^{-1})			
Plain mortar 1	0.00463	0.062	0.969	83758
Plain mortar 2	0.00371	0.049	0.776	67010
Mesh 1	0.00051	0.007	0.106	9188
Mesh 2	0.00043	0.006	0.090	7808
	0.00027	0.004	0.057	4909
	0.00023	0.003	0.047	4091
Rockfast 2	0.01056	0.141	2.210	190979
	0.00560	0.075	1.171	101202
Fibre 1	0.00480	0.064	1.005	86869
Fibre 2	0.00474	0.063	0.993	85806
Double layer 1	0.00220	0.029	0.460	39718
Double layer 2	0.00270	0.036	0.566	48869

Appendix 4 - Bibliography

Books:

D.F. Orchard, Concrete Technology, Galliard (Printers) Ltd., Great Yarmouth, 1979, Volume II, fourth edition

American Concrete Institute, Causes, Mechanism, and Control of Cracking in Concrete, ACI, USA, 1968

M.R. Rixom, Chemical Admixtures for Concrete, E & FN Spon, London, 1978

F.D. Lydon, Concrete Mix Design, Applied Science Publishers, London,1982, second edition

John Gould and Erik Nisses-Petersen, Rainwater Catchment Systems For Domestic Supply: Design, construction and implementation, Intermediate Technology Publications, London, 1999

Ivan Odler, Special Inorganic Cements, E & FN Spon, London, 2000

J.M. Illston ed., Construction Materials: Their nature and behaviour, E & FN Spon, London, 1994, second edition

S.B. Watt, Ferrocement Water Tanks and their Construction, Intermediate Technology Publications, London, 1978

A.M. Neville, Properties of Concrete, Longman Group, Essex, 1995, fourth edition

Company Publication:

Derek R. Brown, Modifiying Portland Cement Performance, Blue Circle Cements, Nottingham, 2001

Blue Circle, Rockfast Cements, Blue Circle Cements, 1995

Student Project:

Constantine T., 2001, Cracking in Waterproof Mortars, 3rd year project report, Warwick University, Engineering Department.

Web pages:

Dai Rees, [November 2000], DTU Technical Release Series – TR-RWH 01, Partially Below Ground (PBG) tank for rainwater storage – instructions for manufacture
www.eng.warwick.ac.uk/DTU/pubs/tr/rwh/rwhtr01/tr01.html [16/11/01]

Domestic roofwater research program – Components of a DRWH system [online]
www.eng.warwick.ac.uk/DTU/rwh/components.html [16/11/01]

11/07/2000
Company news [online]
www.lafargenorthamerica.com [12/12/01]

Patrick Groth, [November 1996], Steel fibre reinforced concrete
Epubl.luth.se/avslutade/0280-8242/95-37/html/paper_d.html [9/01/02]

Richard Austin

25/2/2000

(Ferrocement) Re. Fibers v metal mesh [online]

www.ferrocement.net/archives/msg02189.html

[18/12/01]

DEVELOPMENT TECHNOLOGY UNIT

Working Paper No. 32

Assessment of the Potential for Non-motorised Irrigation of Small Farms from Streams in Manicaland, Zimbabwe.

Development Technology Unit, Department of Engineering,
Warwick University, Coventry CV4 7AL
Tel: (0) 203 523122 Fax: (0) 203 418922 Telex: 311904 UNIVWK G

Working Paper No.32

Assessment of the Potential for Non-motorised Irrigation of Small Farms from Streams in Manicaland Zimbabwe

January 1991

Note: This Working Paper supersedes W.P. 31 dated October 1989

Acknowledgement: The DTU is happy to acknowledge the financial support of the UK Overseas Development Administration for the research reported in this working paper. (ODA Contract R4492: R&D of Hydraulic Ram Pumps).

This document is for circulation outside the D.T.U. For further copies contact:
Development Technology Unit, Department of Engineering,
Warwick University, Coventry CV4 7AL
Tel: (0) 523122 Fax: (0) 203 418922 Telex: 311904 UNIVWK G

APPENDIX C : PHOTOGRAPHS

A treadle-powered pump (right) and a plastic hydram pump (below) are shown under test at Chitepo Adult Training Centre, Bonda, Manicaland, Zimbabwe in September 1989.

The treadle is of University of Zimbabwe/Loughborough University design and the hydram of Warwick University design.

ASSESSMENT OF THE POTENTIAL FOR THE NON-MOTORISED IRRIGATION OF SMALL FARMS FROM STREAMS IN MANICALAND, ZIMBABWE

by members of
Warwick University Development Technology Unit
in cooperation with the staff of Manicaland Development Association.

CONTENTS

APPENDICES

Financial Note:
All costs are, unless otherwise designated, expressed in 1990 Zimbabwean dollars at a time when US$1 = Z$2.5 Development Technology Unit, Warwick University, Coventry, England.

1. INTRODUCTION

The intensification of agriculture in African countries, especially in the traditional sector of their rural economies, is receiving much attention today. This escalation of agricultural activity is seen as an essential means of achieving self- sufficiency in food production (despite rising populations). It is also seen as a way to increasing rural income generation, which would have the valuable effect of removing a prime cause behind over-rapid urbanisation. However, unless considerable care is taken, the intensification of land-use can lead to irreversible degradation of soils. Whilst simply improved farming techniques can do much to increase yields, a substantial improvement in agricultural production usually also entails increasing such inputs as the water, fertiliser and energy used in cultivation.

In Zimbabwe the productivity of most land is water limited. In very few areas can more than one rain-fed crop be harvested each year, and in much of southern and western Zimbabwe even that one crop is often lost in a dry year. Drought relief programmes are then operated. Seasonal malnutrition is also a concern. On large commercial farms, irrigation using water reservoirs or boreholes is widely practised with good results: the hectarage so irrigated has grown rapidly in the last 30 years. On small- holdings in the less fertile land of the Communal Areas, irrigation is, by contrast, quite rare despite being under- estimated in official surveys. The farmers (often women) have little capital, electricity for pumping is not available, the plots are too small or steep for commercial irrigation equipment to be economic and its maintenance can be a problem. For example a petrol-driven pumpset is too powerful, costly and complex for a farmer of 2 hectares and yet is not easily shared. Irrigation often conflicts with farmers' "minimum risk" strategies.

Part of Manicaland is mountainous and has the best rainfall in Zimbabwe. Much of the North and East of the Province is classed as "Agroecological Zone I" on the national map of "Natural Regions and Farming Areas", (a climate advantage which is offset by the steep and rocky nature of much of the land). Many of the streams are perennial. Despite this there is still a long dry season in winter when little rain falls and most cropland lies idle. Besides the three relatively large irrigation schemes operated by DERUDE, a little gravity-fed irrigation is practised in the Communal Areas, but there are very few pumps, long furrows or dams for seasonal water storage. Within the constraints of dry-season stream flows and other demands on surface waters, a substantial expansion of small-holder irrigation seems possible using stream water. Only at a later date will it be necessary to tap underground water, which is generally more costly to access and has special problems (like salination and depletion) in use.

The Manicaland Development Association (MDA) was formed in 1980 to assist rural grassroot communities with their social, economic and agricultural development. MDA has two Training Centres and District Development Committees in seven districts of Manicaland Province. It undertakes agricultural training and extension, it employs domestic technology instructors to improve nutrition and women's welfare and it promotes rural income-generation using appropriate technologies. Its main focus is upon the needs of those living in Communal

Areas. In relation to irrigation, its northern Herbert Chitepo Training Centre (HCTC) at Bonda is well placed for both demonstrations and extension work.

Warwick University has had connections with HCTC since its early days. The University has within its Engineering Department a Development Technology Unit (DTU) that specialises in rural industrialisation and the design of technologies for rural development in Asia and (especially) Africa. One device it has been researching slowly for some years is the water-powered "hydraulic ram pump" pump, seeking to modify traditional designs so that they might be made in small workshops at a cost that would permit their use for irrigation. Early prototypes were build at HCTC in 1986 and 1988. Recently the DTU has obtained British Government funding to accelerate its research into these pumps. The DTU has links with programmes using ram pumps in Zambia and Zaire.

The development of appropriate pumps is only one part of any programme to promote irrigation on smallholdings. There are many possible irrigation techniques (not all using pumps) and many factors affecting the viability of any one method in a particular area. A nine month period of pump testing and improvement at HCTC, involving the placement of a DTU representative there to work with HCTC's Appropriate Technology Section, was started in August 1989. It was decided to attach to this a short study of the relevance of non-motorised smallholder irrigation in Manicaland. The aims of the study carried out between mid-August and mid-September 1989 were:

- to identify major social, economic or geographical constraints on the expansion of irrigation of small plots in Communal Areas
- to develop a better specification for water-lifting devices
- to examine the agricultural options available for the use of extra water.

A further field study was undertaken in December 1990, mainly to clarify the likely area of irrigable land. By this time one of the pumps being tested at Bonda had run continuously for 9 months giving greater confidence in its suitability for irrigation applications.

The Working Paper describes the findings of two studies and records the initial test performance of two pumps (one water- powered, the other human-powered) installed near the Model Farm of CATC. The Paper is necessarily brief and simple, reflecting the brevity and simplicity of the studies. It has been written partly to give a framework to more detailed evaluation in the future, and partly to back up a request for irrigation programme funding that MDA is making.

2. IRRIGATION TECHNOLOGY

2.1 Choice of technology for smallholder irrigation in Manicaland

There are very many ways of supplementing rainfall to increase agricultural yields; irrigation is a well developed art. We are concerned here with a specific group of farmers in a specific area. Namely those living in Communal Areas of the moister parts of Manicaland. Existing irrigation of their dry season gardens is by gravity feed from streams, or is a secondary use of water primarily provided for domestic purposes. In this context it has been already argued that the expansion of irrigation using surface waters should take precedence over the introduction of irrigation using underground water from aquifers.

Of the various methods of applying water to a plot, those that are most efficient in water use - for example sprinkler and drip irrigation - are also those that need the most expensive equipment and highest water pressures. For both reasons they are not suitable primary techniques for the target group. Application by furrow, hosepipe or bucket has therefore been assumed, although in some places particularly permeable soils may require other treatment. Furrow irrigation needs considerable land preparation (of the same sort as erosion limitation on steep plots); a variant is free flooding from a top furrow, which however is inefficient in water use and can cause soil erosion.

Since dry season river flows are only a small fraction of Summer flows, seasonal storage behind small dams is common on large commercial farms. There are few examples of such reservoirs in Communal Lands, and in some parts of Manicaland suitable sites would be hard to find. The cost and degree of cooperation between individual smallholders necessary to construct and safely maintain dams is quite high: dam construction should not normally be a part of any self-managed irrigation schemes for small-scale farmers. There is a number of government-administered smallholder irrigation schemes in Zimbabwe where expensive shared facilities (dams, boreholes) are employed; however these schemes are usually subsidised to a degree that makes them poor models for copying.

Movement of water from source to plot can be by gravity or by pumping. Existing smallholder irrigation uses gravity, but there are many sites where the furrow leading from an abstraction point on a stream to irrigable land would be very costly to create and difficult to maintain. Long furrows, furrows through permeable soils (needing lining) and furrows through rocky terrain (needing blasting) are all likely to be too expensive. Contour furrows need protecting from damage or siltation by cross flows during rains. There is no doubt scope for increasing gravity-fed irrigation in Manicaland, sometimes in conjunction with the creation of very small (even temporary) dams to act both as diversion structures and as overnight water stores. Some of the gravity-fed sites now in use contravene the Stream Bank Protection Regulations, being within 30 metres of a watercourse: enforcement of these regulations is expected to tighten in future.

For a majority of new sites some form of water lifting will be required or will be preferable to the alternative of using a long furrow; it is specifically pumped irrigation that is considered in the rest of this report. In commercial irrigation, engine- driven pumps (the cheapest cost around Z$4000) or electric pumps are invariably used. These are usually uneconomic when irrigating less that 2 hectares. Mains electricity is rarely available at streamsides and photo-voltaic electricity is far too costly. Windpumping also has far too high a capital cost for irrigation pumping in Zimbabawe, even on sites with especially favourable wind regimes.

Three water-lifting techniques meet the requirement for low capital cost, low running cost and simplicity, namely human- powered, animal-powered and water-powered pumping. Animal-driven devices are not known or available in Zimbabwe, although they have been used in Asia for many centuries. They may become of more interest if there is an increase in the use of draught animals in Manicaland, but do not constitute a good option today.

Human-powered irrigation, at least of small plots, is widely practised. Applied to plots of even 0.2 hectares it becomes very arduous. Such a plot requires about 1000 large buckets (10000 litres) of water per day, about 4.5 buckets per minute for 4 hours per day. Even with 3 carriers, such a rate could only be maintained if the distance carried were less than 30 metres and the lift less than say 2.5 metres. Pumping is much preferable to carrying, as it employs energy far more efficiently. Of the many types of "hand" pump, those that make good use of back and leg muscles are superior to those only using arm muscles. It should be possible for a team of two people to raise 10000 litres per day through 5 metres using a human powered pump. In countries where human-powered irrigation is widely used, a lift of about 3 metres is regarded as a limit to viability. Higher lifts are however used in Bangladesh.

The last technique listed above is water-powered pumping. This is only feasible for lifting water from streams in hilly areas as it uses energy extracted from the small fall of a large flow to lift a small flow to a height. There are few water-powered pumps available. In China water turbine-pumpsets are manufactured, and a few other devices exist in particular countries, but the only widely used water-driven pump is the hydraulic ram pump. Even this is unsuitable for irrigation in its usual high-cost, high-lift, low- capacity form; however a variant designed specifically for low- lift irrigation in Africa is becoming available and is described below.

2.2 The hydraulic ram pump

Background.

The automatic hydraulic ram has been used successfully in rural areas for lifting water for nearly two hundred years. The hydraulic ram is a pumping device which utilises energy from the fall of a flow of water to lift a fraction of the supply to a much greater height. Because it has only 2 moving parts and no bearings, it has a reputation for reliability and low maintenance. By its very nature it cannot be used to pump "static" water from ponds or wells: it is restricted to hilly area where stream gradients are relatively steep.

Existing designs

Commercially manufactured ram pumps are usually made from steel castings and have to be imported to Zimbabwe. These are rarely used for irrigation because of:

- the high cost of the rams and large diameter pipe networks needed for irrigation
- the lack of experienced technical back-up to assist system design on site
- the unreliability of spares supply and access to maintenance.

The reasons are particularly true for small farms in remote areas. Secondhand Blakes ram pumps command high prices (Z$4000 upwards) in Zimbabwe today.

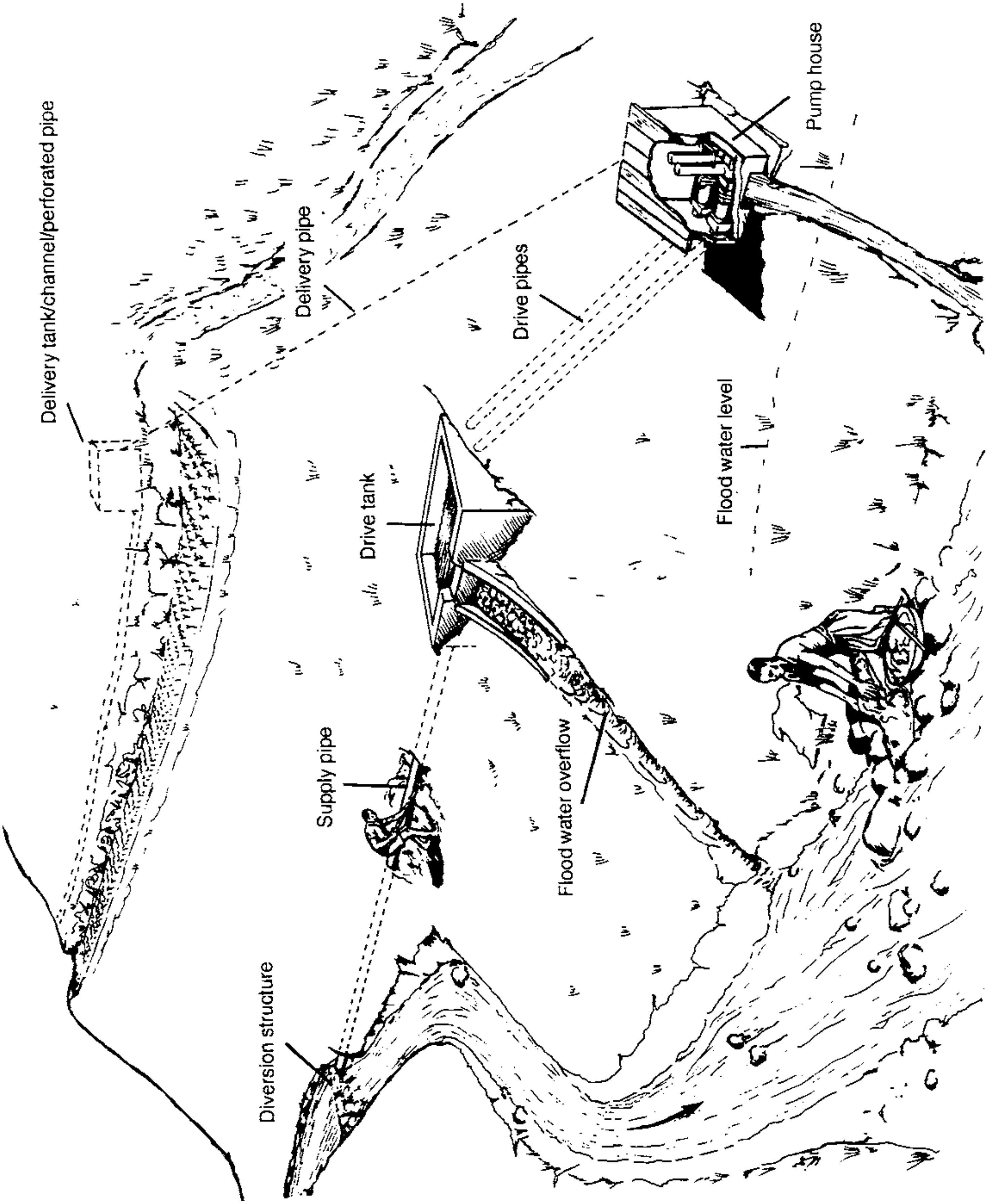
Warwick Design

The low-lift plastic ram pumps developed by the DTU are aimed at small-scale irrigation in Africa. The main design objectives are:

- to produce a pump which can irrigate, reliably, small farms at a height of 12m above a water course
- to make it possible to manufacture within the country of use, preferably in rural areas, using components and tooling techniques widely available
- to create employment in workshops with both manufacture and maintenance.

A hydraulic ram pump system (refer to diagram on the next page.)

The diversion structure channels water flow into a pipe or conduit which follows the contour (with a slight slope) away from the stream to the drive tank. The drive tank is at a point about 2m above the stream. Separate drive pipes bring the flow down the 2m "head" to drive the parallel pumps (between 1 and 4 pumps). A common delivery pipe takes the pumped water up to a delivery tank. The system can deliver about 7% of a stream flow at any single location. Another set of pumps at another location down stream could lift another 7% of the stream flow remaining and so on. This 7% maximum could be increased if the drive head were greater than 2m or the delivery head less than 12m.



Delivery tank/channel/perforated pipe
Delivery pipe
Drive pipes
Pump house
Drive tank
Flood water level
Supply pipe
Flood water overflow
Diversion structure

Performance
The characteristic of a particular (DTU) ram pump was found to be:

Delivery Head (m)	6.2	6.7	9.6	10.0	15.5	16.0	20.3	21.5
Delivery Flow (l/min)	17.0	15.5	12.0	11.3	7.0	6.8	4.6	4.2

The figures in the above table were taken from tests taken with a prototype plastic pump at HCTC, Bonda in September 1989 and are also plotted on the next page. The pump drive head was 1.2m and the delivery pipe had negligible friction. The pump was tuned for maximum delivery, not maximum efficiency, and so the average efficiency was only 23%, which is pretty low. Note how the delivery flow falls as the delivery head is increased. In practice, with the losses in 100m of 25mm delivery pipe, and 30m of conduit, these results would be more representative of a system drive head of 1.5m. From these figures it was predicted that in a standard configuration (2m system drive head and 12m delivery head with a single pump), about 17500 litres per day would be delivered, capable of irrigating about one third of a hectare in the dry season. Reconstruction of the demonstration system at HCTC in 1990 to fit these conditions has confirmed the prediction.

Siting
The siting of a ram pump is a very important part of the system design, affecting both the cost and performance. Some guidelines are:

- choice of a site where the water course falls reasonably rapidly - this will reduce the length of the conduit and make a larger drive head possible
- the flow through each pump should be at least 3 litres/sec (and preferably 6 litres/sec)
- the land for irrigation should not be more than 12m above the pump(s) and as close as possible to the pump system - reducing cost and headloss in the delivery pipe
- there should be some way of protecting the system from flood damage and/or theft - possibly by siting above the high water level, having capabilities for removal, or enclosing in some protective chamber.

Manufacture
Assistance will be required with site selection and installation by trained personnel whose travel would prove costly if they were not locally based. The pumps and spares should be made in- country to ensure the reliability of supply. There are advantages in integrating manufacture with installation because it allows for feedback from the user and a quick, reliable maintenance service. Further, it would be a form of rural employment generation outside agriculture which is greatly needed in Zimbabwe. So, if possible, the pumps should be manufactured locally.

This, however, has implications for the manufacturing techniques used because the numbers of pumps required are unlikely to warrant the purchase of expensive machinery. The system is therefore designed so that it could be manufactured using:

Materials

plastic (PVC) extruded pressure and drain pipe, and standard pipe fittings
standard steel bolts, rods and sheets
concrete

Techniques

use of basic metal and plastic cutting tools
use of a drill and thread cutting dies and taps
use of solvents
basic building skills

Cost

The components to produce one plastic ram pump (including foundations, drive pipe and drive tank) cost about Z$600 and 100m of 25mm diameter delivery pipe costs another Z$400. Allowing a further Z$500 for labour and for the (say 30m) conduit gives an estimated price of Z$1500 for a single pump installation. This includes no provision for storage of the 8 cubic metres of water pumped at night: a concrete tank this size might cost Z$3000 but a pond of this size much less. If several pumps are operated at one site, the cost per pump will be much lower: a 4-ram pump system irrigating 1.2 hectares might cost Z$4000 excluding water storage. Costs are somewhat site specific.

Development status

There are a number of test sites in UK run by the DTU, as well as the one at the Chitepo Training Centre. A workable design is at present being tested, however there is need for improvement and the design features to be improved are:

- durability
- simplicity of manufacture and use of alternatives to components scarce or unavailable in Zimbabwe (for example pressure elbows which have disappeared from the local market)
- overall pump system costs.
- Further areas of development will be:
- increased pump efficiency and performance
- wider sharing of expertise.

2.3 The treadle pump

Background

Broadly speaking, the treadle pump uses foot power through one or two rocking foot bars (or boards) to produce pumping power. The use of feet as the propeller means that the strong leg and back muscles are employed as well as the operator's body weight. Ergonomic research shows that the mechanical advantage over hand pumps can be up to 3 times. Because the treadle pump is human powered, it can of course lift from both static and running water sources.

Harare design

Following research into human-powered pumping by Loughborough University (WEDC) jointly with the University of Zimbabwe (Civil Eng), there have been demonstrations in Harare of both a treadle pump and hand powered "rope and washer" pump. For the reason outlined above the treadle pump is likely to be more appropriate for the large water needs of irrigation.

The Harare design:

- will pump to a maximum of over 15m, but only about 6m is suitable
- has adjustable mechanical advantage - allowing for the different size and strength of the operators and for different lifts
- is made of component easy to fabricate and so replace if needed is manufactured in country.

The pump works by the operator standing on 2 foot bars attached to 2 parallel piston-pumps which are operated by each leg alternately in a walking motion. There is a common suction pipe leading from the water and a common delivery pipe which would be used for direct irrigation, without storage.

Performance

In a test carried out to evaluate the usefulness of a treadle pump, compared to the traditional method of carrying a bucket, we obtained the following results.

	Treadle	Bucket
Man A	30.0 litres/min	9.6 litres/min
Man B	30.0 litres/min	12.0 litres/min
Average	30.0 litres/min	10.8 litres/min

This was for a lift of 6m and a carry distance of 40m, working for half hour periods and using 25mm delivery pipe and a 20 litre bucket respectively.

The ratio of efficiencies is 2.8 to 1 in favour of the treadle which (if it was operated for 4 hours in a day) would deliver 7,200 litres. The men involved in the test commented that the treadle was much easier to sustain, and once they were adapted to its motion, they could keep up say 50 minutes pumping in each hour. By contrast, they felt that they needed more frequent rests when carrying buckets. However it must be noted that in contrast to pumping, the effort to carry buckets depends more on distance than on height lifted. Thus the comparative advantage of the pump would decrease with shorter distances carried and increase with longer ones. The distance used in the experiment was only slightly greater than the 30 metres from a stream prohibited from cultivation by law.

Manufacture
The prototype used was manufactured in an informal sector fabrication workshop in Mbare, Harare, where they are in serial production. The sale price for a single pump is about Z$800 to which must be added the cost of a delivery pipe at about Z$4 per metre. No water storage tanks are required. The leather piston cups are bought in, being available on the market.

Development status
The pump is a working product, and a number have been produced in recent months. However it has not experienced extended field use and there is some scope for further improvement. Areas deserving attention include cost reduction, better piston sealing, reduction in the number of welds required for manufacture, addition of an intake filter and simplification of the method of adjusting the machine to users of different physical size (indeed the ergonomics of the pump merit further study and redesign). Research into these issues is under way at Warwick University.

3. AREA IRRIGABLE BY LOW LIFTS FROM STREAMS

Stream water, especially in the dry season, is a scarce resource; existing downstream users will not permit the emptying of streams for irrigation in upstream Communal Areas. An abstraction of 25% of dry-season flows might be an acceptable target for an irrigation programme, after which seasonal storage must be used.

Extensive hydrological records, extending over two or more decades, are available from the Ministry of Water Development which maintains over 20 gauging stations in Manicaland. The rainfall, and the dry season runoff per unit area, vary widely across the Province and even within a given agroeconomic zone. Peak monthly flows vary widely from year to year (although most gauging stations cannot record flows in major floods), but minimum monthly flows are more stable. At one site (Tindini Rodel Township) in 1982-4 for example, peak (February) flows were below 10% of their 10-year average but minimum (September) flows were 40% of their 10-year average. At this location, the September flow exceeded for 9 years out of 10 was about 2.2 l/s for each square km of stream catchment area. By contrast the Mare River near its mouth in Nyanga National Park has a September 90%-exceedance flow of 5.5 l/s per square km of catchment. This latter figure is untypically high, since the Mare River drains Zimbabwe's highest mountain, and its flow is regulated by three dams. We may assume that over the whole of the highland parts of Manicaland (including the Honde Valley) there is about 3 l/s of reliable dry-season flow per square km of catchment.

Using 25% of this 3 l/s yields a flow of 65000 l/day/sq. km, capable of irrigating about 1.3 hectares of land in Winter if applied carefully. On this basis, water availability would appear to limit irrigation to about 1.3% of the land surface. The fraction of the land which is cultivable is not know but is probably around 25%, on which basis some 5% of cultivable land might be irrigated.

In May, rather than September, this fraction might be doubled as mean river flows are about twice those in September.

Table of Mean Monthly Flows over 10 Years at Rodel Township
(Gauging Station DGP 27), expressed as l/s/sq.km of catchment

Month	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
Flow	4.0	4.2	7.2	12.7	20.5	15.8	8.9	7.0	5.8	4.7	3.9	3.4

(Minimum flow recorded in 10 years = 1.2 l/s/sq. km Maximum flow recorded in 10 years = 67 l/s/sq. km)

A more difficult question to answer is "*is 1.3% of the land area topologically suitable for irrigation?*" Suitability requires adequate depth of soil, modest slope (under 8%?) and elevation not more than say 10m above an adjacent stream bed. Subtracting "forbidden" land (within 30m of a stream) from the total, and acknowledging that many of the saddles and outwash aprons surrounding the granite massifs are too high, it seems that in much of highland Manicaland the availability of suitable plots may be as severe a constraint on irrigation as that caused by water shortage.

On a 1:50000 map with 20m contour spacing, a slope of 5% requires contours to be 400m apart, which is 8mm on the map; a slope of under 2% gives contours over 20mm apart. Examinations of map sheets 1832 B3 and B4, 1832 D1 and D2, covering in all about 3000 sq.km of North Manicaland, indicates very little land of under 2% slope. Less than 30% of the land is under cultivation perhaps half of which is slope 5% or more. This scale of map is rather small for such estimates and their confirmation requires more careful work on larger scale maps.

Stream gradients, even in the lower reaches of the Odzi, Pangwe, Honde and Nyadiri Rivers, generally exceed 1% (with valley side 4 to 10 times steeper). Thus conduits (furrows) to attain a height of 10m would be typically 400-800m long. By prudent choice of sites that correspond to rapids, feed pipes to give 2m head to a ram pump would be about 30m long. In general a lift of 5m is required to raise water clear of the immediate stream gorge.

A more detailed study of a particular Communal Area, namely the Honde Valley was made in December 1990. That part of the Honde basin that lies within Zimbabwe has an area of around 500 square kilometres. The valley is extensively cultivated except where slopes exceed 15%: there is little terracing. About half of the Valley is within five kilometres of a metalled road, however the nearest urban market is Mutare, some 70 kilometres from the centre of the area.

The Honde River is, along much of its length, too large and therefore costly to dam, so low-cost irrigation must make use of the water only in tributary streams. Irrigation using manual pumping (and hence no dam) is not generally feasible from this river, given the lift needed from its dry season level to any legally irrigable land. By matching observation (shortly after the first rains of the 1990/91 summer), resident's comments and map studies, it was concluded that only at about four kilometres from its source was any tributary likely to have a usable perennial flow. As a general rule, therefore, water is both available and economically extractable from those parts of streams between 4 kilometres and 15 kilometres from their sources: there is a total length of 40 kilometres of streams satisfying this condition in the Honde Valley.

A head of two metres to drive a ram pump is obtained in part (say 0.6 metres) by constructing a small dam and in part by taking advantage of the natural fall of a stream. Assuming an economic ceiling of 35 metres on drive pipe/feed pipe length, and a local slope at the pump site of three times the average stream slope, gives a minimum "average stream slope" of 1.3% for the installation of water-powered pumps. All the 40 kilometres of stream already shortlisted has a slope exceeding this value.

A stream can also be too steep for water-powered irrigation in that a high slope favours irrigation by gravity via a contour canal. Taking 10 metres as the height above the stream bed to command a useful amount of land that is both above the stream's immediate gorge (typically 5 metres deep) and a legally adequate distance from the water (30 metres), the length of an irrigation canal must exceed 10 metres/streamslope. At stream slopes exceeding 4% (14 kilometres out of the 40 kilometres identified) such a canal would be only 250 metres long and would probably be cheaper to build than an equivalent hydraulic ram pumping station.

There are already a few irrigation canals fed by tributaries on the north side of the Honde River. These tributaries descend a high escarpment, two of them having dramatic waterfalls visible from 10 kilometres away. The one canal investigated (Makanga Valley) carried 1.3 litres per second - enough to irrigate about two hectares. Another canal is reported to be considerably larger. Canal building is not however a common skill and on many otherwise favourable sites hard rock requires blasting or cutting if contour channels are to be created. Once clear of the immediate stream gorge channels can run in softer ground but these suffer from considerable seepage. The present few channels are unlined, but using fibre-cement techniques now being employed for roofing in Manicaland, lining might be economically achievable.

On the south side of the Honde River there is at least one example of irrigation water being piped from a spring. Suitable pipe is now very expensive in Zimbabwe (eg. Z$1600 for 400 metres of 40 mm outside diameter) and such irrigation is only likely to be economic on especially favourable sites where the pipe slope can be maintained at over, say, 8%.

Valley sites in the study area are very variable in terms of stream slope, the depth of the flood gorge and the slope of cultivable land above the flood gorge. About 3% of valley sides are suitable for manual irrigation having shallow gorges and cross slopes under 5%. Another 30% of valley sides appear suitable for ram pump or contour canal irrigation, having gorges not over 5 metres deep and subsequent slopes of 5 to 10%. For both cases each irrigable slope above a gorge is likely to be about 60 metres wide. In total therefore, there are about 4 irrigable hectares per kilometre of suitable river, giving a Honde Valley total of 160 hectares, which is 0.3% of the Valley's area.

River flow records are not available in the Valley, but using the flows derived earlier for adjacent areas, there is ample water for 1.3% of land to be irrigated. This compares with the figure of 0.3% apparently suitable terrain in the Honde Valley. Choosing the lower of these figures and applying it to the total area of Communal Lands in Manicaland lying in Natural Regions I or II (high rainfall) yields a total of about 1,000 hectares of irrigable land, or ten times that number of typical 0.1 hectare gardens. In the short term various factors may reduce this figure by up to 60% and a range of 400 to 1,000 hectares potential may be used. In the long term extensive construction of contour canals could double or treble the irrigable areas.

4. PATTERNS OF IRRIGATED AGRICULTURE

4.1 Dry Versus Wet Season Agriculture

This part of the study attempted to find out how small-scale irrigation usage would effect agricultural practices within the villages visited in N.E. Manicaland. A total of 5 villages were visited all within the Honde Valley region on the 29th August 1989 and 30th August 1989: Hambira, Matingo, Gatsi, Dombomupunga and Nyanguzu. Hambira, Matingo and Gatsi are in the Samaringa Area. Dombomupunga is the Sahumani Area. Nyanguzu is in the Nyasanza Area.

From the information collected it would appear that dry-season irrigation would be the most desirable. This period extends from April to October which is also the winter period. From an irrigation viewpoint this seasonality is quite convenient as it means that irrigation water is required when the evaporative demand from the crops is comparatively low.

There are 3 constraints effecting land which can be irrigated by the hydraulic ram pump or other non-motorised pumps. These are water availability from rivers, the height of land above the river and the economical return that can be obtained from the crops grown on the land. Since the amount of irrigation water made available by each pump is relatively small, enough to irrigate approximately 1/3 ha of land at a height of 10m depending upon the evapotranspiration rate, valuable cash crops would be most appropriate to grow. The Gatsi farmers quoted in 1989 that Z$2000 gross income could be earned per hectare of land if vegetables are grown. The farm manager at CATC observed that at 12 tonnes per ha per crop and prices of 35-40c per kg, for tomatoes or cabbages, 1 hectare could gross Z$4800 per crop of Z$14400 per year. These very different 1989 figures may reflect differences in access to markets. In 1990 a group of women in the Honde Valley (which is warm in winter) estimated the dry season income from a small (20 metres x 20 metres) garden as over Z$5 per day, corresponding to Z$25,000 per hectare per season. These figures are affected by 50% inflation over 16 months. A late 1990 gross additional income of at least Z$16,000 per hectare per year seems obtainable if irrigation is introduced.

Besides the possibility of dry-season cultivation there are three options for wet-season cultivation:

> **Option A** would be to increase cotton or maize crop yields by planting seeds just before the wet season started. Compared to dry season irrigation, under option A it may be possible to irrigate a larger area of land, say 1/2 ha. per pump since seeds for the first 30 days after germination require smaller amounts of water. There is evidence that irrigation in October just before the rains could almost double maize yields, earning Z$400 extra per hectare. Both dry-season irrigation and option A have limits upon the amounts of land which can be irrigated because of low flows in rivers before the rains.
>
> **Option B** is to extend the wet-season cultivation of lower value staple/cash crops. This is possible because in April and May, immediately after the rains, water in the rivers is still fairly plentiful. This option would be more costly since more pumps would need to be installed to meet the water demands, however larger areas can be irrigated than under the other options.

Option C is using the pumps as an insurance policy for the cultivation of staple crops during drought years. Possibly 1 year out of 10 does not receive enough wet-season rainfall for crop survival. Installing costly facilities for use only once or twice a decade is rarely cost effective compared with other forms of drought insurance.

Generally speaking it would not seem viable to install a pump for maize or cotton crop use alone, as assumed in options A, B, and C. This is partly because these crops are of much lower value than vegetables. Also these crops are usually grown some distance away from the rivers at a height greater than 10m. Once a pump was installed however its usage could be extended to options A, B, or C, providing all constraints were met.

4.2 Nutrition and Food Security

Since maize is the only storable crop, in the absence of either irrigation or purchasing power, diet would be restricted to sadza, although there is some tradition of boiling and drying of vegetables. In discussions most families said that they do purchase vegetables to improve their diet during the dry season and their interest in irrigation was in significant part to replace these purchases with their own produce.

In N.E. Manicaland serious drought resulting in loss of crops is fairly rare. It seems unlikely a significant area could be irrigated in a drought year when rivers are particularly low. As discussed earlier the economics of irrigation only used in occasional years are particularly poor and other forms of drought relief in these areas would generally be cheaper. Of course in S.W. Manicaland the situation is quite different and drought relief irrigation systems might be used one year in two.

4.3 Maintaining Fertility

All the farmers visited used artificial fertilisers. This was because on average each household owned only 5 cattle, whilst some owned none. Bags of fertilisers were bought from MDA in 1989 at approximately Z$26 per 50 kg bag however subsequent inflation has been substantial. Normally 5 bags of fertiliser are used for 1 hectare of maize - 3 bags of compound D containing amino- nitrate, potash and potassium nitrate and 2 bags of amino- nitrates. An increase in crop production would mean that a) more nutrients would be taken from the soil and b) the land would not be left fallow for the soil to replenish itself. The common answer to the problem appears to be "add more fertiliser".

There is concern that pumping so much fertiliser into the soil is reducing the organic component of the soil which in turn endangers the soil structure. Environment and Development Organisation in Africa (ENDA) is presently researching methods to increase the organic element. Organic matter other than manures are crop residues, specifically grown compost and soil from elsewhere (eg. anthills). At present the management of dung and crop residues is rather poor. In other parts of Africa where land pressures are higher, composting, zero grazing of animals etc. are used to supplement or replace artificial fertilisers. It would be unwise to rely wholly on artificial fertiliser to provide the extra fertility required when irrigation and multi- cropping is extended. Pesticides are not generally used in Communal Areas because they are both hazardous to health and expensive.

4.4 Soil Erosion

On several of the sites visited, cultivation was being carried out within 30m of the river bed on slopes. Under the "Streambank Protection Regulation" this action is illegal. This Regulation is particularly concerned with reducing siltation in rivers which makes water unavailable to households downstream. Elsewhere, the problems of soil erosion on steep slopes is more a concern of top soil fertility loss which the Regulation does not cover.

From 1989 we were told that the law would be enforced more strictly; the penalties are a Z$25 fine plus the destruction of the crops grown. Farmers are reluctant to give this land up from cultivation, partly because it is an important source of their income, partly because it is so entrenched in their culture and partly because of a lack of realisation of the destruction it causes downstream. However agricultural extension workers have been informing communities of how to prevent unnecessary soil loss. We saw several examples of terracing on the steepest slopes which indicates people's efforts.

Grass leys and furrow ridges were other methods used on more gentle slopes. Grass leys which are grown in contour sections are adequate barriers to soil movement on a slope of 2%. On steeper slopes of 3%, 4% and 5% furrow ridges are needed.

For a 10m lift of water, it seems likely that the land at this height will still be part of the steep valley sides assuming that most river valleys are "V" shaped which is indicated on the survey maps of N.E. Manicaland. If this is the case, soil erosion protection will need to be applied. Agricultural experts are available to mark out the contour lines, but then much person power would be needed to build the furrow ridges. Wild flooding appears to be the type of irrigation known and used in N.E. Manicaland. Water is delivered at several points from a supply channel running along the upper edge of a sloping plot or field and is allowed to move freely down the slope. Its popularity is no doubt due to its simplicity. It is a system involving a minimum amount of land preparation and does not require much labour in operation. However it is dangerous to use it on light soils that are prone to erosion. Also it does not distribute water uniformly, which means extra water is required to ensure that all points receive the necessary amount.

The best way to apply water, especially in situations where the water supply rate is low, for example 20 litres per minute, needs further thought and experimentation. Very careful application of suitable irrigation methods is required on sloping plots.

4.5 Economics and Evidence of User Interest

The foregoing discussion has indicated an extra gross income per hectare, attributable to irrigation of some Z$16000 (in the form of produce sold or in food retained by the cultivator's family). This income has to pay for the irrigation itself, for other inputs and to provide a recompense for the farmers' extra labour. It seems acceptable to assign 50% of this income to payment for the water supplied - Z$8000 per hectare per year. As non- motorised irrigation costs are dominated by the capital costs of initial provision rather than running costs, and as the different components of a pumped irrigation system have a life of between 3 and 15 years, a simple payback period of 3 years has been assumed.

Combining these assumptions yields a ceiling for the capital cost of creating an irrigation system of Z$24000 per hectare. Estimates in section 2.2 and 2.3 suggest that for favourable sites it should be possible to keep well below this ceiling. Overnight storage of irrigation water may create a greater income by using it for fish farming.

According to local farmers there was very strong interest in irrigation being applied on their land. In those places in the Honde Valley, eg. Hambira Village, where gravitational systems were already installed, peoples' initial interest was to expand these. Concerning pumps, they wanted assurances that they would do their job, could be maintained and repaired when they failed and that they would be able to pay back a loan borrowed for their initial purchase over several years. In 1989 the Agricultural Finance Corporation lent money at an interest of 13%. Because this rate was seen as high, farmers were reluctant to take up loans. Interest in irrigation is stronger in 1990 than a year earlier due to most farmers being under greater pressure to generate new income.

5. LEGAL AND SOCIAL IMPLICATIONS

The re-use of domestic water for small gardens is regarded as "Primary" water use. In practice very small-scale irrigation is also unofficially treated as primary water use. Licences are effectively not required for scattered small-scale irrigation of gardens. Different unofficial opinions suggested a ceiling of 1/2 ha. on unlicensed irrigation. In terms of an irrigation programme for Communal Areas, there will come a stage when water abstraction is significant, eg. 10% of flows, at which point licensing probably will be required. Although small farmers no doubt have the political sympathy of the licensing authorities it will be important that they are not thought to use water wastefully.

Income for a typical farmer varies. At present in a good year, she could earn Z$1000 from her farm plus a little more from craft work (eg. crocheting) in Winter. Many of the men work in the towns and can earn Z$2000 per year. However because they require lodgings in the city they may send back only Z$700 per year. The cost of living is going up dramatically, without a corresponding income rise. Because primary education is free, children still go to school, but not all go to secondary school which costs about Z$200 per year per child and is about to rise sharply. Secondary schooling lasts for 4 years from age 14 to 18 and most families can only afford secondary education for 2-4 children.

In all the villages we visited, we spoke only to the women, because they are the farmers. They were all keen to grow another crop during the dry season. The women at Hambira mentioned that there was usually less work to do at this period. This was despite the fact that many had small winter gardens close to their houses, watered by gravity or by bucket. Time is spent collecting firewood and doing limited amounts of gardening. An extra crop would increase their workload but also time would be saved for those who now have to carry buckets of water from the river. They did not see the extra workload as a limiting factor because they would simply employ their family in the fields. A typical family has 7 children, which is beneficial in terms of labour supply. The main ambition of most farmers is to be able to send more of their children to school. Since agriculture is their primary source of income, this indicates the importance of creating more income form irrigated crop production. The women said that they would sell extra crops within the local community since there is no transport to take produce to a bigger market. The local market is best when the schools are open because farmers can sell their produce to the schools and to the school teachers.

For a modest increase in smallholder irrigation neither legalities nor work loads appear to be major constraints. Improvements in marketing, especially in terms of transport, will be required if there is a substantial increase in the production of Winter vegetables.

6. SMALLHOLDER IRRIGATION DEVELOPMENT PROGRAMME

The Manicaland Development Association has for some time wished to respond to the requests of its member groups for help with irrigation. Small-holder irrigation is a specialism in which few are expert, and techniques need careful adaptation to local conditions. If MDA is to mount a major irrigation promotion programme, probably lasting 5 years, that programme is likely to require the following components:

a) The identification and refinement of suitable irrigation water-supply techniques for use in Communal Areas.

b) The identification and refinement of agricultural techniques to make the best use of irrigation water.

c) The identification, in much greater detail than in this study, of user priorities, of physical and legal constraints, of markets for dry-season produce and of favourable specific locations.

d) The creation of an irrigation construction team able to survey, assess, design, procure parts for and oversee the construction of small irrigation schemes by MDA member groups. This may be backed by a workshop making pumps, assistance in marketing and help in obtaining water licences.

e) The establishment of a training and extension programme to disseminate the findings of the research components above, especially with respect to:

- agriculture
- maintenance and operation of irrigation schemes
- (later) construction of new systems.

A good case can be made for incorporating into the irrigation programme a fish-farming component. The two activities can often be profitably combined, sharing some common facilities. Increasing the production of fish protein for sale or local consumption is another of MDA's long term interests.

7. CONCLUSIONS

Within the limitations of a brief study it appears that there is considerable scope for the expansion of dry-season (Winter) irrigation of small plots in the Communal Areas of Manicaland using water from small rivers. It is likely that similar conclusions hold for the eastern side of south Manicaland. Using a variety of assumptions it appears that there is enough water to irrigate about 1.5% of the land without requiring seasonal storage of water behind dams. However only 0.3% of land is topologically suitable, which gives a figure of around 1000 hectares in the Communal Areas of Manicaland having good rainfall.

Economically a ceiling of about Z$24000 per hectare has been identified on the capital cost of water provision. Dry-season irrigation appears economically superior to supplementary irrigation just before or after the rains, or to "insurance" irrigation for drought years.

Two suitable pumps have been tested in prototype form. A ram pump system, possible cost Z$1500, lifted enough water in 24 hours to apply 5mm of water to 0.35 hectares of land 10 metres above the headworks, providing a drop of 2 metres was available to drive it. In commercial form, with overnight water storage for each 0.1 hectare garden, multiple-ram pump systems would cost some Z$8000 per hectare. A treadle pump system tested against a lift of 6 metres and a distance carried of 40 metres raised 72000 litres in four hours, enough to irrigate 0.15 hectares: this was 2.8 times greater than raised by the same people in the same time using buckets.

No major legal or social barriers to expanding irrigated gardens were identified and farmers interviewed expressed considerable enthusiasm to expand their incomes in the way. Changes in marketing facilities, especially transport, will probably be needed if dry-season vegetable production is much expanded.

A 5 year programme for MDA to help its member groups expand irrigated agriculture has been outlined.

APPENDIX A:

HYDRAULIC RAM PUMP PERFORMANCE ANALYSIS

In section 2.2 some hydraulic ram pump test results were given. These were obtained in 1989 from the prototype at HCTC. Facilities for the measurement of input water flows (and hence input power) at the test site were very crude, but indicated a drive flow of 6.3 l/s or a little less. Combined with a system drive head of 1.42m, this corresponds to an input power of 88W. On this basis, the results can be restated as follows:

Viable	**Units**								
Lift	m	6.2	6.7	9.6	10.0	15.5	16.0	20.3	21.5
Flow	1/min	17.0	15.5	12.0	11.3	7.0	6.8	4.6	4.2
Flow	1000 l/day	24.5	22.3	17.3	16.3	10.1	9.8	6.6	6.1
Output power	W	17.2	16.9	18.8	18.5	17.8	17.9	15.3	14.7
Efficiency	%	20	19	21	21	20	20	17	17
Pressure swing	%	16	20	13	10	6.5	6	3	3.5

From these figures, the following conclusions may be drawn:

a) The efficiency is very low but fairly constant; the pump adapts to changes in delivery head such that output power is inversely proportional to head.
b) At output heads greater than 20m (corresponding to pressures over 2 bars) performance falls off. In fact the pump durability is unacceptably low at such high pressures.
c) Extrapolating from the test results, a pump lifting water 12m via a delivery pipe having a 1m friction loss should deliver 12500 l/day, or 17500 l/day if the system drive head were increased from 1.42 to 2.0m.
d) The pressure swing figures indicate that for large-flow, low-head applications the pressure vessel is too small.

The low efficiency could be significantly improved, at the cost of a small loss in output, if the pump were retuned to run faster, eg. at 60 rather than 30 strokes per minute.

A separate analysis of individual losses indicated:

the exhaust flow kinetic energy accounts for 21% energy losses
the drive pipe friction " " " " 6% " "
the conduit friction (30m x 110mm pipe) " 15% " "
back-leakage through the delivery valve " 4% " "

This leaves 35% losses to be attributed to impulse-valve leakage, to hydraulic friction in valves and other causes. The percentages above apply to a delivery head of 10m, however only the back-leakage component would be greater at higher heads. Loss reduction requires either better fitting parts or the use of a larger ram pump; it is of importance only where it is desired to abstract a significant proportion (say over 5%) of total stream flow at a single site.

Concerning durability, tests indicate that when delivery heads exceed 15m, the pump is very vulnerable to loss of air in its pressure vessel. Absence of this air results in hammering and the fatigue of components within a few hours. An overpressure relief valve is clearly necessary to protect against this.

Tests also showed that air-control needs further attention. Too much air (gulping) reduces output, whereas failure to draw in any air at all results in the gradual depletion of air in the pressure vessel. Provided the pressure-vessel air is periodically replenished, the pump will operate satisfactorily under water.

Further tests with a modified design in 1990 indicated similar power outputs despite much smaller drive flows. System efficiencies of 40% were obtained, corresponding to pump efficiencies of over 60%.

APPENDIX B:

SHORT BIBLIOGRAPHY

Bell M, Faulkner R, Hotchkiss P, Lambert R, Roberts N & Windram A The Use of Dambos in Rural Development with Reference to Zimbabwe Loughborough University (UK) & University of Zimbabwe (ODA Project R3836) 233p report on micro-irrigation, Nov 1987.

Blackie M.J. (ED) Proceedings of African Regional Symposium on Smallholder Irrigation University of Harare & Hydraulic Research Ltd (Wallingford UK), Sept. 1984 pp 1-67 comprise general papers on appraisal, organisational roles, health and human resources; pp 77-435 comprise papers from technical sessions and include: - 4 papers on design of small-scale schemes in Zimbabwe pp 87- 114, - Balarin J.D., Fish Farming and Irrigation pp 151-166, - Rakuni M., Cropping Patterns and Productivity (in Zimbabwe) pp 379-390

Carruther I. & Clark C. The Economics of Irrigation Liverpool University Press, 1981

Chakroff M. Freshwater Fish Pond Culture and Management Peace Corps/Volunteers in Technical Assistance (USA), Rev Ed Arlington 1978

Fraenkel P. Water-Pumping Devices, A handbook for users and choosers IT Publications for FAO, London 1986

Manicaland Development Association Annual Report and other documents

Rukuni M. State of the art of Irrigation in Zimbabwe - Historical Bibliography Zimbabwe agric J., vol 82, pp 85-90, 1985 The Evolution of Smallholder Irrigation Policy in Zimbabwe, 1928-1986 Irrigation & Drainage Systems 2, pp 199-210, Kluwer, Dordrecht, 1988

Stern P.H. Small Scale Irrigation IT Publications (London) & International Irrigation Information Centre (Ottawa), 1979

University of Warwick, Development Technology Unit Annual Report 1989-9, DTU Product Data Sheet 2, High-lift Steel Hydraulic Ram Water Pumps DTU Product Data Sheet 3, Low-lift Plastic Hydraulic Ram Water Pumps

Watt S.B. A Manual on the Hydraulic Ram for Pumping Water IT Publications, London, Rev Ed 1978

DEVELOPMENT
TECHNOLOGY
UNIT

Working Paper No 33

Comparison Between DTU and Commercial Hydraulic Ram Pump Performance

Development Technology Unit, Department of Engineering,
Warwick University, Coventry CV4 7AL
Tel: (0) 203 523122 Fax: (0) 203 418922 Telex: 311904 UNIVWK G

Comparison between DTU and Commercial Ram Hydraulic Ram Pump Performances

UPDATE AND COMMENTS, APRIL 1996

Since this paper was written, the DTU Mark 6.4 pump has been phased out, to be replaced by first the M8.4 and more recently the S2. All these are pumps normally run with a 2" G.I. drivepipe and delivering to up to 100m using a drive flow of 40 to 120 litres/min. Preliminary results for the M8/S2 on a single setting are given at the end of this paper, indicating that, setting for setting, the M8 is superior to its predecessors. Generally the DTU later models are more efficient than the M6. Of course performance as measured in this paper is only part of the story. The DTU pumps are made largely of mild steel and are therefore more subject to corrosion than most 'commercial' machines. In recent years there has been a trend towards use of plastics in ram pumps as pvc or ABS piping replaces galvanised iron. However plastic pumps can rarely operate reliably at delivery heads exceeding 40 meters. There is also a growing interest in providing the air-cushioning of the output via an enclosed air packet (for example closed cell foam) instead of an air vessel with a free air-water surface. This arrangement can effectively increase the usable drive head by allowing the impulse valve exhaust to emerge under water.

In the paper, the early comparisons with commercial pumps use data from a study by T H Delft which were obtained from slightly larger models (from the respective manufacturers' ranges), and with a much higher drivehead, than the comparisons in the rest of the paper.

Details of the current (1996) DTU designs, namely S1 (for use with a 1" steel drivepipe), S2 (2" steel) and P90 (90 mm. pvc) are given in DTU Technical Releases TR11, TR14 and TR12 respectively.

CONTENTS

T.D.Jeffery
April 1991

List of Tables

Page No.

List of Graphs

List of Figures

1. INTRODUCTION

This paper details the performance of the DTU Mark 6.4 hydraulic ram pump in comparison to commercial models run under similar conditions. Details of the performance of a number of commercial pumps tested at Delft University, Netherlands are used for comparison.

1.1 Delft University

Between 1982 and 1984 J. Tacke of Delft University of Technology carried out tests on a number of commercially available hydraulic ram pumps. These were published in 1988 with comprehensive details of all results obtained and the development of a mathematical model for prediction of ram performance. As part of this programme field tests were conducted by the Foundation of Dutch Volunteers in Rwanda. These aimed to investigate the technical performance and durability under operating conditions in a community setting, social acceptance and community participation in installing, operating and maintaining a hydraulic ram system.

The presentation by Delft of all laboratory test results is excellent and allows a good level of comparison for tests on subsequent designs. Unfortunately no details of the findings or conclusions resulting from the field trials in Rwanda are made available. However the DTU are greatly indebted to Delft for their provision of such a useful resouce.

1.2 DTU, University of Warwick

The Development Technology Unit has been investigating hydraulic ram pump design, performance and manufacture since 1985. This began with student projects and has grown into a full-time research programme largely funded by the Overseas Development Administration. The aims of the programme are:

a) to analyse in detail the operation of the ram pump gaining a comprehensive understanding of the operating principles and complex hydraulic interactions occuring within the pump.

b) to produce pump designs suitable for manufacture in developing countries using available materials and production processes.

c) to thoroughly test such designs for their performance and endurance (including extensive field trials) and offer findings for widespread dissemination.

d) to develop and prove methods for surveying and design of complete water supply installations.

e) to produce design charts and computer based tools to enable design and field engineers to confidently include hydraulic ram pumps as an option in their water supply schemes.

f) to provide technical expertise and training for two African based programmes installing ram pumps for village water supply and irrigation.

In terms of hardware development the DTU has two distinct working areas. The first is the development of designs of steel hydraulic ram pumps based on the 2" diameter BSP pipe that has been found to be widely available in developing countries and is of an appropriate capacity for small village water supply schemes. The second area of hardware development is the production of plastic hydraulic ram pumps based on widely available 110mm plastic pipe and especially suitable for irrigation close to water courses. Rough specifications and performance indications are given below.

	Steel	Plastic
Materials required		
	2" Galvanised pipe	110mm PVC pipe
	2" Fittings	Small amount of
	Mild steel bar	mild steel parts
Manufacturing process		
	Welding	Hand tools
	Drilling	
	Turning	
Typical performance ranges		
	40-140	
Drive flow	40/140 l/min	200-350 l/min
Drive head	2-25m	0.5-2.5m
Delivery flow	2-12 l/min	2-25 l/min
Delivery head	up to 150m	up to 15m
Efficiency	50-90%	20-60%
Expected life	10 years	3 years

This paper is concerned solely with the comparison of the 2" steel pump with its commercially available rivals. Such pumps have found the widest application to date in supplying water for domestic use.

During the research many designs of ram pump have been produced and tested. In 1990 a design was chosen as having proven itself sufficiently in terms of performance and durability to set a 'benchmark' against which furthest developments could be assessed. It is this model, Mark 6.4, that is used in the following comparison against commercial designs.

2. SUMMARY OF WORK CARRIED OUT AT DELFT

2.1 Selection of Pumps

Delft decided to select 12 rams from 6 manufacturers that were applicable in typical village or domestic water supply schemes. In all, details of pumps from 10 manufacturers were obtained and selection made based on the following criteria:

a) as many types of ram design as possible should be included.

b) tests should include both traditional and modern designs.

c) rams should show a reasonable price to performance ratio.

To enable the latter of the criteria, manufacturers were sent a set of conditions and asked to provide details of the pump they would recommend and its expected output and efficiency.

Table 1 shows the results of this comparison.

TABLE 1 - COMPARISON OF HYDRAULIC RAMS

Arrangement: Source Supply = 90 l/min

Supply Head Hs = 7.50 m

Delivery Head hd = 75 m

Type of Hydraulic Ram	Drive [inch]	Pipe [mm]	Volume of Driving Water Required [l/min]	Pumping Rate q [l/min]	Efficiency trd [%]	Approx Price for Ram Alone [US$-1982]
Vulcan 21/2	2	65	36-114	6.10	68	1200
Blake Hydram 31/2	2	65	45- 96	6.00	67	1000
Sano No. 5/65 mm	2	65	50-110	6.95	77	1000
Rife 20 HDU	2	50	38- 95	5.40	60	1100
Schlumpf 5A23	2	50	50-100	5.50	61	2700
Alto CH 66-110-18	2	65	50- 90	5.40	60	2100
Briau D4	2	50	45- 90	5.40	60	3200
CeCoCo - H50	2	50	25-115	6.90	77	3500
WAMA No. 6	2	65	60-100	4.50	50	1500
BZH-Ram W6	2	65	45- 90	2.70	30	1200

Delft concluded that in terms of performance, the choice should favour the first eight pumps and of these the first six provide the best performance to price ratio. From the information given the choice seems to be a reasonable one.

An alternative comparison to evaluate the performance to price ratio would have been to include a cost per litre delivered under these conditions. This significantly alters the ranking of pumps as is shown in Table 2.

TABLE 2

Type of Hydraulic Ram	Pumping Rate q [ℓ/min]	Efficiency (Trade) (%)	Approx Price for Ram Alone [US$-1982]	Cost per litre delivered	Rank
Vulcan 2½	6.10	68	1200	197	3
Blake Hydram 3½	6.00	67	1000	167	2
Sano No. 5/65 mm	6.95	77	1000	144	1
Rife 20 HDU	5.40	60	1100	204	4
Schlumpf 5A23	5.50	61	2700	491	8
Alto CH 66-110-18	5.40	60	2100	389	6
Briau D4	5.40	60	3200	593	10
CeCoCo - H50	6.90	77	3500	507	9
WAMA No. 6	4.50	50	1500	333	5
BZH-Ram W6	2.70	30	1200	444	7

It would also have been interesting to include details from rams currently manufactured in developing countries. Details of the rams selected are given in Table 3.

2.2 Assessment of Experimental Procedure

The tests conducted were fairly comprehensive with each ram being observed for approx. 1 month. The test rig employed allowed a supply head of between 0.5 and 3.0 m with a drive pipe length of 12m. Such a low drive head range is clearly the product of laboratory limitations and does not reflect typically observed ranges in the field of 2 to 20m.

Flows were measured by collecting water for a timed period and weighing it. For each setting this was repeated a number of times to give a more accurate average flow. This is potentially a very accurate method of flow measurement but is open to numerous experimental errors. However it does avoid the need to use flow meters that have their own inaccuracies despite careful calibration.

The designs of ram pump tested exhibit a number of different types of impulse valve. Four of the six have traditional valves above the axis of the pump body but the Schlumpf has its valve below the axis. No mention is made in the text of the Delft work as to where the supply head was measured from. To allow accurate comparison the head should be taken from the orifice of the impulse valve, not the axis of the pump body. In reality both the SANO and Schlumpf pumps may in some situations be able to utilise a given supply head more effectively by their impulse valve design.

TABLE 3 - HYDRAULIC RAMS SELECTED

Type of Hydraulic Ram	Manufacturer	Drive Pipe Diameter [ins]	[mm]	Intake capacity [l/min]	Description
Blake Hydram No. 2 Blake Hydram No. 3.5	John Blake Ltd England	1.5 2.5	40 65	12-25 45-96	A well-established standard design made of cast-iron. Both waste valve and delivery valve consist of a rubber disc covering a perforated gunmetal seat.
Alto J 26-80-8 Alto CH 50-110-18	J.M. Desclaud France	1 2	25 50	8-15 30-60	A recently renewed pump design made of steel pipe components, but using conventional valve designs. Weight-loaded gunmetal waste valve; spring-loaded rubber clack as delivery valve.
Vulcan 1" Vulcan 2"	Green & Carter England	1 2	25 50	4-18 23-46	A standard design made of cast-iron; like the Blake Hydram available for a long time. Both waste valve and delivery valve consist of a rubber disc covering a grid shaped, gunmetal seat.
SANO No. 1-25 mm SANO No. 4-50 mm	Pfister + Langhanss Germany	1 2	25 50	6-16 30-65	A rather unconventional' design, nowadays made of fire zinc-coated steel. Both waste valve and delivery valve are spring-actted and substantially made of gunmetal.
Davey No. 3 Rife 20 HDU	Rife Hydr. Eng Mfg. Co.	1 2	25 50	5-15 38-95	Rife: a fairly standard design made of cast-iron. Weight-loaded rubber waste valve, mounted on a rocker-arm; delivery valve is a rubber disc covering a grid iron seat. Davey: less efficient, less expensive low base configuration, using a weight-loaded gunmetal waste valve and a weight-loaded leather washer as delivery valve.
Schlumpf 4A5 Schlumpf 4A23	Schlumpf Ag Maschinenfabrik Switzerland	1.5 1.5	40 40	30-60 30-60	A design available in 2 models. Model A23 uses a spring-loaded rubber waste valve mounted on a rocker and a weight-loaded rubber washer as delivery valve. The less efficient model A5 uses a weight-loaded rubber waste valve.

The laboratory experiments also included the use of piezo-electric pressure transducers, displacement transducers and strain guages to observe in detail the changes occuring. Although the resolution of these observations is low they are well presented and provide useful insights into pump operation.

The major criticism of the information presented by Delft is that they supply no indication of how each pump was setup when the results were taken. They simply state that 'waste valve adjustment' was 'kept constant' over the whole range of tests. To ensure a fair comparison the waste valves were presumably initially adjusted to the manufacturers recommendation that would give the best overall performance (efficiency and power output) under typical operating conditions. If this was not the case then the results are practically worthless as some of the pumps may have been badly tuned whilst others were well tuned for the given operating conditions.

2.3 Summary of results

The results presented by Delft are comprehensive within the limitations of their test rig. Delft provide data for 3 supply heads for each pump over a range of delivery heads. Of these the highest (3m) has been taken as being the most representative and all results presented below are for this fixed supply head. Table 4 shows some results for efficiency and power at typical supply to delivery head ratios.

TABLE 4 SUMMARY OF TEST RESULTS

Pump	Efficiency (%)			Power (Watts)		
	at 30m	at 60m	max	at 30m	at 60m	max
Blake No 2 (1½")	70.5	67	70.5	15.2	12.8	15.1
Blake No 3½ (2½")	72.5	68	73	38.2	12.8	15.1
Alto (1")	29	-	47	2.2	-	4.3
Alto (2")	41	23	42	10.3	3.4	7.3
Vulcan (1")	58	58	59	5.1	3.9	4.8
Vulcan (2")	75	55	77	15.2	5.9	15.9
SANO No 1 (1")	64	57	67	3.4	2.9	4.0
SANO No 4 (2")	67	66	69	20.6	19.6	20.2
Davey No 3 (1")	54	-	60	3.4	-	4.7
Rife 20HDU (2")	43	48	48	19.1	19.6	12.7
Schlumpf 4A5 (1½")	-	-	62	-	-	15.2
Schlumpf 4A23 (1½")	43	15	62	8.8	2.0	17.2

Five of the pumps tested are of similar specification to the DTU models and were therefore selected for analysis and comparison. Graphs 1 and 2 show power and efficency curves for these five pumps. The large variations between pumps can be seen quite clearly and comparison is complicated by the marked differences in the power and efficiency curves. At low heads for instance the Vulcan 2" ram is the most efficient but has the lowest output power and will only run up to a delivery head of 85 m.

2.4 Conclusions about commercial pumps

Delft offer no conclusions or direct comparisons between the various pumps tested. The results are complex and comparison has to be based on many factors in order to produce sensible recommendations. In any given situation the exact requirements will vary. The points given below are an attempt to pick out the main items in order to draw some conclusions.

1) The Vulcan 2" has the highest efficiency recorded at 76.9% and at low heads (up to 36 m) has the best effiency of all the pumps.

2) The Rife 2" has the lowest efficiency up to 70 m and never increases over 50%. However it has the widest range of delivery heads over which it will operate. Overall it would be fair to say that the Rife pump has the poorest efficiency.

3) Overall the Blake 2½" is the most efficient closely followed by the smaller Blake 1½". They show good efficiencies over the normal operating range and will pump over a wide range of delivery heads.

4) Similarly the SANO 2" shows good efficiencies over a wide range of delivery heads and is more efficient than the Blake pump over about 75 m.

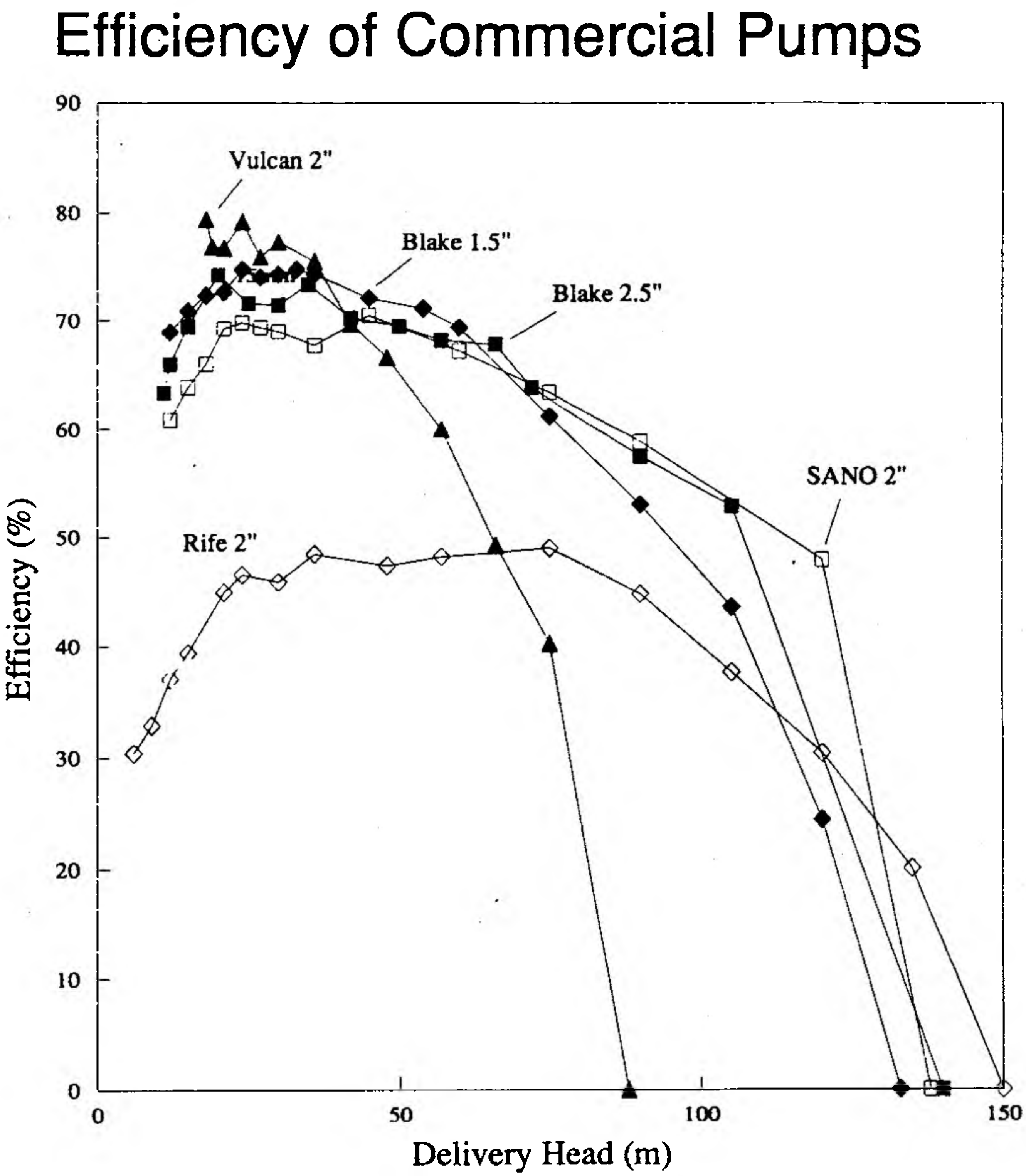
5) The Vulcan 2" has the lowest power output over its small range of delivery heads.

6) Blake 2½" is clearly the most powerful pump over its entire range.

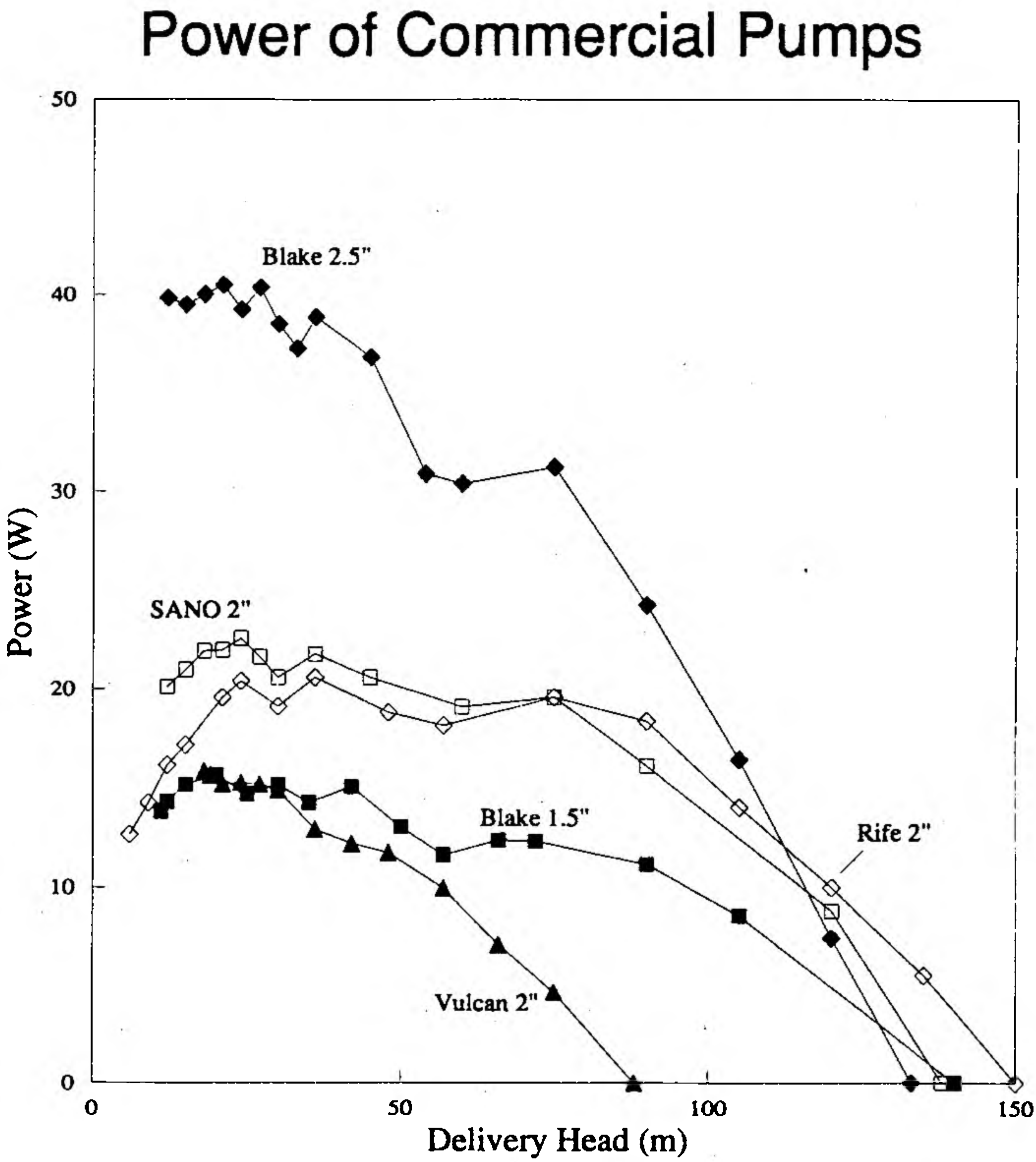
7) At low delivery heads (up to 75 m) the SANO 2" has a slightly higher output than the Rife 2" which above 75 m is better. However there is little to choose between them over the complete range of operation.

8) Attempting to combine both power and efficiency it would seem reasonable to conclude that the Blake 2½" offers the best overall performance. Of the remainder the SANO 2" would seem to be the best compromise.

GRAPH 1


Efficiency of Commercial Pumps
Vulcan 2"
Blake 1.5"
Blake 2.5"
SANO 2"
Rife 2"
Efficiency (%)
Delivery Head (m)
0
10
20
30
40
50
60
70
80
90
0
50
100
150

GRAPH 2


Power of Commercial Pumps
Power (W)
Delivery Head (m)
Blake 2.5"
SANO 2"
Blake 1.5"
Rife 2"
Vulcan 2"
0
10
20
30
40
50
0
50
100
150

2.5 Non-Dimensional Comparison

Delft use a further means of pump comparison by graphically presenting the ratio of delivery to supply flow (q/Q) over a range of delivery to supply head (h/H) ratios. Thus a non dimensional comparison between pumps is possible. For any given head ratio the greater the ratio of flows the better the performace of the pump. Graph 3 shows these ratio curves for the five pumps chosen. Only two conclusions can sensibly be drawn from this comparison:

1) The Rife 2" is notably worse than all its rivals.

2) There is little to choose between all of the other makes of pump.

Despite this lack of obvious conclusions the ability to compare pumps using non dimensional parameters may prove valuable.

3. SUMMARY OF DTU TESTS

3.1 Experimental rig and procedure

The DTU has established performance testing rigs at the University to allow comprehensive analysis of prototype pumps. The major restrictions imposed by the rigs location are:

i) drive pipe length - limited to 10.5 m and horizontal
ii) drive head - restricted to 2, 3, 4 or 5 m

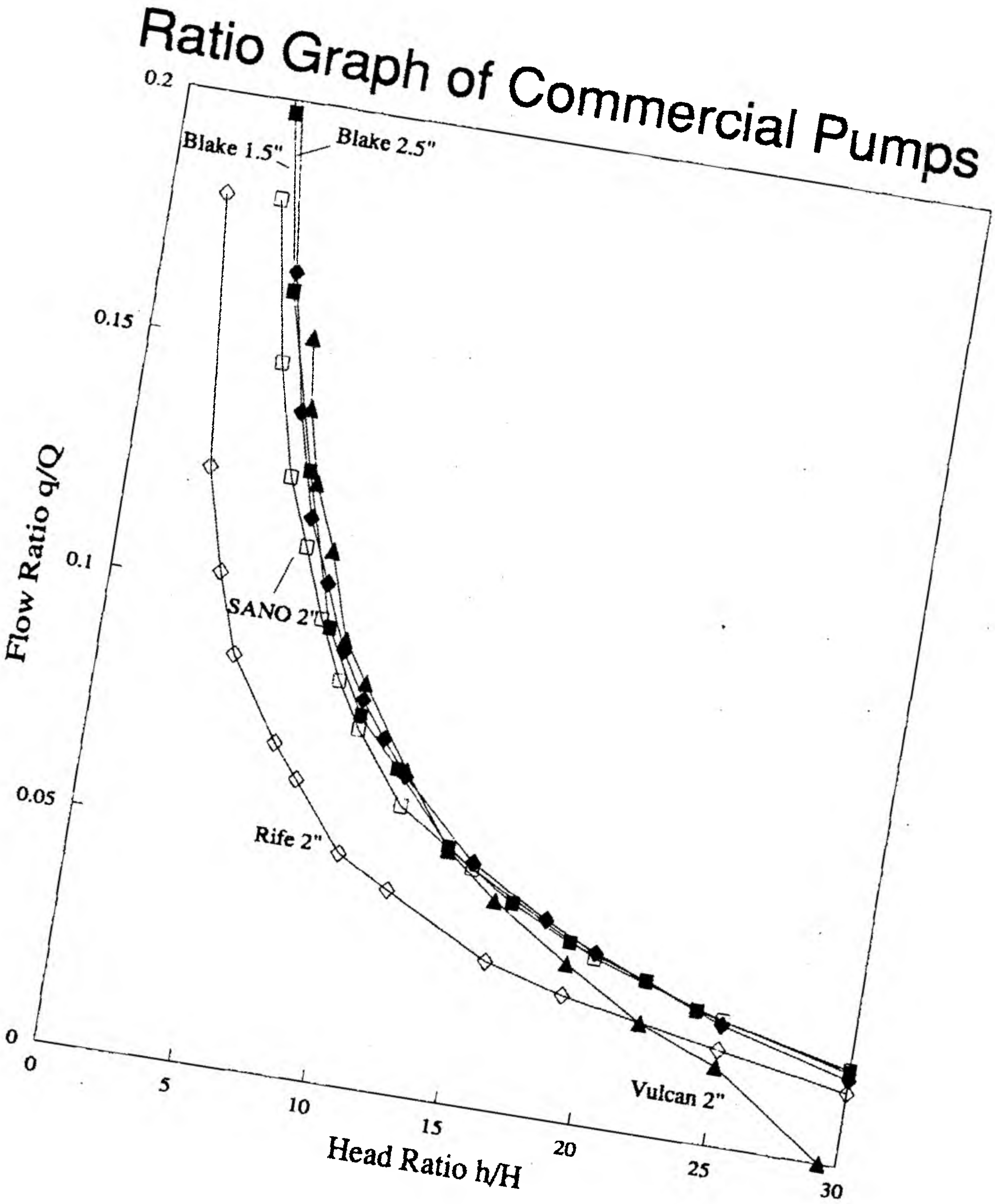
The delivery head is controlled using a needle valve providing an accurately variable orifice over which the desired head can be dropped (measured by a pressure guage). Both drive and delivery flows are measured by float type flow meters for fast and accurate readings.

For their tests Delft used three drive head setting, 1, 2 and 3 m. However for comparison between DTU and commercial pumps only one drive head was chosen (3.0 m).

3.2 Pump Tuning

The DTU pump design selected for testing was the Mark 6.4 which typically uses a 2" BSP drive pipe but also runs using 1½" pipe (see Figure 1). As the Delft work gives no clear indication of how pumps were tuned a series of results were recorded using different settings of the impulse valve. Low stroke, low weight settings generally give high efficiency and low power output Up to a point it is also true to say that high stroke,

GRAPH 3

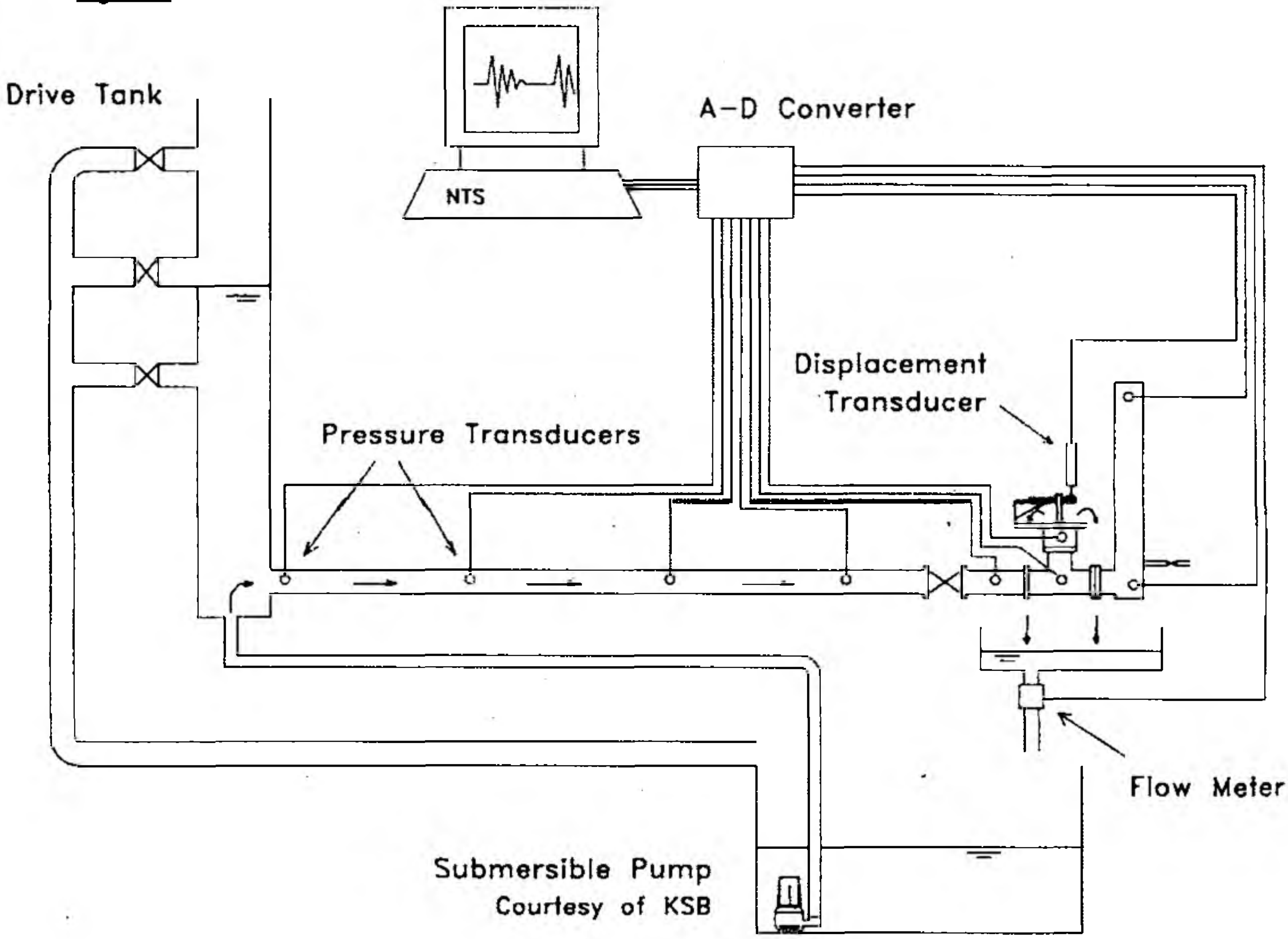


Ratio Graph of Commercial Pumps
0.2
0.15
0.1
0.05
0
Flow Ratio q/Q
Blake 1.5"
Blake 2.5"
SANO 2"
Rife 2"
Vulcan 2"
0
5
10
15
20
25
30
Head Ratio h/H

high weight settings give low efficiency and high output power.

Choosing which of these results are used to compare against the Delft ones is dealt with in Section 4.1.

Figure 1



3.3 Summary of test results

Table 5 gives a selection of results for identical conditions as those in Table 4 for the commercial pumps.

Graphs 4 and 5 show the variation in efficiency and power over the full range of potential delivery heads.

Graph 6 presents the non dimensional flow ratio to head ratio curves for these results.

The design of the M6.4 pump allows it to be tuned to suit a wide range of flow and head conditions, making one model applicable to many sites. The results show the wide range of efficiency and power obtainable under these given conditions and emphasize how important tuning can be. If drive water is limited then peak efficiency will be required to make best use of that available. If there is plenty of drive water pumps should be tuned to give maximum power output despite the lower efficiency of such a setting.

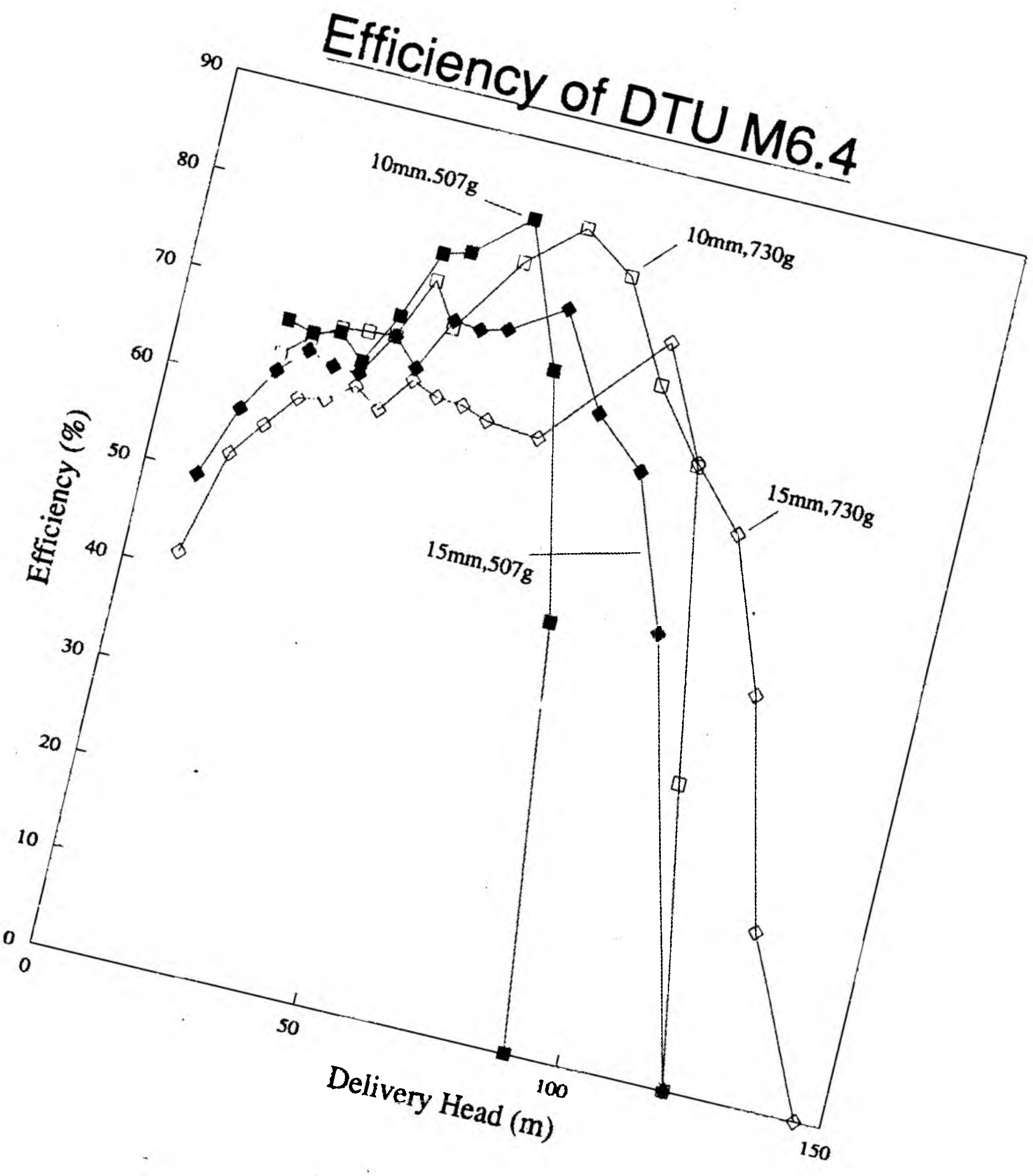
It is clear that both good efficiency and power are obtainable over a broad range of delivery heads and that the pump is capable of operating at very high head ratios.
The stroke of teh M6.4 design can range between 5 mm 40 mm with infinite adustment between these limits. The minimum weight of the valve assembly is 507g comprising the plug, stem, nuts etc. although recent modification to the pump enables further reduction. Maximum weight is limited by the physical dimensions but 1000g should be considered as the upper limit.

The results show for just a few combinations of stroke and weight how the efficiency and power output characteristics of the pump can be dramatically altered. Low stroke and weight give high efficiency but low power over a limited range of delivery heads whereas high stroke and weight give lower efficiency but high power output over a wider possible range of heads. Tuning of the pump to best suit any particular set of conditions is a complicated process to explain and is dealt with in other DTU literature.

TABLE 5 SUMMARY OF DTU M6.4 TEST RESULTS

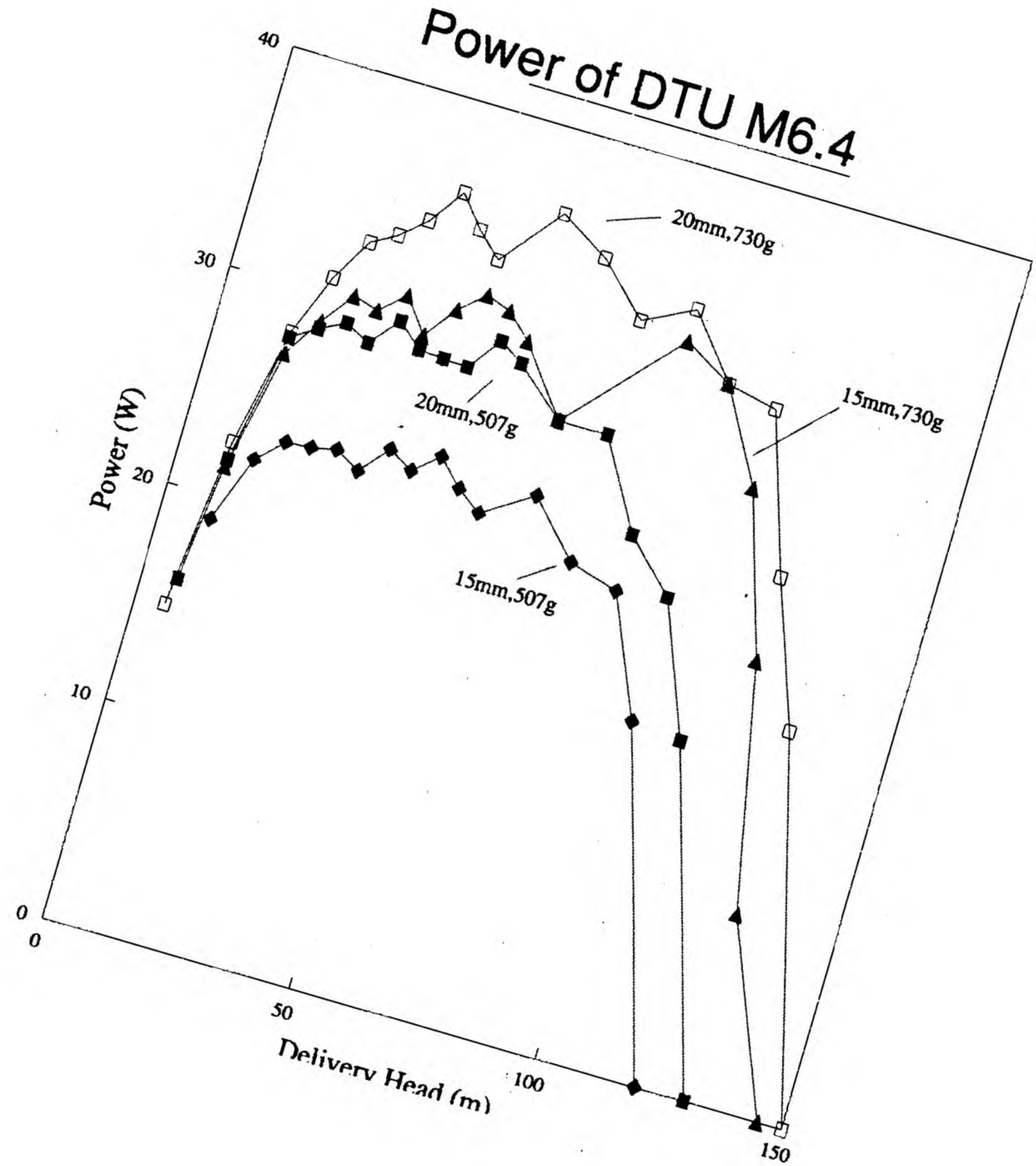
	Pump Settings			Efficiency			Power		
	Drive Pipe Dia(")	Stroke Length (mm)	Weight (g)	at 30m	at 60m	max	at 30m	at 60m	max
1)	1.5	10	507	68	83.3	88.9	16.7	17.2	18.3
2)	1.5	15	507	59.7	64.8	68.7	22.6	22.6	24.6
3)	1.5	20	507	50	54.7	60.4	24.5	25.5	28.1
4)	2	10	507	66.7	82.2	82.2	15.7	18.1	18.1
5)	2	15	507	63.2	70.8	74.2	23.5	22.6	24.5
6)	2	20	507	58	62.5	65.9	28.5	29.4	30.1
7)	2	10	730	67.1	77.8	82.6	24	24	27.5
8)	2	15	730	59.8	61.4	73.3	30.2	30.4	32.4
9)	2	20	730	51.9	58.7	61.6	33.4	36.3	36.3

GRAPH 4

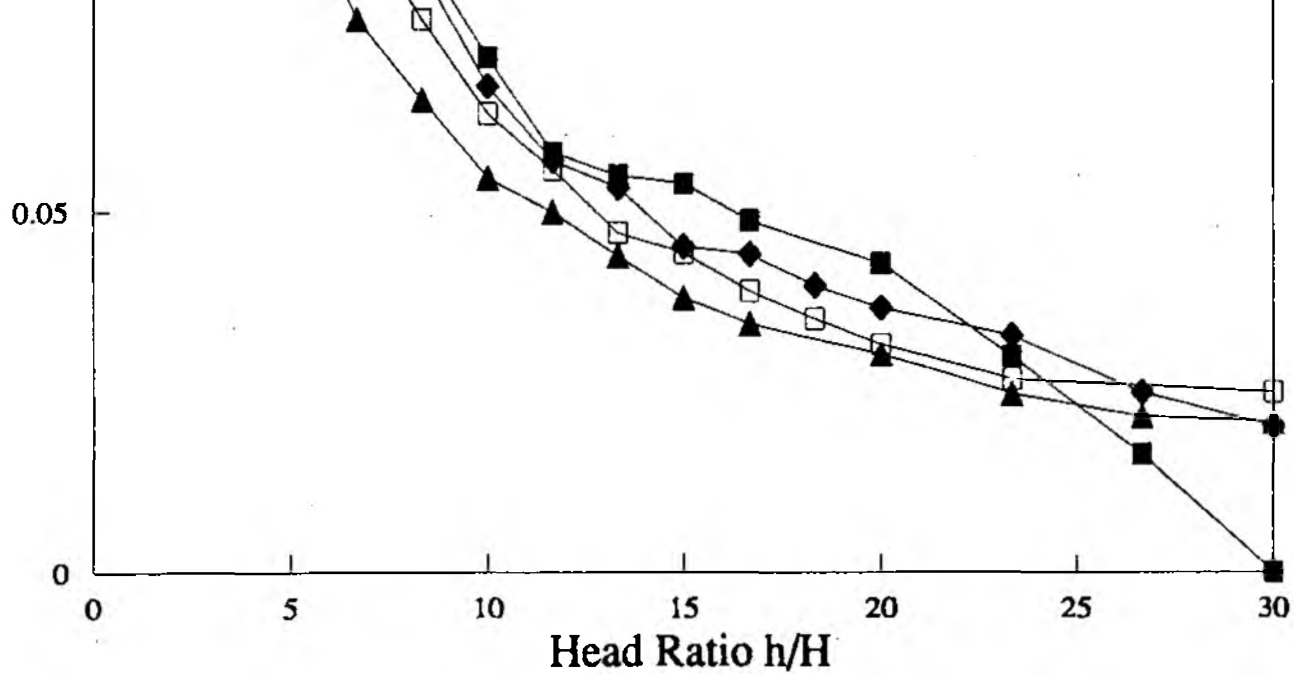


Efficiency of DTU M6.4
Efficiency (%)
Delivery Head (m)
10mm.507g
10mm,730g
15mm,730g
15mm,507g
0
10
20
30
40
50
60
70
80
90
0
50
100
150

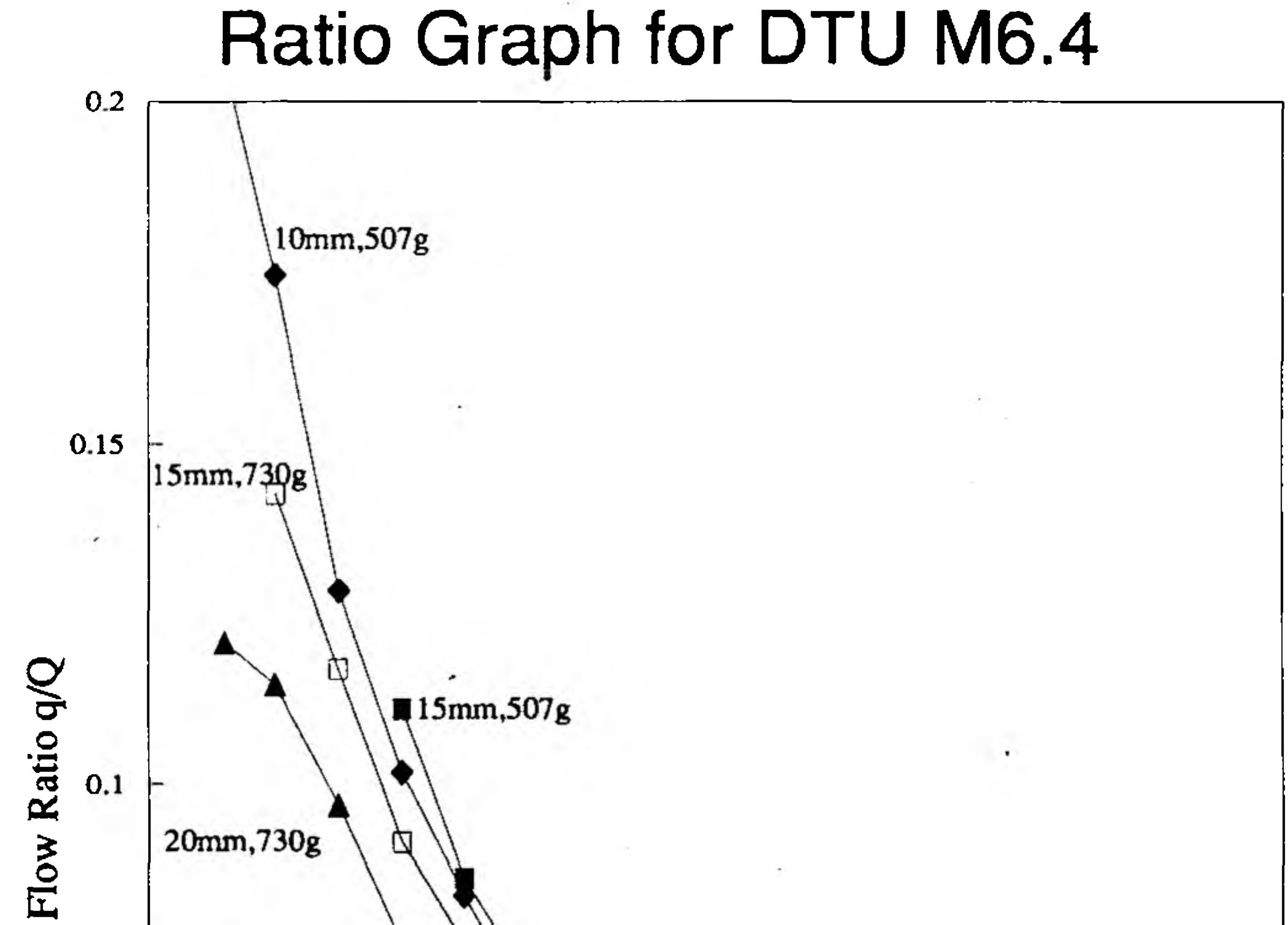
GRAPH 5



Power of DTU M6.4
Power (W)
Delivery Head (m)
40
30
20
10
0
0
50
100
150
20mm,730g
20mm,507g
15mm,730g
15mm,507g


0.05
0
0
5
10
15
20
25
30
Head Ratio h/H

GRAPH 6



Ratio Graph for DTU M6.4
0.2
0.15
0.1
Flow Ratio q/Q
10mm,507g
15mm,730g
15mm,507g
20mm,730g

4. COMPARISON OF DTU AND COMMERCIAL PUMPS

4.1 Problems in comparison

There are inevitably problems and potential inaccuracies in taking data from two separate sets of tests carried out on two different test rigs. The main areas of difficulty are outlined below along with some explanation of their significance and potential methods for overcoming them.

a) The drive pipe lengths of the two test rigs differ by 1.5 m with the Delft rig using an inclined pipe whereas the Warwick tests use a horizontal one. The length of the drive pipe of a hydraulic ram pump system affects (among other things) the time taken for pressure waves to traverse the length of the pipe, total friction in the system and the energy available for pumping. The exact effects of these parameters on pump performance are complicated to evaluate and given the relatively small difference can be assumed to have no major effect for the purposes of this comparison.

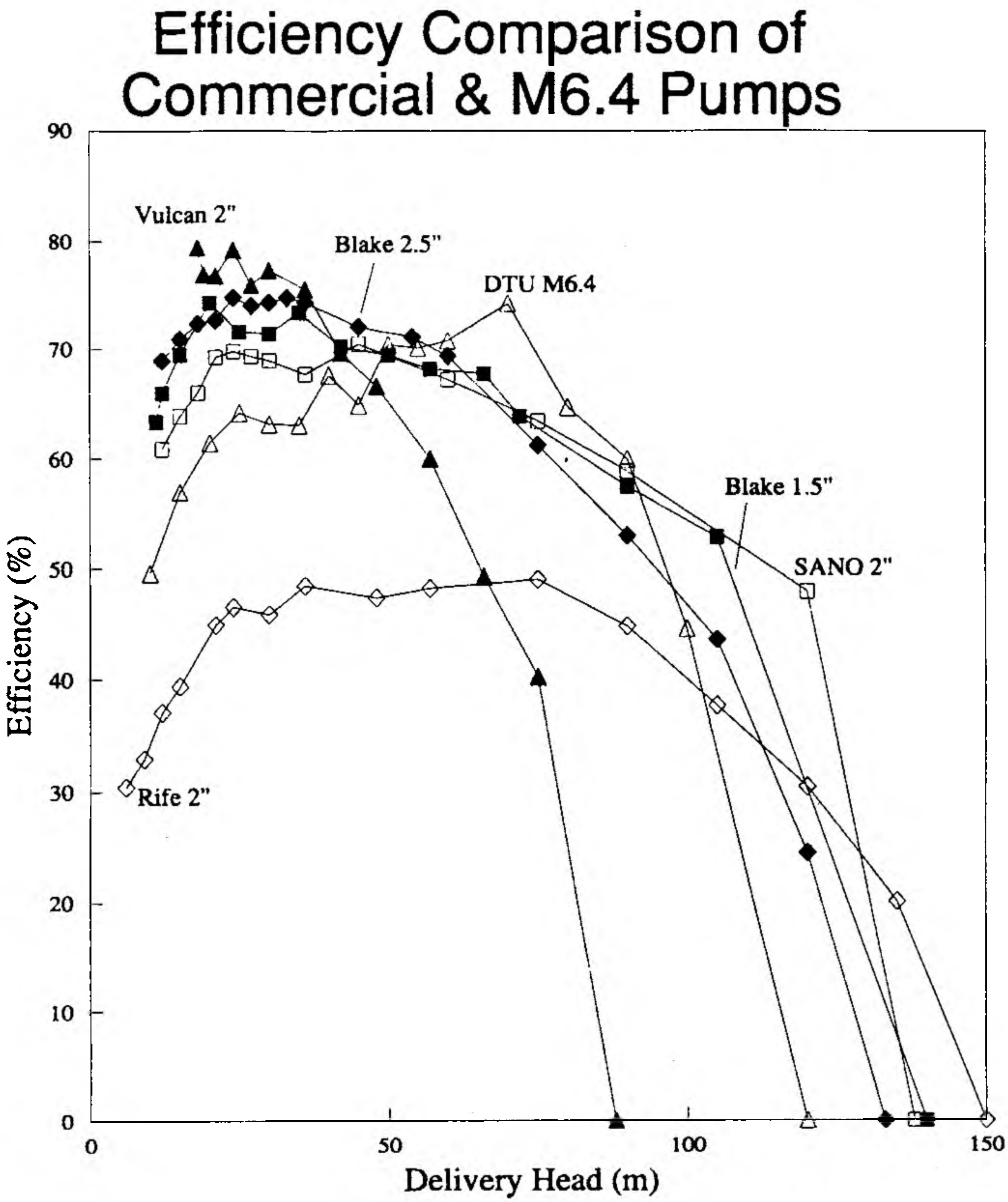
b) As has already been mentioned, the Delft work is unclear about the tuning of each of the pumps tested, other than the fact that they were left constant throughout testing once installed. To allow a sensible comparison it has been assumed that the commercial pumps tested by Delft were each set to some recommended point that gave a reasonable efficiency and power output across a broad range of conditions. In order to compare the DTU pump a best average setting from those taken has been chosen and also comparisons of the optimum settings for efficiency and power. The setting chosen for the general comparison is labelled as No. 5 in Table 5 using a standard 2" drive pipe, a valve stroke of 15 mm and weight of 507 g.

4.2 Efficiency comparisons

The efficiency of the DTU pump is of the same order as the better of the commercial models. Graph 7 shows the chosen average and peak efficiency settings of M6.4 against the 5 commercial pumps.

In a typical operating range say 20-80 m the DTU average setting gives efficiencies ranging between approx 63% and 74%. The DTU peak setting of those chosen returns results for the same range between 64% and 83%. Detailing the comparison between these settings and those for the commercial models is best don visually. The main points that can be drawn are:

GRAPH 7



Efficiency Comparison of Commercial & M6.4 Pumps
Vulcan 2"
Blake 2.5"
DTU M6.4
Blake 1.5"
SANO 2"
Rife 2"
Efficiency (%)
Delivery Head (m)
0
10
20
30
40
50
60
70
80
90
0
50
100
150

a) The Blakes machines have the best efficiency over the widest range of delivery heads with between 62% and 75% in the 20-80 m operating band. This is very similar the DTU average setting although the shape of the curves is somewhat different giving maximum and minimum efficiencies at different heads.

b) The DTU M6.4 pump has a markedly different efficiency profile from all the commercial models with peak efficiency occurring at much higher heads.

c) The DTU M6.4 is capable of operating over a range of delivery heads as great as any of the commercial models.

Separate tests to determine the peak efficiency of the M6.4 have shown that it is capable of running at efficiencies of over 90%.

4.3 <u>Power Comparison</u>

Graph 8 shows the chosen average and peak power settings for the M6.4 against the 5 commercial pumps.

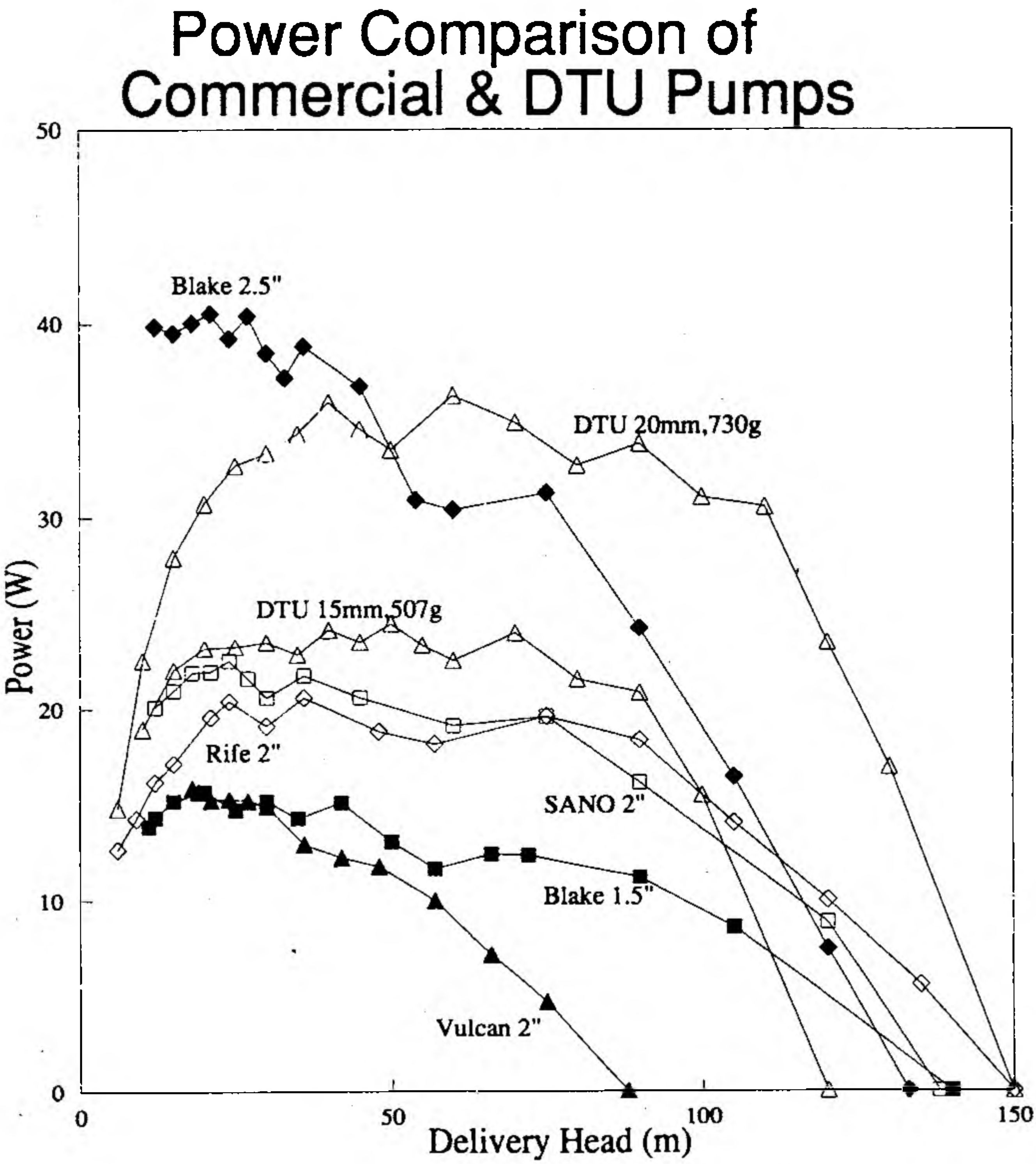
a) The average setting of the DTU M6.4 is clearly more powerful over a very wide range of delivery heads than all of the commercial models of the same size. The Blake 2½" model is clearly more powerful as should be expected from this considerably larger capacity pump.

b) The peak setting of the M6.4 used gives power output similar to and, at higher heads, better than even the Blake 2½" model. Increased weight on the M6.4 would further increase the power output although the limitation of the pipe size and drive head would probably limit peak power to around 40 watts.

5. COST COMPARISON

Table 2 in section 2.1 shows the costs of 10 rams in US$ in 1982 as given by Delft. These costs are for the pumps alone with no drive pipe, delivery pipe, shipping etc. included, and range from US$ 1000 to 3500. No updated prices are available for comparison with the DTU pump so an annual inflation rate of 5% has been assumed. A comparative cost for the DTU pumps is hard to ascertain as they have only been manufactured as one-off prototypes to date in the UK. The design of all the DTU pumps is intended to allow manufacture in the country of use, avoiding any shipping and importation problems but working to the constraints imposed in non-industrialised areas.

GRAPH 8



Power Comparison of Commercial & DTU Pumps
Power (W)
Delivery Head (m)
0
10
20
30
40
50
0
50
100
150
Blake 2.5"
DTU 20mm,730g
DTU 15mm,507g
Rife 2"
SANO 2"
Blake 1.5"
Vulcan 2"

The Baptist Community of Western Zaire (CBZO) have a village water supply programme installing the DTU M6.4 in its rural areas. Currently the manufacture of these pumps is being contracted to a workshop in Kinshasa who are producing them in small batches as requested. The total cost of these units in Zaire is approx. US$450 but once proper manufacture has started it is estimated that this will reduce to $250-$300.

The actual cost to an end user in a developing country of a ram pump will include any shipping and importation costs if it is made eslewhere. To accurately compare the DTU pumps made in country with imported commercial models the costs associated with transportation should be included. However such costs are so dependant upon the shipping distance, customs duties etc. that they would be impossible to estimate with any accuracy.

Table 6 takes the information from Table 2 and adds to the costs an allowance for inflation. It also includes information about the DTU Mark 6.4 pump for the conditions specified by Delft. The information concerning expected delivery flow was arrived at by projecting measured performance figures for different conditions. The DTU is in the process of developing a sophisticated computer simulation and model which are being used in pump analysis and will form the basis of comprehensive design charts. Ultimately this will be able to predict the performance of any pump that has been suitably calibrated under any set of conditions with a good degree of accuracy.

The predicted performance of the DTU pump ranks it 5th in terms of delivery flow and efficiency. The comparison of costs shows the vast difference in the price of the pumping unit and the cost per litre delivered. This illustrates the legitimacy of the approach of producing high performance pumps at low cost in the country of use. If a true comparison to include transportation costs were made the results would clearly be more conclusive still.

TABLE 6 - COST COMPARISON

Type of Hydraulic Ram	Pumping Rate q [L/min]	Efficiency (Trade) (%)	Approx Price for Ram Alone [US$-1982]	Cost per litre delivered	Rank
Vulcan 21/2	6.10	68	1800	295	4
Blake Hydram 31/2	6.00	67	1500	250	3
Sano No. 5/65 mm	6.95	77	1500	216	2
Rife 20 HDU	5.40	60	1650	306	5
Schlumpf 5A23	5.50	61	4050	737	9
Alto CH 66-110-18	5.40	60	3150	584	7
Briau D4	5.40	60	4800	890	11
CeCoCo - H50	6.90	77	5250	761	10
WAMA No. 6	4.50	50	2250	500	6
BZH-Ram W6	2.70	30	1800	666	8
DTU M6.4 2"	5.5	65	300	55	1

6. SUMMARY OF CONCLUSION

The following points summarise the conclusions that can be drawn from these tests.

a) The DTU Mark 6.4 pump gives efficiencies comparable with the more efficient commercial pumps over a wide range of condition.

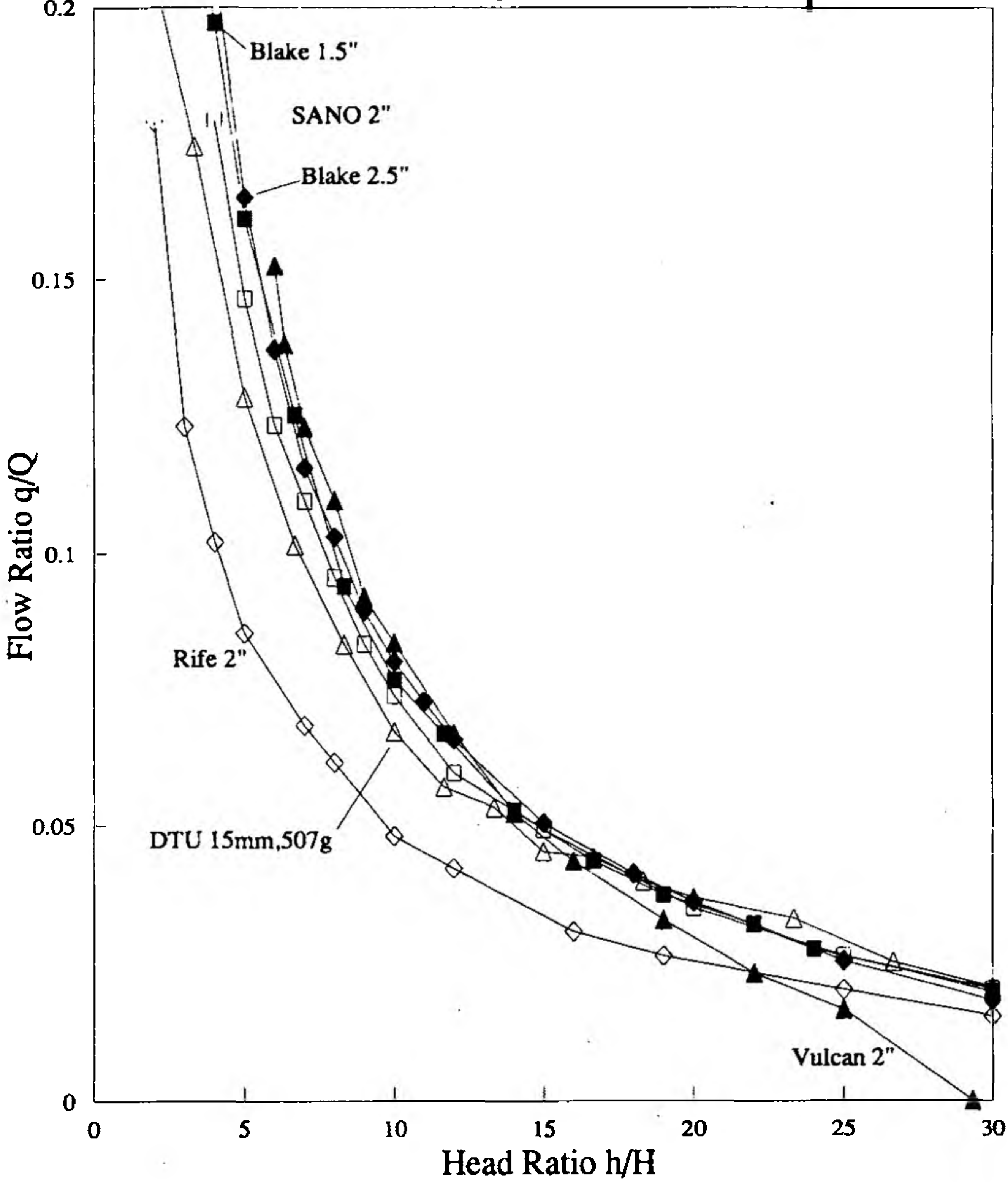
b) The M.6.4 produces a higher output power than all commercial models of the same size over a wide range of conditions.

c) The M6.4 has a cost to output ratio considerably lower than all of its commercial rivals.

The tests show that it is possible to produce simple and comparatively cheap hydraulic ram pumps that can match and exceed the performance of commericial models. Whilst endurance tests to date indicate that the DTU pumps are likely to exhibit adequate durability no data is yet available for a long term comparison with commercial rivals. The current design specification is to produce low maintenance pumps whose steel components have a 10 year life and rubber components last approximately 6 months.

More detailed information concerning the design, manufacture, installation and performace of DTU pumps is available on request.

Ratio Comparison of Commercial & DTU Pumps

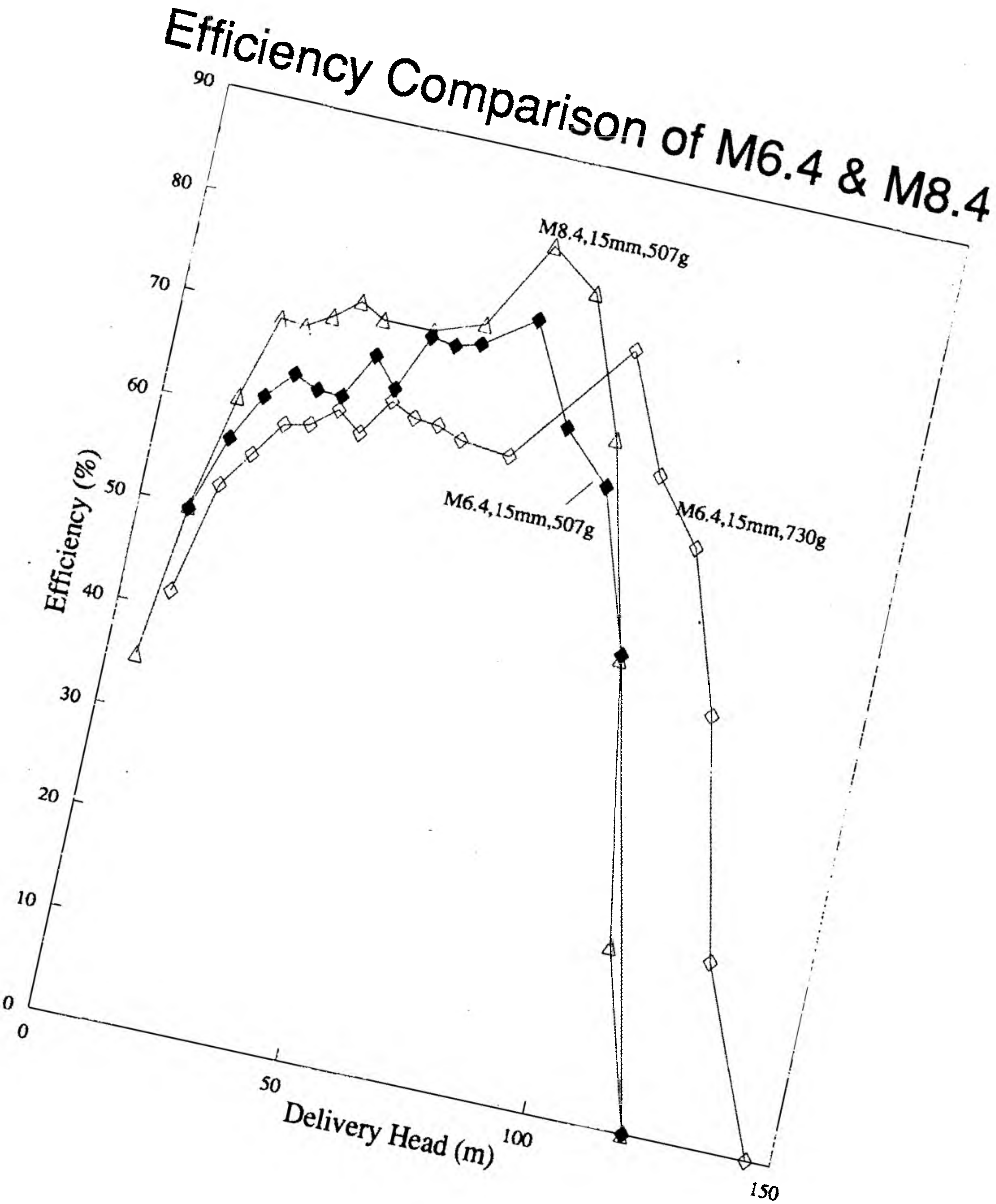


Appendix A **CONTINUING DTU PUMP DEVELOPMENT**

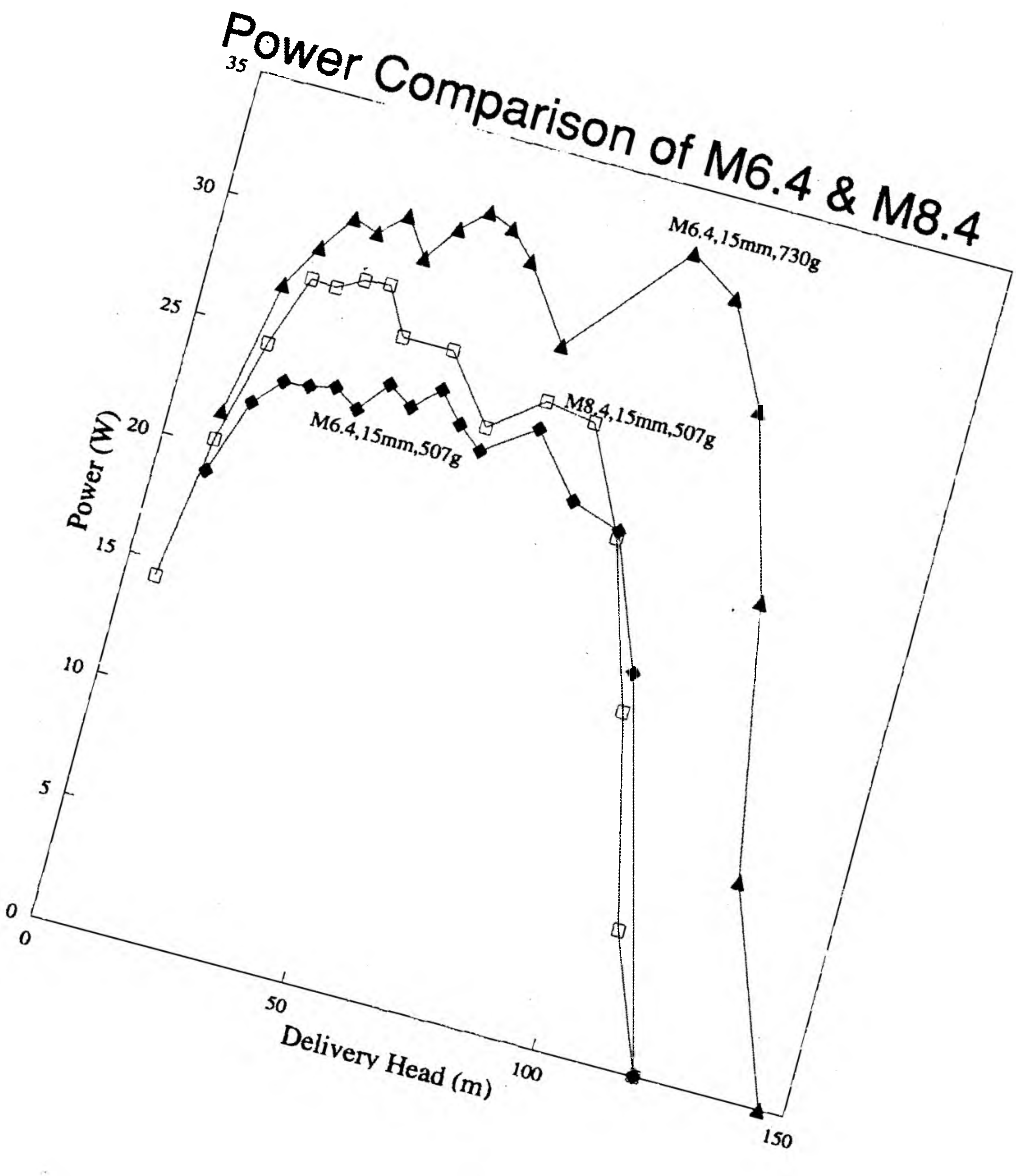
The Mark 6.4 pump was set as a DTU standard in 1990 to allow organisations interested in using the pummp to have a standard model to work with. Since that time a number of developments have occurred in the on-going research programme and new design initiatives produced. One such is the development of the Mark 8.4 that uses the same valves as the M6.4 but replaces the 2" fittings used in the pump body witha body of 4" velded construction. This alteration dramatically reduces the peak overpressure experienced the system, damps out the high frequency oscillations experienced during the cycle and increases both the efficiency and power output for any given set of conditions.

Graphs 10 and 11 allow comparison of the M8.4 with the M6.4 under identical conditions.

GRAPH 10


Efficiency Comparison of M6.4 & M8.4
Efficiency (%)
Delivery Head (m)
M8.4,15mm,507g
M6.4,15mm,507g
M6.4,15mm,730g
0
10
20
30
40
50
60
70
80
90
0
50
100
150

GRAPH 11



Power Comparison of M6.4 & M8.4
Power (W)
Delivery Head (m)
M6.4,15mm,730g
M8.4,15mm,507g
M6.4,15mm,507g
0
5
10
15
20
25
30
35
0
50
100
150

Appendix B TABLES OF PERFORMANCE RESULTS

Tablulated results used in production of Graphs.

The following pages list the information supplied by Delft and that gained under similar conditions on the DTU pump. In all cases the **Supply Head** = 3m.

1) Blake Hydram No2 Drive pipe dia = 1.5" Supply Head = 3.00m

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
11	3.666667	0.686	7.7	36.9	0.208672	55.64589	63.30344	13.84845
12	4	0.686	7.3	37	0.197297	59.18919	65.91422	14.3226
15	5	0.663	6.2	38.45	0.161248	64.49935	69.42889	15.2055
20	6.666667	0.667	4.8	38.3	0.125326	71.01828	74.24594	15.696
25	8.333333	0.669	3.6	38.3	0.093995	68.9295	71.59905	14.715
30	10	0.645	3.1	40.3	0.076923	69.23077	71.42857	15.2055
35	11.66667	0.683	2.5	37.25	0.067114	71.58837	73.37526	14.30625
42	14	0.64	2.2	41.65	0.052821	68.66747	70.23945	15.1074
50	16.66667	0.696	1.6	36.8	0.043478	68.11594	69.44444	13.08
57	19	0.737	1.25	33.6	0.037202	66.96429	68.14921	11.64938
66	22	0.714	1.15	36.2	0.031768	66.71271	67.73762	12.40965
72	24	0.695	1.05	38.4	0.027344	62.89062	63.87833	12.3606
90	30	0.697	0.76	38.9	0.019537	56.6581	57.48865	11.1834
105	35	0.758	0.5	32.6	0.015337	52.14724	52.87009	8.58375
140	46.66667		0		0	0	0	0

2) Blake Hydram No 3.5 Drive pipe dia = 2.5" Supply Head = 3.00m

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
12	4	0.671	20.3	97.6	0.207992	62.39754	68.87193	39.8286
15	5	0.664	16.1	97.5	0.165128	66.05128	70.86268	39.48525
18	6	0.657	13.6	99.2	0.137097	68.54839	72.34043	40.0248
21	7	0.646	11.8	101.9	0.1158	69.47988	72.64732	40.5153
24	8	0.667	10	97	0.103093	72.16495	74.76636	39.24
27	9	0.633	9.15	102.1	0.089618	71.69442	74.02247	40.39268
30	10	0.664	7.85	97.8	0.080266	72.23926	74.30194	38.50425
33	11	0.686	6.9	94.6	0.072939	72.93869	74.77833	37.22895
36	12	0.652	6.6	100	0.066	72.6	74.29644	38.8476
45	15	0.672	5	99.1	0.050454	70.63572	72.04611	36.7875
54	18	0.735	3.5	85.1	0.041128	69.91774	71.10609	30.9015
60	20	0.727	3.1	86.3	0.035921	68.25029	69.35123	30.411
75	25	0.644	2.55	101.5	0.025123	60.29557	61.26862	31.26938
90	30	0.697	1.65	91.7	0.017993	52.18103	53.02625	24.27975
105	35	0.761	0.96	76.1	0.012615	42.89093	43.60239	16.4808
120	40	0.828	0.38	61.7	0.006159	24.01945	24.48454	7.4556
133	44.33333	0.88	0	50.3	0	0	0	0

3) Vulcan 2" Drive pipe dia = 2" Supply Head = 3.00m

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
18	6	0.422	5.4	35.4	0.152542	76.27119	79.41176	15.8922
19	6.333333	0.41	5.05	36.55	0.138167	73.68901	76.88301	15.68783
21	7	0.425	4.44	36.05	0.123162	73.89736	76.75969	15.24474
24	8	0.428	3.9	35.5	0.109859	76.90141	79.18782	15.3036
27	9	0.413	3.45	37.45	0.092123	73.69826	75.91687	15.23003
30	10	0.42	3.05	36.4	0.083791	75.41209	77.31305	14.96025
36	12	0.455	2.2	32.75	0.067176	73.89313	75.53648	12.9492
42	14	0.445	1.78	34	0.052353	68.05882	69.64785	12.22326
48	16	0.44	1.5	34.55	0.043415	65.12301	66.5742	11.772
57	19	0.453	1.07	32.8	0.032622	58.71951	60.02362	9.971865
66	22	0.481	0.66	28.8	0.022917	48.125	49.28717	7.12206
75	25	0.522	0.38	23.2	0.016379	39.31034	40.28838	4.65975
88	29.33333	0	0	0	0	0	0	0

4) SANO No4 2" Drive pipe dia = 2" Supply Head = 3.00m

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
12	4	0.656	10.25	57.1	0.17951	53.85289	60.87602	20.1105
15	5	0.645	8.55	58.4	0.146404	58.56164	63.85362	20.96888
18	6	0.622	7.45	60.3	0.123549	61.77446	65.97786	21.92535
21	7	0.641	6.4	58.3	0.109777	65.86621	69.24266	21.9744
24	8	0.618	5.75	60.15	0.095594	66.91604	69.80273	22.563
27	9	0.633	4.9	58.7	0.083475	66.78024	69.33962	21.63105
30	10	0.648	4.2	56.7	0.074074	66.66667	68.96552	20.601
36	12	0.608	3.7	61.9	0.059774	65.75121	67.68293	21.7782
45	15	0.653	2.8	56.8	0.049296	69.01408	70.4698	20.601
60	20	0.663	1.95	56.1	0.034759	66.04278	67.18346	19.1295
75	25	0.613	1.6	61.45	0.026037	62.48983	63.44171	19.62
90	30	0.665	1.1	54.95	0.020018	58.05278	58.876	16.1865
120	40	0.798	0.45	37.1	0.012129	47.30458	47.93609	8.829
138	46		0		0	0	0	0

5) Rife 20HDU 2" Drive pipe dia = 2" Supply Head = 3.00m

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
6	2	0.965	12.9	72	0.179167	17.91667	30.38869	12.6549
9	3	0.882	9.7	78.7	0.123253	24.65057	32.91855	14.27355
12	4	0.865	8.25	80.8	0.102104	30.63119	37.05783	16.1865
15	5	0.853	7	81.85	0.085522	34.20892	39.39223	17.1675
21	7	0.847	5.7	83.1	0.068592	41.15523	44.93243	19.57095
24	8	0.828	5.2	84.2	0.061758	43.2304	46.53244	20.4048
30	10	0.858	3.9	81.1	0.048089	43.2799	45.88235	19.1295
36	12	0.835	3.5	83.25	0.042042	46.24625	48.41499	20.601
48	16	0.866	2.4	78.7	0.030496	45.74333	47.34895	18.8352
57	19	0.89	1.95	74.9	0.026035	46.86248	48.2108	18.17303
75	25	0.858	1.6	80	0.02	48	49.01961	19.62
90	30	0.84	1.25	82.4	0.01517	43.99272	44.82965	18.39375
105	35	0.888	0.82	75.3	0.01089	37.02523	37.70363	14.07735
120	40	0.947	0.51	66.5	0.007669	29.90977	30.44322	10.0062
135	45	0.999	0.25	55.75	0.004484	19.73094	20.08929	5.518125
154	51.33333		0		0	0	0	0

6) DTU M6.4 2" 1.5" Drive pipe Stroke = 10mm Weight 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
8	2.666667	0.965	10	52	0.238095	39.68254	43.01075	13.08
12	4		8.1	51	0.188811	56.64336	54.82234	15.8922
15	5		6.8	52	0.150442	60.17699	57.82313	16.677
20	6.666667		5.2	51	0.113537	64.3377	61.68446	17.004
25	8.333333		4.2	52	0.087866	64.43515	62.27758	17.1675
30	10		3.4	50	0.072961	65.66524	63.67041	16.677
35	11.66667		3	51	0.0625	66.66667	64.81481	17.1675
40	13.33333		2.4	48	0.052632	64.91228	63.49206	15.696
45	15		2.2	48	0.048035	67.24891	65.73705	16.1865
50	16.66667		2.1	47	0.046771	73.27394	71.2831	17.1675
56	18.66667		2	42	0.05	88.33333	84.84848	18.312
60	20		1.75	42	0.043478	82.6087	80	17.1675
70	23.33333		1.2	42	0.029412	65.68627	64.81481	13.734
80	26.66667		0.6	41	0.014851	38.11881	38.46154	7.848
90	30		0	40	0	0	0	0

7) DTU M6.4 2" Drive pipe 1.5" Stroke 15mm Weight 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
7	2.333333	0.965	14.5	77	0.232	30.93333	36.97632	16.59525
10	3.333333		11.7	79	0.173848	40.56464	42.9989	19.1295
15	5		9	79	0.128571	51.42857	51.13636	22.0725
20	6.666667		7	78	0.098592	55.86854	54.90196	22.89
25	8.333333		5.5	78	0.075862	55.63218	54.89022	22.48125
30	10		4.6	77	0.063536	57.18232	56.37255	22.563
35	11.66667		4.05	77	0.055517	59.21864	58.29735	23.17613
40	13.33333		3.4	74	0.048159	59.39566	58.5702	22.236
45	15		3.1	76	0.042524	59.53361	58.78635	22.80825
50	16.66667		2.8	74	0.039326	61.61049	60.76389	22.89
55	18.33333		2.5	71	0.036496	63.26034	62.35828	22.48125
60	20		2.3	71	0.033479	63.6099	62.7558	22.563
70	23.33333		2.15	73	0.030346	67.77229	66.75538	24.60675
80	26.66667		1.75	68	0.026415	67.79874	66.90562	22.89
90	30		1.35	62	0.022259	64.5507	63.93054	19.86525
100	33.33333		1.15	61	0.019215	62.12754	61.67873	18.8025
110	36.66667		0.7	60	0.011804	42.1023	42.28446	12.5895
120	40		0	58	0	0	0	0

8) DTU M6.4 2" Drive pipe 1.5" Stroke 20mm Weight 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
6	2	0.965	13.5	101	0.154286	15.42857	23.58079	13.2435
10	3.333333		11.8	100	0.133787	31.21693	35.18187	19.293
15	5		9.4	100	0.103753	41.5011	42.96161	23.0535
20	6.666667		7.2	100	0.077586	43.96552	44.77612	23.544
25	8.333333		6	100	0.06383	46.80851	47.16981	24.525
30	10		4.9	98	0.052632	47.36842	47.61905	24.0345
35	11.66667		4.2	98	0.044776	47.76119	47.94521	24.0345
40	13.33333		3.9	100	0.040583	50.05203	50.04812	25.506
45	15		3.5	98	0.037037	51.85185	51.72414	25.75125
50	16.66667		3.25	97	0.034667	54.31111	54.03159	26.56875
55	18.33333		2.9	96	0.031149	53.99212	53.758	26.07825
60	20		2.6	95	0.028139	53.4632	53.27869	25.506
70	23.33333		2.3	94	0.025082	56.01599	55.72863	26.3235
80	26.66667		2.15	95	0.023156	59.43278	59.01527	28.122
90	30		1.7	93	0.01862	53.99781	53.85428	25.0155
100	33.33333		1.3	90	0.014656	47.3882	47.46258	21.255
125	41.66667		0	0	0	0	0	0

9) DTU M6.4 2" Drive pipe 2" Stroke 20mm Weight 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
7	2.333333	0.965	14	108	0.148936	19.85816	26.77596	16.023
10	3.333333		13.3	108	0.140444	32.77015	36.5485	21.7455
15	5		11.3	106	0.119324	47.72967	48.16709	27.71325
20	6.666667		8.7	105	0.090343	51.19418	51.01143	28.449
25	8.333333		7.1	103	0.074035	54.29267	53.73903	29.02125
30	10		5.8	100	0.061571	55.41401	54.82042	28.449
35	11.66667		5.2	102	0.053719	57.30028	56.59204	29.757
40	13.33333		4.4	100	0.046025	56.7643	56.19413	28.776
45	15		3.9	98	0.041445	58.02338	57.40922	28.69425
50	16.66667		3.5	97	0.037433	58.64528	58.04312	28.6125
55	18.33333		3.35	96	0.036158	62.67314	61.81849	30.12488
60	20		3	96	0.032258	61.29032	60.60606	29.43
70	23.33333		2.3	89	0.026528	59.24644	58.78058	26.3235
80	26.66667		2.1	85	0.025332	65.01809	64.29392	27.468
90	30		1.6	86	0.018957	54.9763	54.79452	23.544
100	33.33333		1.3	86	0.015348	49.62613	49.63727	21.255
110	36.66667		0.85	84	0.010222	36.46021	36.73149	15.28725
130	43.33333		0			ERR	ERR	0

10) DTU M6.4 2" Drive pipe 2" Stroke 15mm Weight 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
6	2	0.965	13.3	78	0.205564	20.55641	29.13472	13.0473
10	3.333333		11.6	78	0.174699	40.76305	43.15476	18.966
15	5		9	79	0.128571	51.42857	51.13636	22.0725
20	6.666667		7.1	77	0.101574	57.55842	56.2822	23.217
25	8.333333		5.7	74	0.083455	61.20059	59.59849	23.29875
30	10		4.8	76	0.067416	60.67416	59.40594	23.544
35	11.66667		4	74	0.057143	60.95238	59.82906	22.89
40	13.33333		3.7	73	0.053391	65.84897	64.31986	24.198
45	15		3.2	74	0.045198	63.27684	62.17617	23.544
50	16.66667		3	71	0.044118	69.11765	67.56757	24.525
55	18.33333		2.6	68	0.039755	68.90928	67.51653	23.3805
60	20		2.3	65	0.036683	69.69697	68.35067	22.563
70	23.33333		2.1	66	0.032864	73.39593	71.95301	24.0345
80	26.66667		1.65	68	0.024868	63.82818	63.17301	21.582
90	30		1.42	71	0.020408	59.18367	58.82353	20.8953
100	33.33333		0.95	71	0.013562	43.84963	44.01205	15.5325
120	40							

11) DTU M6.4 2" Drive pipe 2" Stroke 10mm Weight = 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
20	6.666667	0.965	4.8	48	0.111111	62.96296	66.66667	15.696
25	8.333333		3.8	48	0.085973	63.04676	65.97222	15.5325
30	10		3.2	48	0.071429	64.28571	66.66667	15.696
35	11.66667		2.7	49	0.058315	62.20302	64.28571	15.45075
40	13.33333		2.35	45	0.0551	67.95623	69.62963	15.369
45	15		2.15	42	0.053952	75.53325	76.78571	15.81863
50	16.66667		2	43	0.04878	76.42276	77.51938	16.35
60	20		1.85	45	0.042874	81.46002	82.22222	18.1485
70	23.33333		1.25	43	0.02994	66.86627	67.82946	14.30625
80	26.66667		0.6	37	0.016484	42.30769	43.24324	7.848
90	30		0	0	0	0	0	0

12) DTU M6.4 2" Drive pipe 2" Stroke 10mm Weight 730g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
20	6.666667	0.965	7.3	77	0.104735	59.34959	63.20346	23.871
25	8.333333		5.9	75	0.085384	62.61457	65.55556	24.11625
30	10		4.9	73	0.071953	64.75771	67.12329	24.0345
35	11.66667		4.1	71	0.061286	65.3712	67.37089	23.46225
40	13.33333		3.9	77	0.053352	65.80027	67.53247	25.506
45	15		3.6	73	0.051873	72.62248	73.9726	26.487
50	16.66667		2.8	67	0.043614	68.32814	69.65174	22.89
60	20		2.45	63	0.040462	76.87861	77.77778	24.0345
70	23.33333		2.3	65	0.036683	81.92451	82.5641	26.3235
80	26.66667		2.1	71	0.030479	78.22932	78.87324	27.468
90	30		1.63	71	0.023497	68.14185	68.87324	23.98545
100	33.33333		1.15	62	0.018899	61.10655	61.82796	18.8025
110	36.66667		0.5	60	0.008403	29.97199	30.55556	8.9925
120	40		0	60	0	0	0	0

13) DTU M6.4 2" Drive pipe 2" Stroke 15mm Weight 730g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
10	3.333333	0.965	13.1	105	0.142546	33.26079	41.5873	21.4185
15	5		11	105	0.117021	46.80851	52.38095	26.9775
20	6.666667		8.8	105	0.091476	51.83645	55.87302	28.776
25	8.333333		7.4	104	0.076605	56.17667	59.29487	30.2475
30	10		6.1	102	0.063608	57.24713	59.80392	29.9205
35	11.66667		5.4	102	0.055901	59.62733	61.76471	30.9015
40	13.33333		4.5	100	0.04712	58.11518	60	29.43
45	15		4.2	99	0.044304	62.02532	63.63636	30.9015
50	16.66667		3.9	104	0.038961	61.03896	62.5	31.8825
55	18.33333		3.5	103	0.035176	60.97152	62.29773	31.47375
60	20		3.1	101	0.031665	60.16343	61.38614	30.411
70	23.33333		2.4	92	0.026786	59.82143	60.86957	27.468
90	30		2.2	90	0.025057	72.66515	73.33333	32.373
100	33.33333		1.9	102	0.018981	61.37196	62.0915	31.065
110	36.66667		1.5	98	0.015544	55.44041	56.12245	26.9775
120	40		1	98	0.010309	40.20619	40.81633	19.62
130	43.33333		0.4	98	0.004098	17.34973	17.68707	8.502
145	48.33333		0	98	0	0	0	0

14) DTU M6.4 2" Drive pipe 2" Stroke 20mm Weight 730g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
6	2	0.965	15.1	140	0.120897	12.08967	21.57143	14.8131
10	3.333333		13.8	134	0.114609	26.78869	34.32836	22.563
15	5		11.4	129	0.096939	38.77551	44.18605	27.9585
20	6.666667		9.4	132	0.076672	43.44753	47.47475	30.738
25	8.333333		8	130	0.065574	48.08743	51.28205	32.7
30	10		6.8	131	0.05475	49.27536	51.9084	33.354
35	11.66667		6	126	0.05	53.33333	55.55556	34.335
40	13.33333		5.5	131	0.043825	54.05046	55.97964	35.97
45	15		4.7	128	0.038118	53.36577	55.07813	34.58025
50	16.66667		4.1	123	0.034483	54.02299	55.55556	33.5175
60	20		3.7	126	0.030253	57.4816	58.73016	36.297
70	23.33333		3.05	126	0.024807	55.40192	56.48148	34.90725
80	26.66667		2.5	118	0.021645	55.55556	56.49718	32.7
90	30		2.3	112	0.020966	60.80219	61.60714	33.8445
100	33.33333		1.9	112	0.017257	55.79776	56.54762	31.065
110	36.66667		1.7	117	0.014744	52.58745	53.27635	30.5745
120	40		1.2	119	0.010187	39.72835	40.33613	23.544
130	43.33333		0.8	118	0.006826	28.89647	29.37853	17.004
150	50		0	116	0	0	0	0

Appendix C FURTHER NOTES AND COMMENTS ON THE WORK AT DELFT

The following are a series of points arising from the work at Delft that have not been mentioned in the main text of this paper.

1) Delft managed to record delivery valve movement of rubber type valves by bonding strain guages to the rubber. This technique overcomes the difficulty of recording and analysing delivery valve movement experienced by the DTU.

2) An electronic beat frequency counter was used at Delft to record the operating frequency and period of pumps under test. This is another technique worth pursuing for ongoing tests at Warwick.

3) There is some confusion and lack of consistancy between ram pump manufacturers and users as to how efficiency is measured. Delft produce the following summary

<u>Rankine</u> Drive tank water level is taken as the datum point and supply and delivery heads measured from that point. Therefore the net amount of potential energy in the water delivered $= pgq(h-H)$
the net amount of energy input $= pgQH$

Hence Rankine efficiency $\eta_R = \dfrac{q \times (h-H)}{Q \times H}$

<u>D'Aubuisson</u> The impulse valve orifice is taken as the datum point and therefore

work done $= pgqh$
energy supply $= pg\ (Q + q)H$

D'Aubuisson Efficiency $\eta_D = \dfrac{q \times h}{(Q + q) \times H}$

<u>Manufacturers</u>

Most manufacturers take a simplified efficiency that actually produces a higher value of efficiency under any given set of conditions

$$\eta_m = \frac{q \times h}{Q \times H}$$

Delft prefer the Rankine expression which yields the lowest results particularly at low delivery heads.

The DTU currently uses the D'Aubuisson formula as the test rig measures the total flow into the pump (Q + q) rather than just the waste flow. This has been used throughout this report.

There seems little to choose between these two which produce similar results under normal operating conditions. The simplified efficiency used by many manufacturers is more inaccurate but produces more flattering figures!

4) Observation on relative position of delivery and impulse valves concerning air intake during recoil.

During recoil, large quantities of air can be drawn in through the open impulse valve as water flows back up the drive pipe. Only a small quantity of air can be drawn through the snifter valve so section A remains relatively full of water. As acceleration of water towards the pump re-occurs the majority of the air is expelled through the impulse valve and only the small volume entering from the snifter enters the air vessel.

If the recoil is large in this configuration and a significant quantity of air drawn in through the impulse valve, the column may receed past the vertical pipe section leading to the delivery valve. When acceleration occurs a significant pocket of air may be trapped under the delivery valve. This would tend to dampen out the next pressure pulse and reduce delivery flow.

5) The Alto ram from France has an inflatable rubber air compartment in the air vessel to ensure a permanent separation of air and water. This is an attractive option as it reduces the need for a reliable snifter to replenish air lost from the air vessel.

6) The SANO rams are fitted with a 'drip valve' in the air vessel that appears to be located at a point where the air/water interface should be. It presumably operates by passing water if the level in the air vessel rises above it. What is its operating mechanism?

8) Delft draw the following conclusions about pump performance and operation from results produced by their mathematical model.

- given an available source supply the pumping rate (q) is primarily determined by the supply head (H_s)and the delivery head (h_d).
- an increase of the delivery head (h_d) decreases the quantity pumped per cycle (V_d) and by that the pumping rate (q) decreases.
- an increase of the supply head (H_s) increases the pumping frequency and by that the pumping rate (q) increases.
- an increase of u_c (ie. the velocity of the water in the drive pipe at waste valve closure) normally increases the pumping rate (q) while the pumping frequency decreases, but more water (Q) is needed to operate the ram. However, there is a limiation as to this point: an increase in u_c such that the value of the ratio u_c/u_o approaches unity (where u_o is the maximum attainable velocity of the water in the drive pipe) implies a decrease in pumping rate (q) while, as before, the waste flow (Q) increases, a condition to be avoided.
- the larger the size (ie. drive pipe bore) of the ram, the more water (Q) is required to operate the ram and the more water (q) can be delivered to a higher level (h_d).

9) Delft recommend that if the delivery flow from one ramains insufficient a second ram should be placed below the first and utilise its drive water. However if there is sufficient drive head available to be able to do this would a better performance be gained by having two rams each working from a portion of the drive head or one ram working with the total head available?

10) Delft state that the drive pipe length should be approximately 4 to 7 times the supply head. They make no justification for this statement.

11) Delft state that the best pumping results are usually obtained when the cut off velocity is between 60% and 80% of the terminal velocity of the system.

12) Delft produce equations for predicting pump performance, which require certain information about the system to tbe known.

A loss coefficient for the whole system is required which covers all the losses in the drive pipe, pump and delivery pipe. Delft state that this can be found by holding open the impulse valve, measuring the flow rate at terminal velocity and inserting this into the equation

$$\text{max velocity} \quad V = \sqrt{\frac{2gH}{\text{loss coef}}}$$

This does not take into account the delivery system losses and can only be measured once the system is installed, preventing prediction of performance prior to installation. It would be more useful to have a loss coefficient measured for each pump and also to have a simple method of calculating pipe loss coefficient based on diameter and length.

The other requirement for use of Delft's model is to find the cut off velocity for the particular impulse valve setting. They state that this can be found when the maximum delivery head obtainable is known using

$$\text{Cutoff} = (g/c) \times h_{max}$$

They recommend that h_{max} is found by closing the delivery side of the pump and letting the pump run up to its maximum head. This would provide a rather crude approximation producing a low result and could prove to be very dangerous.

DEVELOPMENT
TECHNOLOGY
UNIT

Working Paper No 33

Comparison Between DTU and Commercial Hydraulic Ram Pump Performance

Development Technology Unit, Department of Engineering,
Warwick University, Coventry CV4 7AL
Tel: (0) 203 523122 Fax: (0) 203 418922 Telex: 311904 UNIVWK G

Comparison between DTU and Commercial Ram Hydraulic Ram Pump Performances

UPDATE AND COMMENTS, APRIL 1996

Since this paper was written, the DTU Mark 6.4 pump has been phased out, to be replaced by first the M8.4 and more recently the S2. All these are pumps normally run with a 2" G.I. drivepipe and delivering to up to 100m using a drive flow of 40 to 120 litres/min. Preliminary results for the M8/S2 on a single setting are given at the end of this paper, indicating that, setting for setting, the M8 is superior to its predecessors. Generally the DTU later models are more efficient than the M6. Of course performance as measured in this paper is only part of the story. The DTU pumps are made largely of mild steel and are therefore more subject to corrosion than most 'commercial' machines. In recent years there has been a trend towards use of plastics in ram pumps as pvc or ABS piping replaces galvanised iron. However plastic pumps can rarely operate reliably at delivery heads exceeding 40 meters. There is also a growing interest in providing the air-cushioning of the output via an enclosed air packet (for example closed cell foam) instead of an air vessel with a free air-water surface. This arrangement can effectively increase the usable drive head by allowing the impulse valve exhaust to emerge under water.

In the paper, the early comparisons with commercial pumps use data from a study by T H Delft which were obtained from slightly larger models (from the respective manufacturers' ranges), and with a much higher drivehead, than the comparisons in the rest of the paper.

Details of the current (1996) DTU designs, namely S1 (for use with a 1" steel drivepipe), S2 (2" steel) and P90 (90 mm. pvc) are given in DTU Technical Releases TR11, TR14 and TR12 respectively.

CONTENTS

T.D.Jeffery
April 1991

List of Tables

Page No.

List of Graphs

List of Figures

1. INTRODUCTION

This paper details the performance of the DTU Mark 6.4 hydraulic ram pump in comparison to commercial models run under similar conditions. Details of the performance of a number of commercial pumps tested at Delft University, Netherlands are used for comparison.

1.1 Delft University

Between 1982 and 1984 J. Tacke of Delft University of Technology carried out tests on a number of commercially available hydraulic ram pumps. These were published in 1988 with comprehensive details of all results obtained and the development of a mathematical model for prediction of ram performance. As part of this programme field tests were conducted by the Foundation of Dutch Volunteers in Rwanda. These aimed to investigate the technical performance and durability under operating conditions in a community setting, social acceptance and community participation in installing, operating and maintaining a hydraulic ram system.

The presentation by Delft of all laboratory test results is excellent and allows a good level of comparison for tests on subsequent designs. Unfortunately no details of the findings or conclusions resulting from the field trials in Rwanda are made available. However the DTU are greatly indebted to Delft for their provision of such a useful resouce.

1.2 DTU, University of Warwick

The Development Technology Unit has been investigating hydraulic ram pump design, performance and manufacture since 1985. This began with student projects and has grown into a full-time research programme largely funded by the Overseas Development Administration. The aims of the programme are:

a) to analyse in detail the operation of the ram pump gaining a comprehensive understanding of the operating principles and complex hydraulic interactions occuring within the pump.

b) to produce pump designs suitable for manufacture in developing countries using available materials and production processes.

c) to thoroughly test such designs for their performance and endurance (including extensive field trials) and offer findings for widespread dissemination.

d) to develop and prove methods for surveying and design of complete water supply installations.

e) to produce design charts and computer based tools to enable design and field engineers to confidently include hydraulic ram pumps as an option in their water supply schemes.

f) to provide technical expertise and training for two African based programmes installing ram pumps for village water supply and irrigation.

In terms of hardware development the DTU has two distinct working areas. The first is the development of designs of steel hydraulic ram pumps based on the 2" diameter BSP pipe that has been found to be widely available in developing countries and is of an appropriate capacity for small village water supply schemes. The second area of hardware development is the production of plastic hydraulic ram pumps based on widely available 110mm plastic pipe and especially suitable for irrigation close to water courses. Rough specifications and performance indications are given below.

	Steel	Plastic
Materials required		
	2" Galvanised pipe	110mm PVC pipe
	2" Fittings	Small amount of
	Mild steel bar	mild steel parts
Manufacturing process		
	Welding	Hand tools
	Drilling	
	Turning	
Typical performance ranges		
	40-140	
Drive flow	40/140 l/min	200-350 l/min
Drive head	2-25m	0.5-2.5m
Delivery flow	2-12 l/min	2-25 l/min
Delivery head	up to 150m	up to 15m
Efficiency	50-90%	20-60%
Expected life	10 years	3 years

This paper is concerned solely with the comparison of the 2" steel pump with its commercially available rivals. Such pumps have found the widest application to date in supplying water for domestic use.

During the research many designs of ram pump have been produced and tested. In 1990 a design was chosen as having proven itself sufficiently in terms of performance and durability to set a 'benchmark' against which furthest developments could be assessed. It is this model, Mark 6.4, that is used in the following comparison against commercial designs.

2. SUMMARY OF WORK CARRIED OUT AT DELFT

2.1 Selection of Pumps

Delft decided to select 12 rams from 6 manufacturers that were applicable in typical village or domestic water supply schemes. In all, details of pumps from 10 manufacturers were obtained and selection made based on the following criteria:

a) as many types of ram design as possible should be included.

b) tests should include both traditional and modern designs.

c) rams should show a reasonable price to performance ratio.

To enable the latter of the criteria, manufacturers were sent a set of conditions and asked to provide details of the pump they would recommend and its expected output and efficiency.

Table 1 shows the results of this comparison.

TABLE 1 - COMPARISON OF HYDRAULIC RAMS

Arrangement: Source Supply = 90 l/min

Supply Head Hs = 7.50 m

Delivery Head hd = 75 m

Type of Hydraulic Ram	Drive Pipe [inch]	[mm]	Volume of Driving Water Required [l/min]	Pumping Rate q [l/min]	Efficiency trd [%]	Approx Price for Ram Alone [US$-1982]
Vulcan 21/2	2	65	36-114	6.10	68	1200
Blake Hydram 31/2	2	65	45- 96	6.00	67	1000
Sano No. 5/65 mm	2	65	50-110	6.95	77	1000
Rife 20 HDU	2	50	38- 95	5.40	60	1100
Schlumpf 5A23	2	50	50-100	5.50	61	2700
Alto CH 66-110-18	2	65	50- 90	5.40	60	2100
Briau D4	2	50	45- 90	5.40	60	3200
CeCoCo - H50	2	50	25-115	6.90	77	3500
WAMA No. 6	2	65	60-100	4.50	50	1500
BZH-Ram W6	2	65	45- 90	2.70	30	1200

Delft concluded that in terms of performance, the choice should favour the first eight pumps and of these the first six provide the best performance to price ratio. From the information given the choice seems to be a reasonable one.

An alternative comparison to evaluate the performance to price ratio would have been to include a cost per litre delivered under these conditions. This significantly alters the ranking of pumps as is shown in Table 2.

TABLE 2

Type of Hydraulic Ram	Pumping Rate q [C/min]	Efficiency (Trade) (%)	Approx Price for Ram Alone [US$-1982]	Cost per litre delivered	Rank
Vulcan 2½	6.10	68	1200	197	3
Blake Hydram 3½	6.00	67	1000	167	2
Sano No. 5/65 mm	6.95	77	1000	144	1
Rife 20 HDU	5.40	60	1100	204	4
Schlumpf 5A23	5.50	61	2700	491	8
Alto CH 66-110-18	5.40	60	2100	389	6
Briau D4	5.40	60	3200	593	10
CeCoCo - H50	6.90	77	3500	507	9
WAMA No. 6	4.50	50	1500	333	5
BZH-Ram W6	2.70	30	1200	444	7

It would also have been interesting to include details from rams currently manufactured in developing countries. Details of the rams selected are given in Table 3.

2.2 <u>Assessment of Experimental Procedure</u>

The tests conducted were fairly comprehensive with each ram being observed for approx. 1 month. The test rig employed allowed a supply head of between 0.5 and 3.0 m with a drive pipe length of 12m. Such a low drive head range is clearly the product of laboratory limitations and does not reflect typically observed ranges in the field of 2 to 20m.

Flows were measured by collecting water for a timed period and weighing it. For each setting this was repeated a number of times to give a more accurate average flow. This is potentially a very accurate method of flow measurement but is open to numerous experimental errors. However it does avoid the need to use flow meters that have their own inaccuracies despite careful calibration.

The designs of ram pump tested exhibit a number of different types of impulse valve. Four of the six have traditional valves above the axis of the pump body but the Schlumpf has its valve below the axis. No mention is made in the text of the Delft work as to where the supply head was measured from. To allow accurate comparison the head should be taken from the orifice of the impulse valve, not the axis of the pump body. In reality both the SANO and Schlumpf pumps may in some situations be able to utilise a given supply head more effectively by their impulse valve design.

TABLE 3 - HYDRAULIC RAMS SELECTED

Type of Hydraulic Ram	Manufacturer	Drive Pipe Diameter [ins]	[mm]	Intake capacity [l/min]	Description
Blake Hydram No. 2 Blake Hydram No. 3.5	John Blake Ltd England	1.5 2.5	40 65	12-25 45-96	A well-established standard design made of cast-iron. Both waste valve and delivery valve consist of a rubber disc covering a perforated gunmetal seat.
Alto J 26-80-8 Alto CH 50-110-18	J.M. Desclaud France	1 2	25 50	8-15 30-60	A recently renewed pump design made of steel pipe components, but using conventional valve designs. Weight-loaded gunmetal waste valve; spring-loaded rubber clack as delivery valve.
Vulcan 1" Vulcan 2"	Green & Carter England	1 2	25 50	4-18 23-46	A standard design made of cast-iron; like the Blake Hydram available for a long time. Both waste valve and delivery valve consist of a rubber disc covering a grid shaped, gunmetal seat.
SANO No. 1-25 mm SANO No. 4-50 mm	Pfister + Langhanss Germany	1 2	25 50	6-16 30-65	A rather unconventional' design, nowadays made of fire zinc-coated steel. Both waste valve and delivery valve are spring-actted and substantially made of gunmetal.
Davey No. 3 Rife 20 HDU	Rife Hydr. Eng Mfg. Co.	1 2	25 50	5-15 38-95	Rife: a fairly standard design made of cast-iron. Weight-loaded rubber waste valve, mounted on a rocker-arm; delivery valve is a rubber disc covering a grid iron seat. Davey: less efficient, less expensive low base configuration, using a weight-loaded gunmetal waste valve and a weight-loaded leather washer as delivery valve.
Schlumpf 4A5 Schlumpf 4A23	Schlumpf Ag Maschinenfabrik Switzerland	1.5 1.5	40 40	30-60 30-60	A design available in 2 models. Model A23 uses a spring-loaded rubber waste valve mounted on a rocker and a weight-loaded rubber washer as delivery valve. The less efficient model A5 uses a weight-loaded rubber waste valve.

The laboratory experiments also included the use of piezo-electric pressure transducers, displacement transducers and strain guages to observe in detail the changes occuring. Although the resolution of these observations is low they are well presented and provide useful insights into pump operation.

The major criticism of the information presented by Delft is that they supply no indication of how each pump was setup when the results were taken. They simply state that 'waste valve adjustment' was 'kept constant' over the whole range of tests. To ensure a fair comparison the waste valves were presumably initially adjusted to the manufacturers recommendation that would give the best overall performance (efficiency and power output) under typical operating conditions. If this was not the case then the results are practically worthless as some of the pumps may have been badly tuned whilst others were well tuned for the given operating conditions.

2.3 Summary of results

The results presented by Delft are comprehensive within the limitations of their test rig. Delft provide data for 3 supply heads for each pump over a range of delivery heads. Of these the highest (3m) has been taken as being the most representative and all results presented below are for this fixed supply head. Table 4 shows some results for efficiency and power at typical supply to delivery head ratios.

TABLE 4 SUMMARY OF TEST RESULTS

Pump	Efficiency (%)			Power (Watts)		
	at 30m	at 60m	max	at 30m	at 60m	max
Blake No 2 (1¼")	70.5	67	70.5	15.2	12.8	15.1
Blake No 3½ (2½")	72.5	68	73	38.2	12.8	15.1
Alto (1")	29	-	47	2.2	-	4.3
Alto (2")	41	23	42	10.3	3.4	7.3
Vulcan (1")	58	58	59	5.1	3.9	4.8
Vulcan (2")	75	55	77	15.2	5.9	15.9
SANO No 1 (1")	64	57	67	3.4	2.9	4.0
SANO No 4 (2")	67	66	69	20.6	19.6	20.2
Davey No 3 (1")	54	-	60	3.4	-	4.7
Rife 20HDU (2")	43	48	48	19.1	19.6	12.7
Schlumpf 4A5 (1¼")	-	-	62	-	-	15.2
Schlumpf 4A23 (1¼")	43	15	62	8.8	2.0	17.2

Five of the pumps tested are of similar specification to the DTU models and were therefore selected for analysis and comparison. Graphs 1 and 2 show power and efficency curves for these five pumps. The large variations between pumps can be seen quite clearly and comparison is complicated by the marked differences in the power and efficiency curves. At low heads for instance the Vulcan 2" ram is the most efficient but has the lowest output power and will only run up to a delivery head of 85 m.

2.4 <u>Conclusions about commercial pumps</u>

Delft offer no conclusions or direct comparisons between the various pumps tested. The results are complex and comparison has to be based on many factors in order to produce sensible recommendations. In any given situation the exact requirements will vary. The points given below are an attempt to pick out the main items in order to draw some conclusions.

1) The Vulcan 2" has the highest efficiency recorded at 76.9% and at low heads (up to 36 m) has the best effiency of all the pumps.

2) The Rife 2" has the lowest efficiency up to 70 m and never increases over 50%. However it has the widest range of delivery heads over which it will operate. Overall it would be fair to say that the Rife pump has the poorest efficiency.

3) Overall the Blake 2½" is the most efficient closely followed by the smaller Blake 1½". They show good efficiencies over the normal operating range and will pump over a wide range of delivery heads.

4) Similarly the SANO 2" shows good efficiencies over a wide range of delivery heads and is more efficient than the Blake pump over about 75 m.

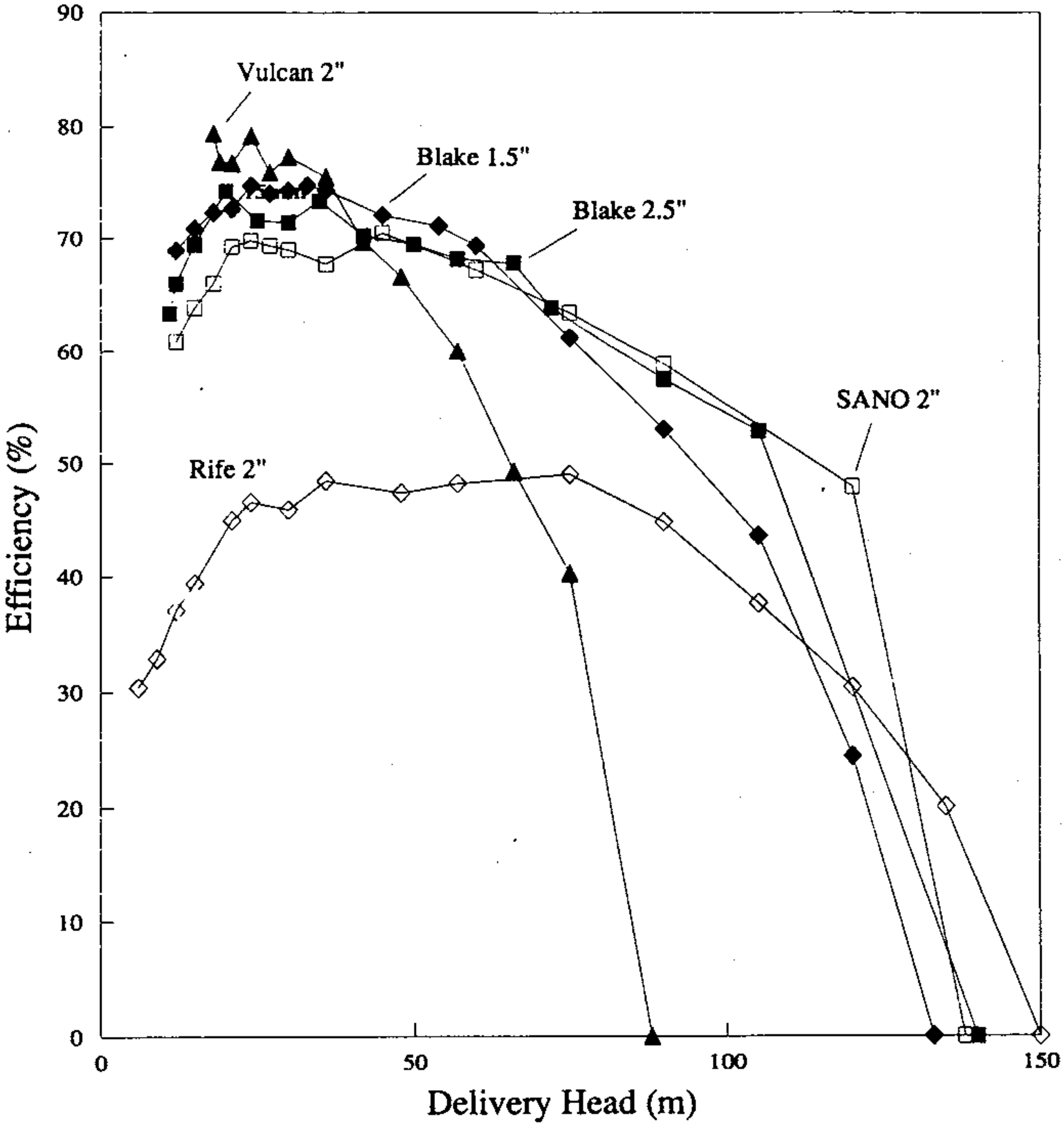
5) The Vulcan 2" has the lowest power output over its small range of delivery heads.

6) Blake 2½" is clearly the most powerful pump over its entire range.

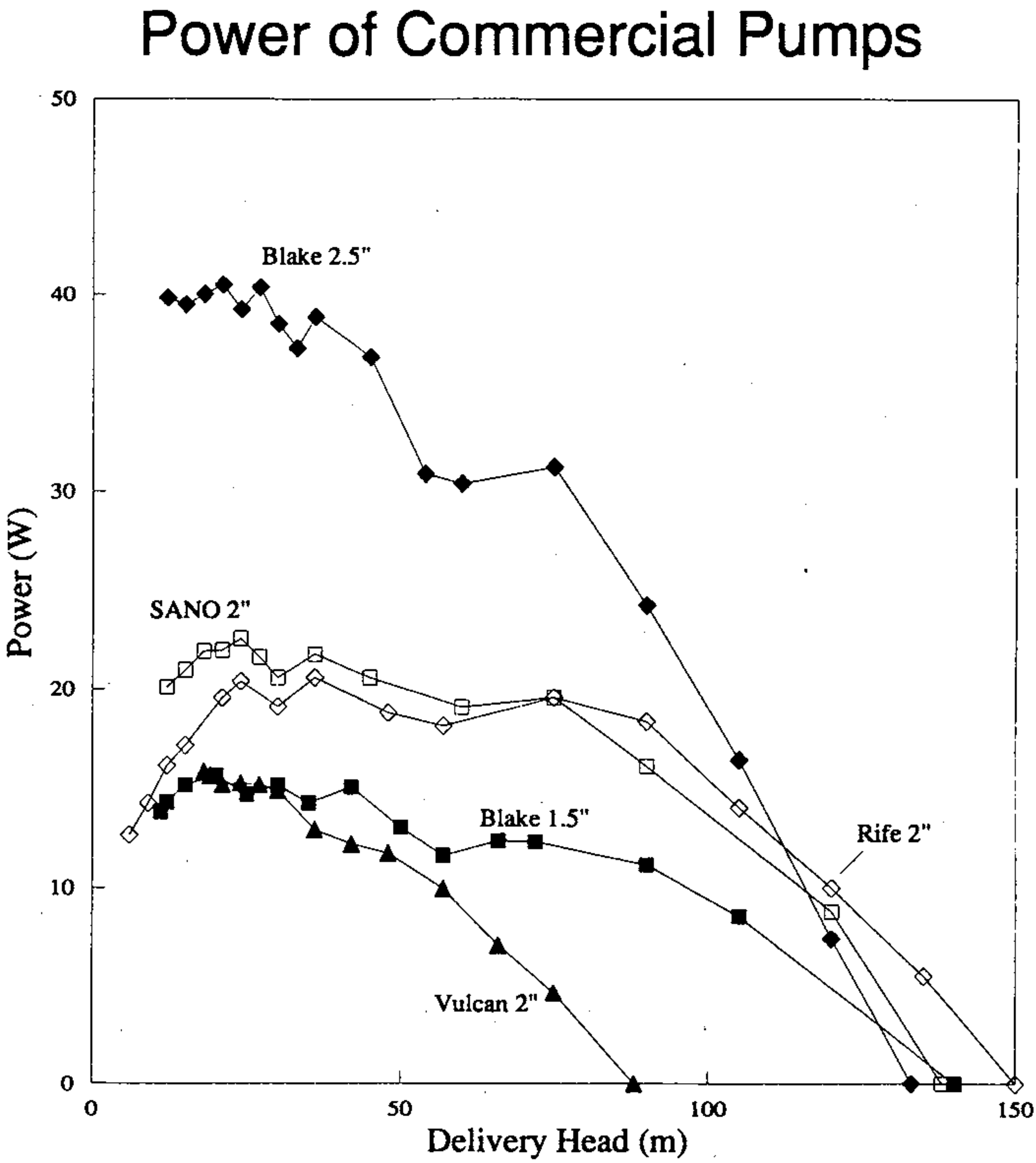
7) At low delivery heads (up to 75 m) the SANO 2" has a slightly higher output than the Rife 2" which above 75 m is better. However there is little to choose between them over the complete range of operation.

8) Attempting to combine both power and efficiency it would seem reasonable to conclude that the Blake 2½" offers the best overall performance. Of the remainder the SANO 2" would seem to be the best compromise.

Efficiency of Commercial Pumps



GRAPH 2



Power of Commercial Pumps
Blake 2.5"
SANO 2"
Blake 1.5"
Rife 2"
Vulcan 2"
Power (W)
Delivery Head (m)
0
10
20
30
40
50
0
50
100
150

2.5 Non-Dimensional Comparison

Delft use a further means of pump comparison by graphically presenting the ratio of delivery to supply flow (q/Q) over a range of delivery to supply head (h/H) ratios. Thus a non dimensional comparison between pumps is possible. For any given head ratio the greater the ratio of flows the better the perfomace of the pump. Graph 3 shows these ratio curves for the five pumps chosen. Only two conclusions can sensibly be drawn from this comparison:

1) The Rife 2" is notably worse than all its rivals.

2) There is little to choose between all of the other makes of pump.

Despite this lack of obvious conclusions the ability to compare pumps using non dimensional parameters may prove valuable.

3. SUMMARY OF DTU TESTS

3.1 Experimental rig and procedure

The DTU has established performance testing rigs at the University to allow comprehensive analysis of prototype pumps. The major restrictions imposed by the rigs location are:

i) drive pipe length - limited to 10.5 m and horizontal

ii) drive head - restricted to 2, 3, 4 or 5 m

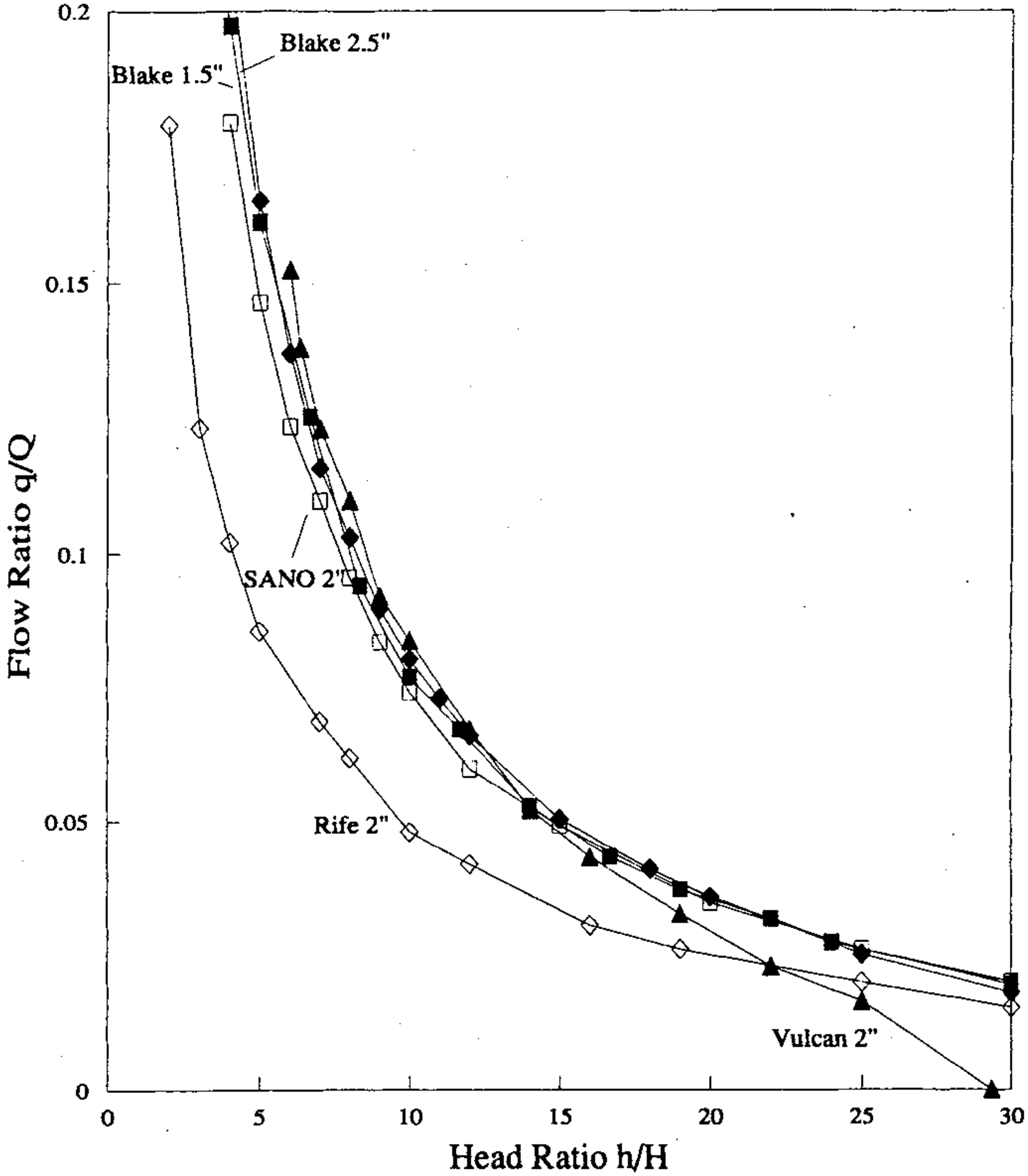
The delivery head is controlled using a needle valve providing an accurately variable orifice over which the desired head can be dropped (measured by a pressure guage). Both drive and delivery flows are measured by float type flow meters for fast and accurate readings.

For their tests Delft used three drive head setting, 1, 2 and 3 m. However for comparison between DTU and commercial pumps only one drive head was chosen (3.0 m).

3.2 Pump Tuning

The DTU pump design selected for testing was the Mark 6.4 which typically uses a 2" BSP drive pipe but also runs using 1½" pipe (see Figure 1). As the Delft work gives no clear indication of how pumps were tuned a series of results were recorded using different settings of the impulse valve. Low stroke, low weight settings generally give high efficiency and low power output. Up to a point it is also true to say that high stroke,

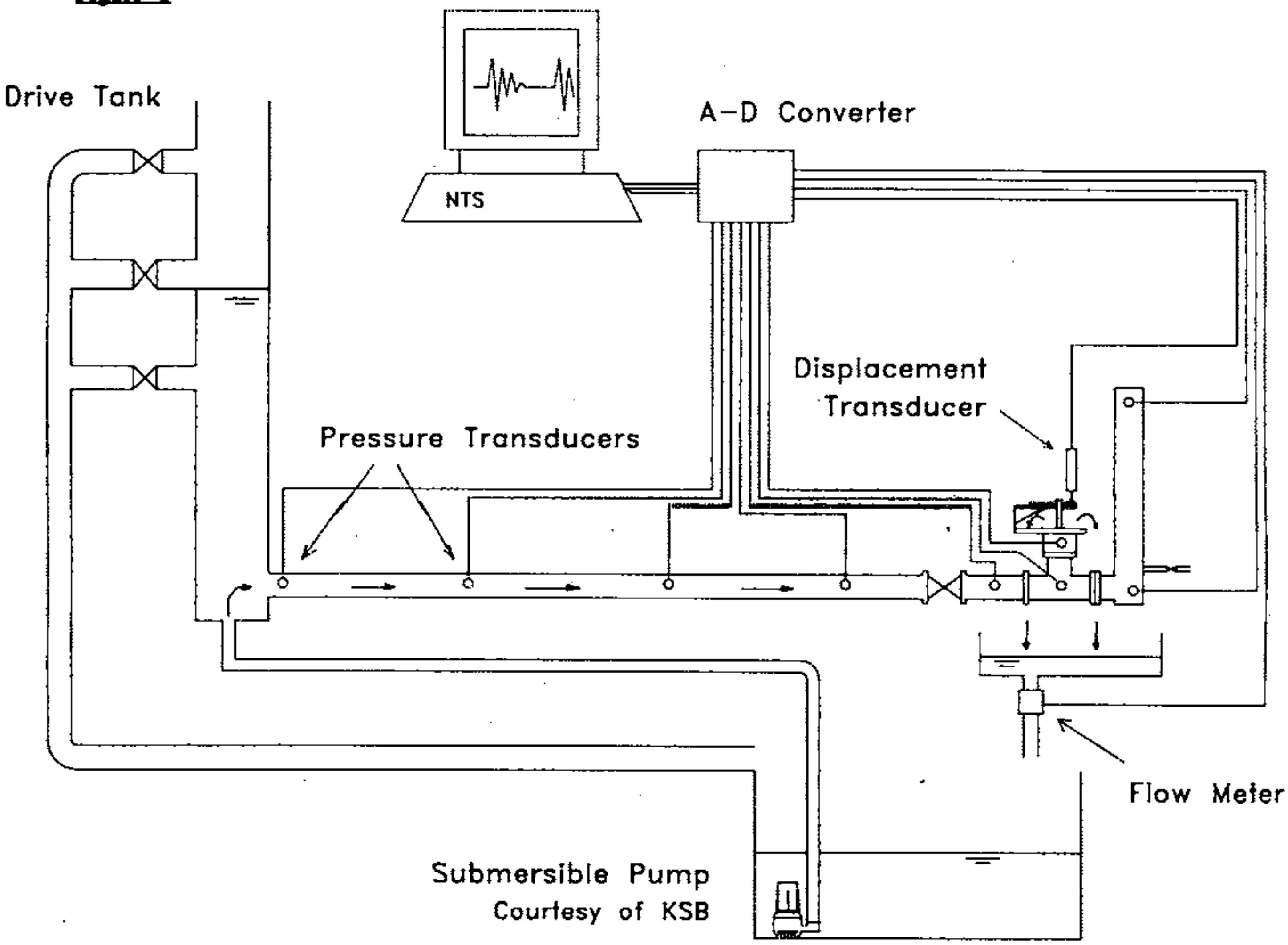
Ratio Graph of Commercial Pumps



high weight settings give low efficiency and high output power.

Choosing which of these results are used to compare against the Delft ones is dealt with in Section 4.1.

Figure 1



3.3 Summary of test results

Table 5 gives a selection of results for identical conditions as those in Table 4 for the commercial pumps.

Graphs 4 and 5 show the variation in efficiency and power over the full range of potential delivery heads.

Graph 6 presents the non dimensional flow ratio to head ratio curves for these results.

The design of the M6.4 pump allows it to be tuned to suit a wide range of flow and head conditions, making one model applicable to many sites. The results show the wide range of efficiency and power obtainable under these given conditions and emphasize how important tuning can be. If drive water is limited then peak efficiency will be required to make best use of that available. If there is plenty of drive water pumps should be tuned to give maximum power output despite the lower efficiency of such a setting.

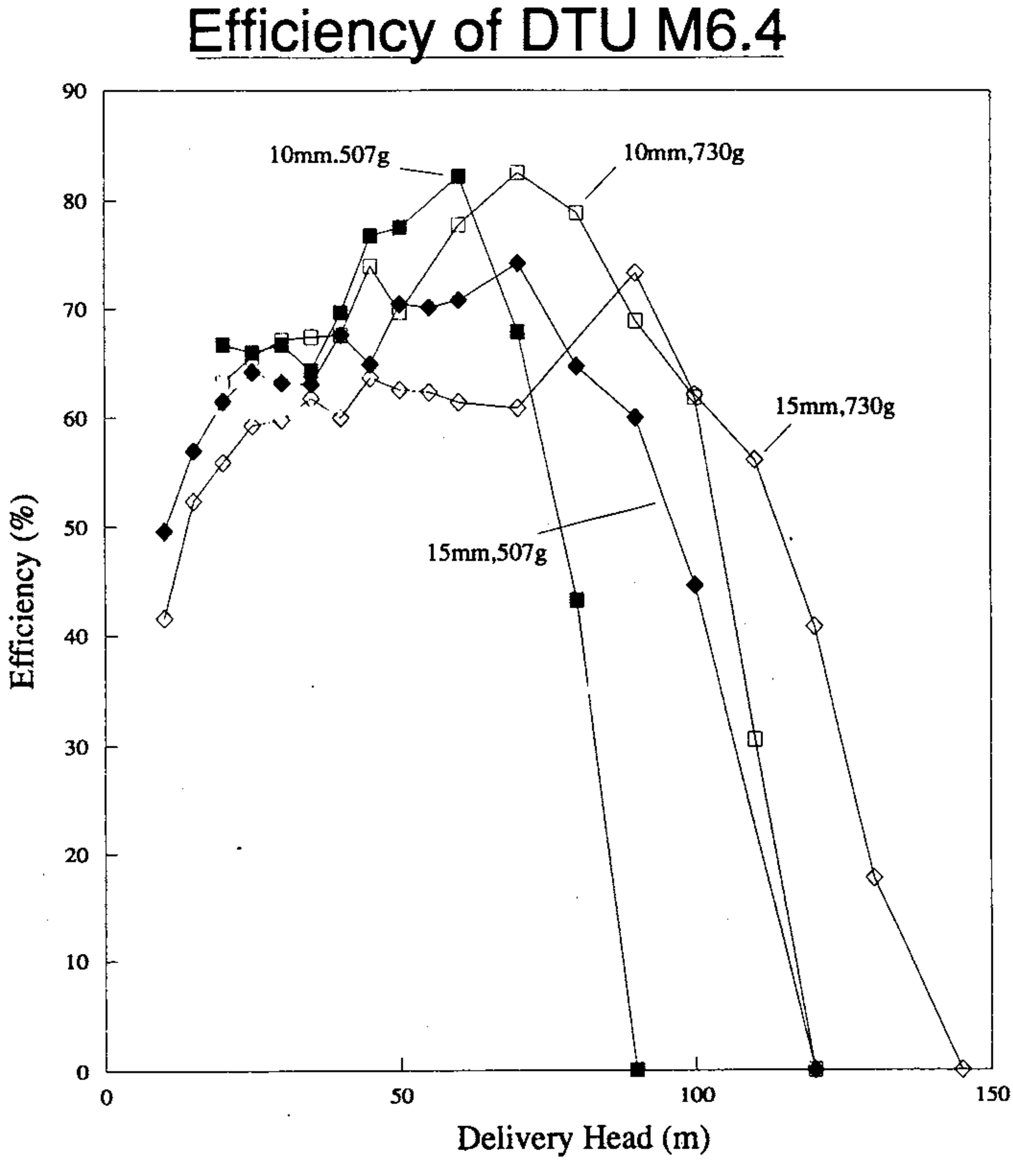
It is clear that both good efficiency and power are obtainable over a broad range of delivery heads and that the pump is capable of operating at very high head ratios.
The stroke of teh M6.4 design can range between 5 mm 40 mm with infinite adustment between these limits. The minimum weight of the valve assembly is 507g comprising the plug, stem, nuts etc. although recent modification to the pump enables further reduction. Maximum weight is limited by the physical dimensions but 1000g should be considered as the upper limit.

The results show for just a few combinations of stroke and weight how the efficiency and power output characteristics of the pump can be dramatically altered. Low stroke and weight give high efficiency but low power over a limited range of delivery heads whereas high stroke and weight give lower efficiency but high power output over a wider possible range of heads. Tuning of the pump to best suit any particular set of conditions is a complicated process to explain and is dealt with in other DTU literature.

TABLE 5 SUMMARY OF DTU M6.4 TEST RESULTS

	Pump Settings			Efficiency			Power		
	Drive Pipe Dia(")	Stroke Length (mm)	Weight (g)	at 30m	at 60m	max	at 30m	at 60m	max
1)	1.5	10	507	68	83.3	88.9	16.7	17.2	18.3
2)	1.5	15	507	59.7	64.8	68.7	22.6	22.6	24.6
3)	1.5	20	507	50	54.7	60.4	24.5	25.5	28.1
4)	2	10	507	66.7	82.2	82.2	15.7	18.1	18.1
5)	2	15	507	63.2	70.8	74.2	23.5	22.6	24.5
6)	2	20	507	58	62.5	65.9	28.5	29.4	30.1
7)	2	10	730	67.1	77.8	82.6	24	24	27.5
8)	2	15	730	59.8	61.4	73.3	30.2	30.4	32.4
9)	2	20	730	51.9	58.7	61.6	33.4	36.3	36.3

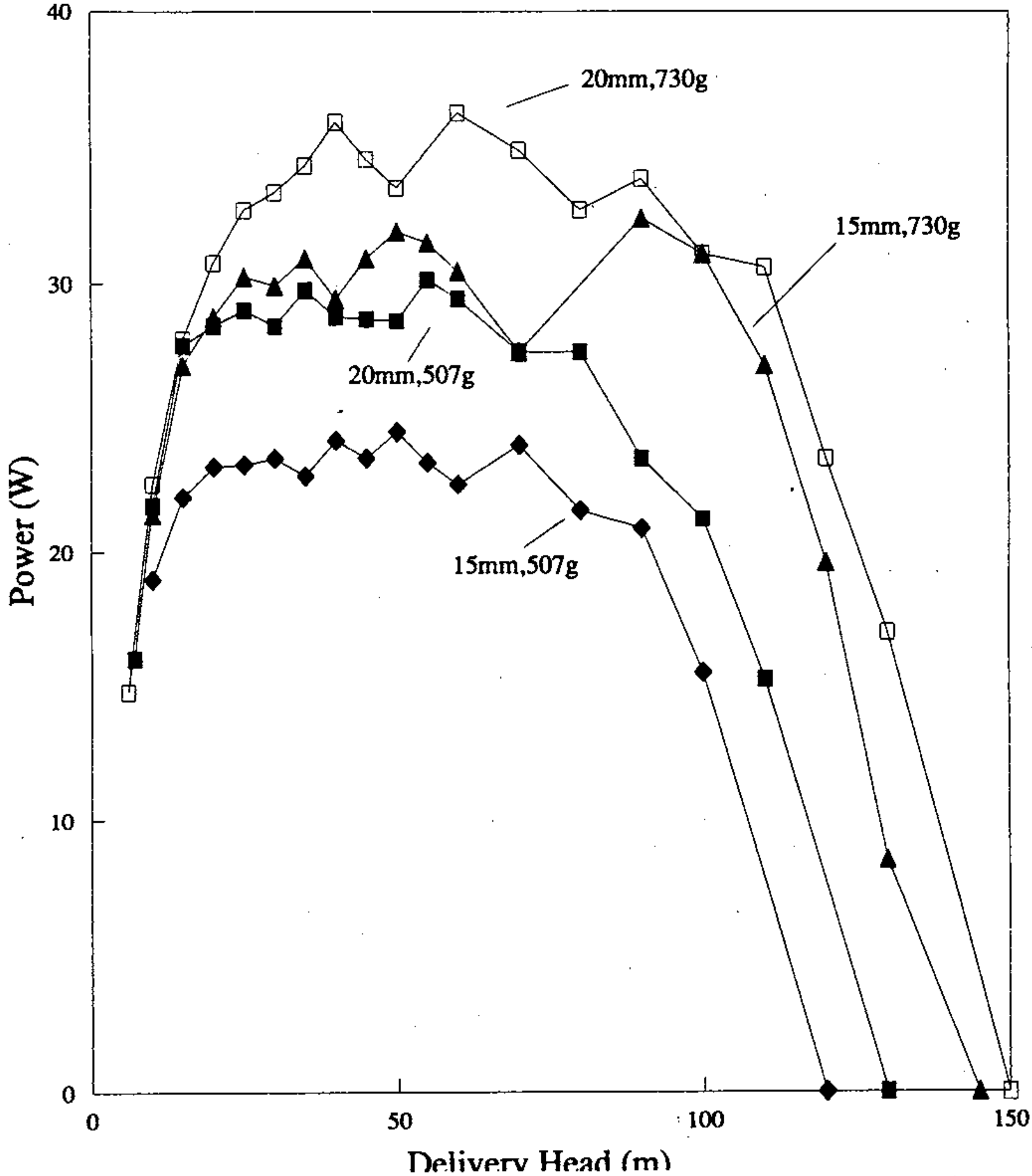
GRAPH 4

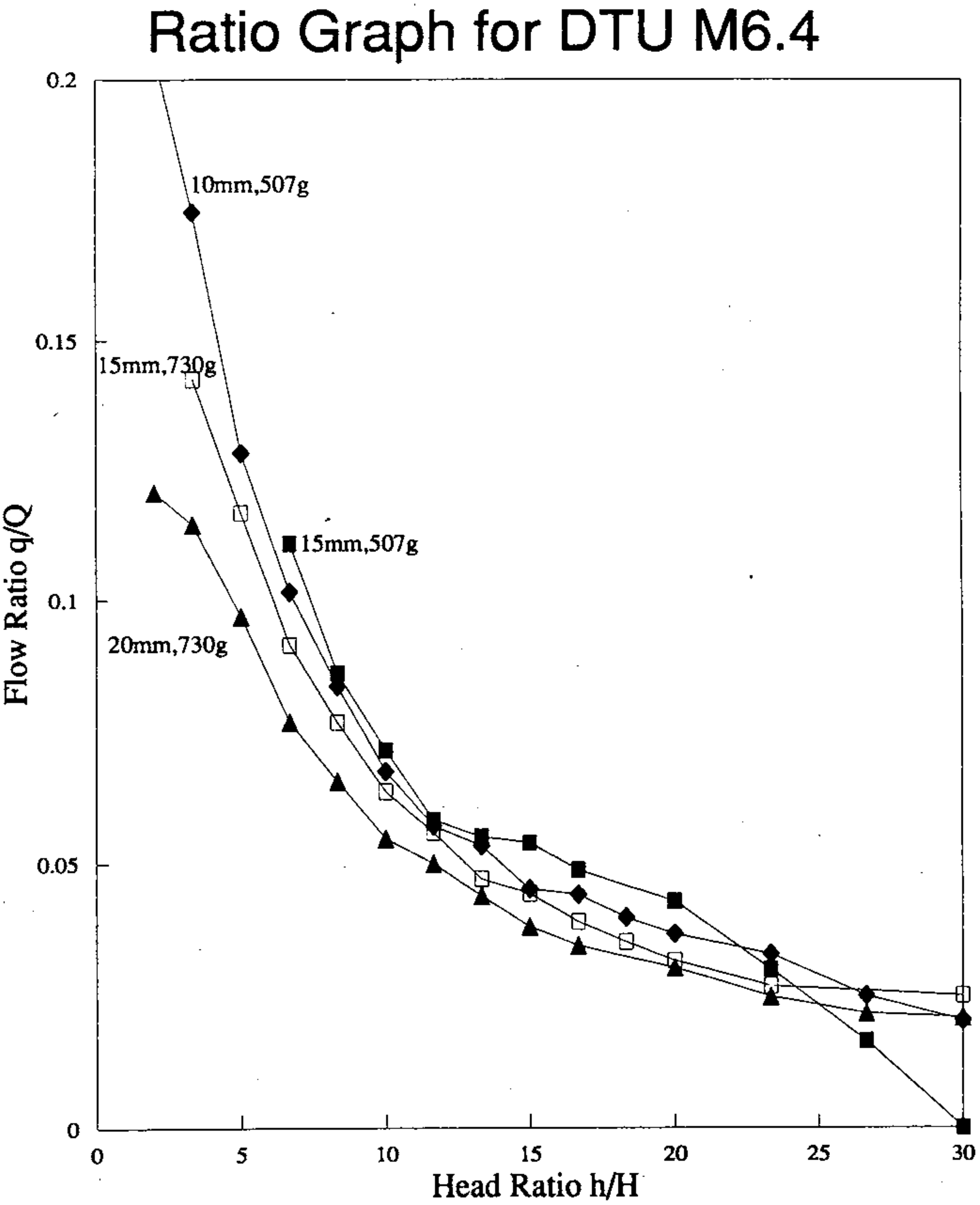


Efficiency of DTU M6.4
Efficiency (%)
Delivery Head (m)
10mm.507g
10mm,730g
15mm,730g
15mm,507g
0
10
20
30
40
50
60
70
80
90
0
50
100
150

GRAPH 5

Power of DTU M6.4




Ratio Graph for DTU M6.4
0.2
0.15
0.1
0.05
0
Flow Ratio q/Q
10mm,507g
15mm,730g
15mm,507g
20mm,730g
0
5
10
15
20
25
30
Head Ratio h/H

4. COMPARISON OF DTU AND COMMERCIAL PUMPS

4.1 Problems in comparison

There are inevitably problems and potential inaccuracies in taking data from two separate sets of tests carried out on two different test rigs. The main areas of difficulty are outlined below along with some explanation of their significance and potential methods for overcoming them.

a) The drive pipe lengths of the two test rigs differ by 1.5 m with the Delft rig using an inclined pipe whereas the Warwick tests use a horizontal one. The length of the drive pipe of a hydraulic ram pump system affects (among other things) the time taken for pressure waves to traverse the length of the pipe, total friction in the system and the energy available for pumping. The exact effects of these parameters on pump performance are complicated to evaluate and given the relatively small difference can be assumed to have no major effect for the purposes of this comparison.

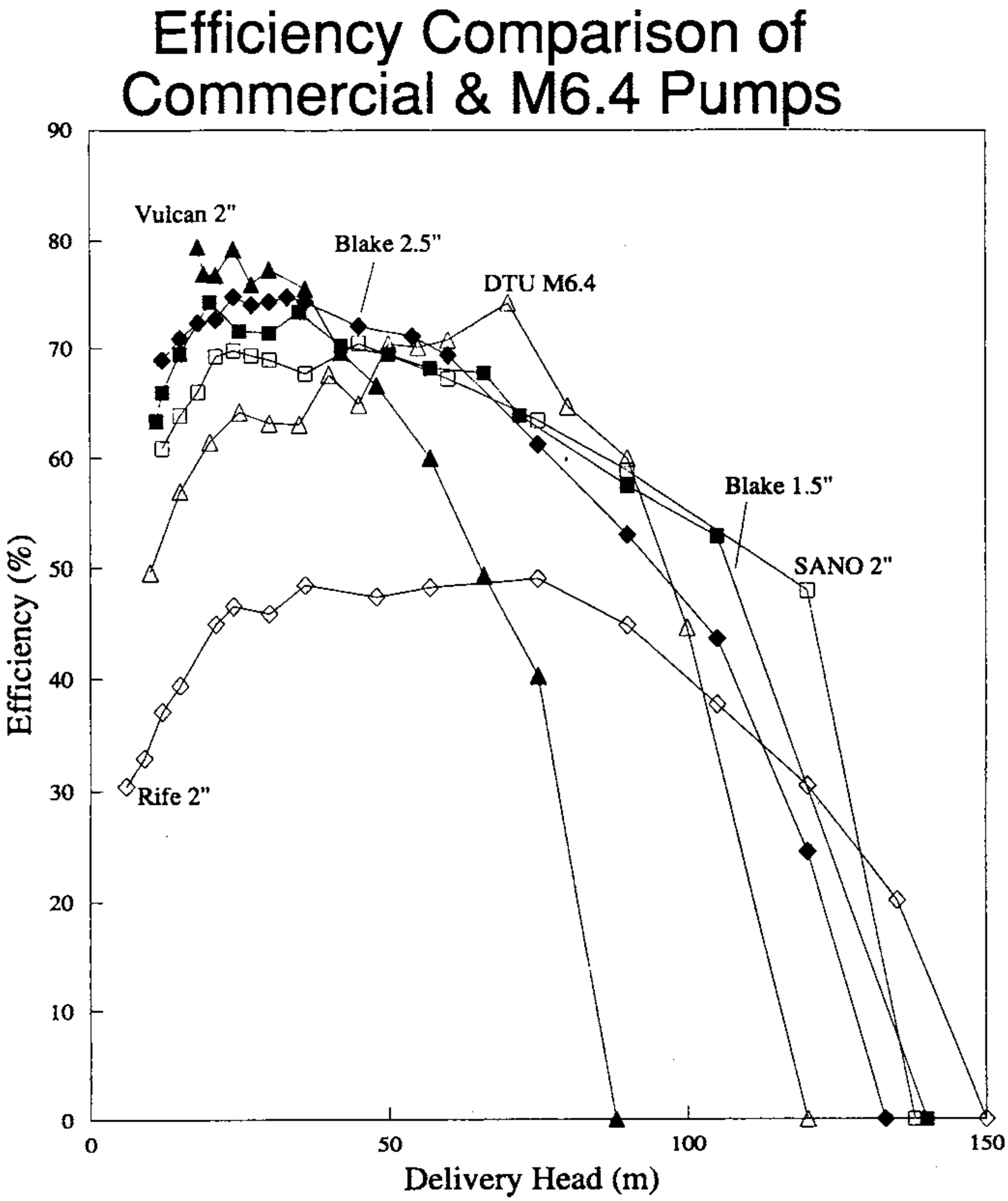
b) As has already been mentioned, the Delft work is unclear about the tuning of each of the pumps tested, other than the fact that they were left constant throughout testing once installed. To allow a sensible comparison it has been assumed that the commercial pumps tested by Delft were each set to some recommended point that gave a reasonable efficiency and power output across a broad range of conditions. In order to compare the DTU pump a best average setting from those taken has been chosen and also comparisons of the optimum settings for efficiency and power. The setting chosen for the general comparison is labelled as No. 5 in Table 5 using a standard 2" drive pipe, a valve stroke of 15 mm and weight of 507 g.

4.2 Efficiency comparisons

The efficiency of the DTU pump is of the same order as the better of the commercial models. Graph 7 shows the chosen average and peak efficiency settings of M6.4 against the 5 commercial pumps.

In a typical operating range say 20-80 m the DTU average setting gives efficiencies ranging between approx 63% and 74%. The DTU peak setting of those chosen returns results for the same range between 64% and 83%. Detailing the comparison between these settings and those for the commercial models is best don visually. The main points that can be drawn are:

GRAPH 7


Efficiency Comparison of
Commercial & M6.4 Pumps
Vulcan 2"
Blake 2.5"
DTU M6.4
Blake 1.5"
SANO 2"
Rife 2"
Efficiency (%)
Delivery Head (m)
0
10
20
30
40
50
60
70
80
90
0
50
100
150

a) The Blakes machines have the best efficiency over the widest range of delivery heads with between 62% and 75% in the 20-80 m operating band. This is very similar the DTU average setting although the shape of the curves is somewhat different giving maximum and minimum efficiencies at different heads.

b) The DTU M6.4 pump has a markedly different efficiency profile from all the commercial models with peak efficiency occurring at much higher heads.

c) The DTU M6.4 is capable of operating over a range of delivery heads as great as any of the commercial models.

Separate tests to determine the peak efficiency of the M6.4 have shown that it is capable of running at efficiencies of over 90%.

4.3 Power Comparison

Graph 8 shows the chosen average and peak power settings for the M6.4 against the 5 commercial pumps.

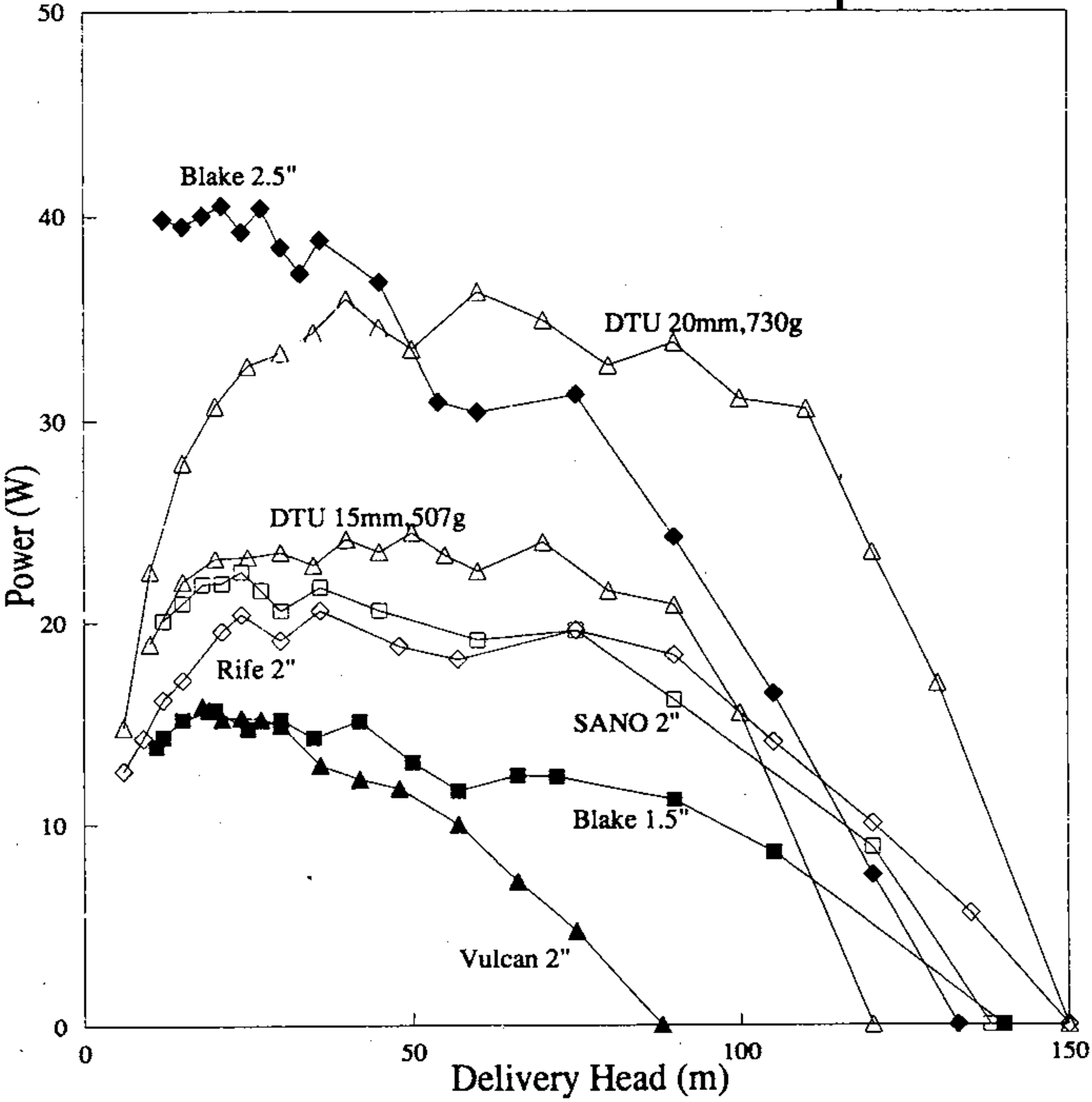
a) The average setting of the DTU M6.4 is clearly more powerful over a very wide range of delivery heads than all of the commercial models of the same size. The Blake 2½" model is clearly more powerful as should be expected from this considerably larger capacity pump.

b) The peak setting of the M6.4 used gives power output similar to and, at higher heads, better than even the Blake 2½" model. Increased weight on the M6.4 would further increase the power output although the limitation of the pipe size and drive head would probably limit peak power to around 40 watts.

5. COST COMPARISON

Table 2 in section 2.1 shows the costs of 10 rams in US$ in 1982 as given by Delft. These costs are for the pumps alone with no drive pipe, delivery pipe, shipping etc. included, and range from US$ 1000 to 3500. No updated prices are available for comparison with the DTU pump so an annual inflation rate of 5% has been assumed. A comparative cost for the DTU pumps is hard to ascertain as they have only been manufactured as one-off prototypes to date in the UK. The design of all the DTU pumps is intended to allow manufacture in the country of use, avoiding any shipping and importation problems but working to the constraints imposed in non-industrialised areas.

Power Comparison of Commercial & DTU Pumps



The Baptist Community of Western Zaire (CBZO) have a village water supply programme installing the DTU M6.4 in its rural areas. Currently the manufacture of these pumps is being contracted to a workshop in Kinshasa who are producing them in small batches as requested. The total cost of these units in Zaire is approx. US$450 but once proper manufacture has started it is estimated that this will reduce to $250-$300.

The actual cost to an end user in a developing country of a ram pump will include any shipping and importation costs if it is made eslewhere. To accurately compare the DTU pumps made in country with imported commercial models the costs associated with transportation should be included. However such costs are so dependant upon the shipping distance, customs duties etc. that they would be impossible to estimate with any accuracy.

Table 6 takes the information from Table 2 and adds to the costs an allowance for inflation. It also includes information about the DTU Mark 6.4 pump for the conditions specified by Delft. The information concerning expected delivery flow was arrived at by projecting measured performance figures for different conditions. The DTU is in the process of developing a sophisticated computer simulation and model which are being used in pump analysis and will form the basis of comprehensive design charts. Ultimately this will be able to predict the performance of any pump that has been suitably calibrated under any set of conditions with a good degree of accuracy.

The predicted performance of the DTU pump ranks it 5th in terms of delivery flow and efficiency. The comparison of costs shows the vast difference in the price of the pumping unit and the cost per litre delivered. This illustrates the legitimacy of the approach of producing high performance pumps at low cost in the country of use. If a true comparison to include transportation costs were made the results would clearly be more conclusive still.

TABLE 6 - COST COMPARISON

Type of Hydraulic Ram	Pumping Rate q [l/min]	Efficiency (Trade) (%)	Approx Price for Ram Alone [US$-1982]	Cost per litre delivered	Rank
Vulcan 21/2	6.10	68	1800	295	4
Blake Hydram 31/2	6.00	67	1500	250	3
Sano No. 5/65 mm	6.95	77	1500	216	2
Rife 20 HDU	5.40	60	1650	306	5
Schlumpf 5A23	5.50	61	4050	737	9
Alto CH 66-110-18	5.40	60	3150	584	7
Briau D4	5.40	60	4800	890	11
CeCoCo - H50	6.90	77	5250	761	10
WAMA No. 6	4.50	50	2250	500	6
BZH-Ram W6	2.70	30	1800	666	8
DTU M6.4 2"	5.5	65	300	55	1

6. SUMMARY OF CONCLUSION

The following points summarise the conclusions that can be drawn from these tests.

a) The DTU Mark 6.4 pump gives efficiencies comparable with the more efficient commercial pumps over a wide range of condition.

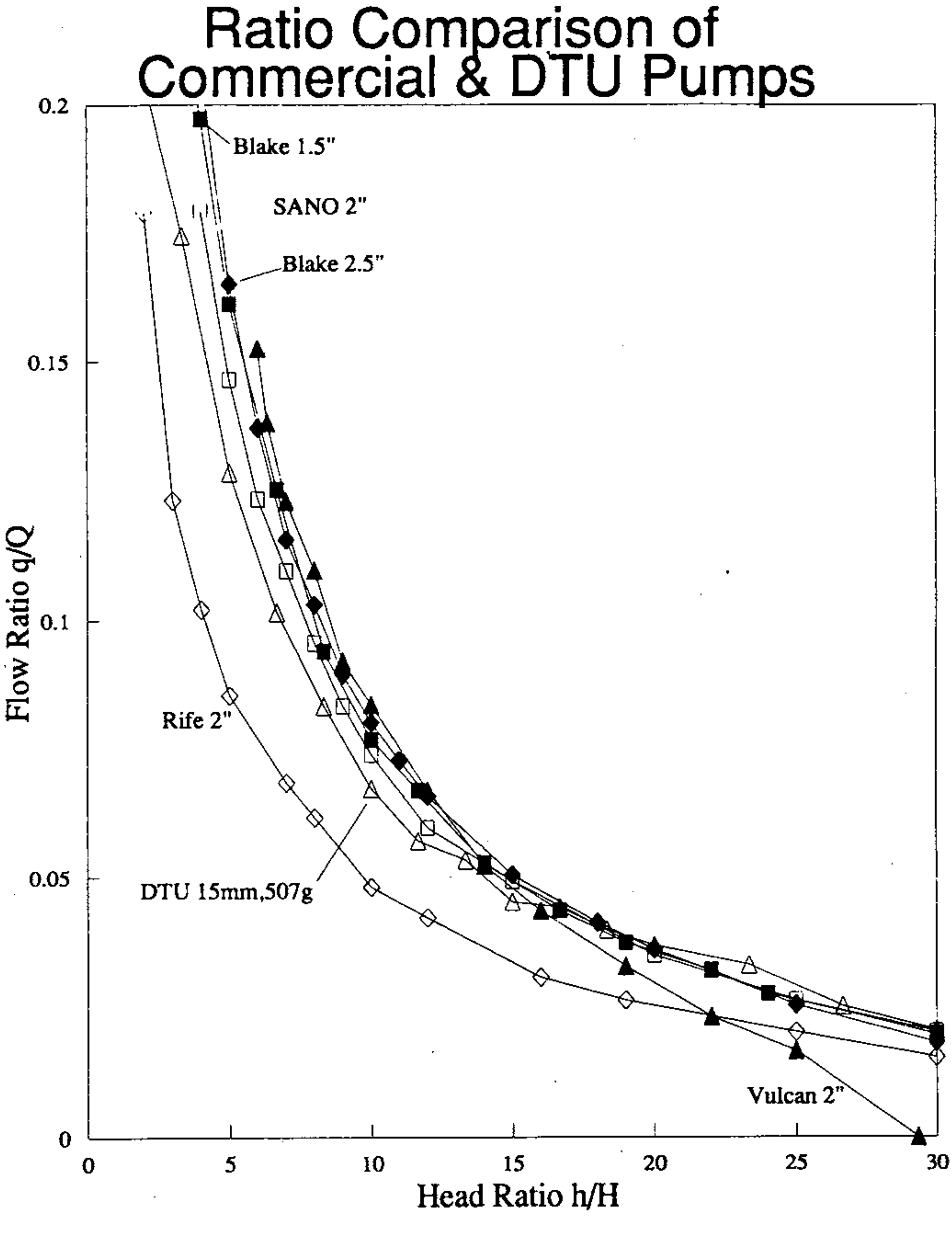
b) The M.6.4 produces a higher output power than all commercial models of the same size over a wide range of conditions.

c) The M6.4 has a cost to output ratio considerably lower than all of its commercial rivals.

The tests show that it is possible to produce simple and comparatively cheap hydraulic ram pumps that can match and exceed the performance of commericial models. Whilst endurance tests to date indicate that the DTU pumps are likely to exhibit adequate durability no data is yet available for a long term comparison with commercial rivals. The current design specification is to produce low maintenance pumps whose steel components have a 10 year life and rubber components last approximately 6 months.

More detailed information concerning the design, manufacture, installation and performace of DTU pumps is available on request.

GRAPH 9

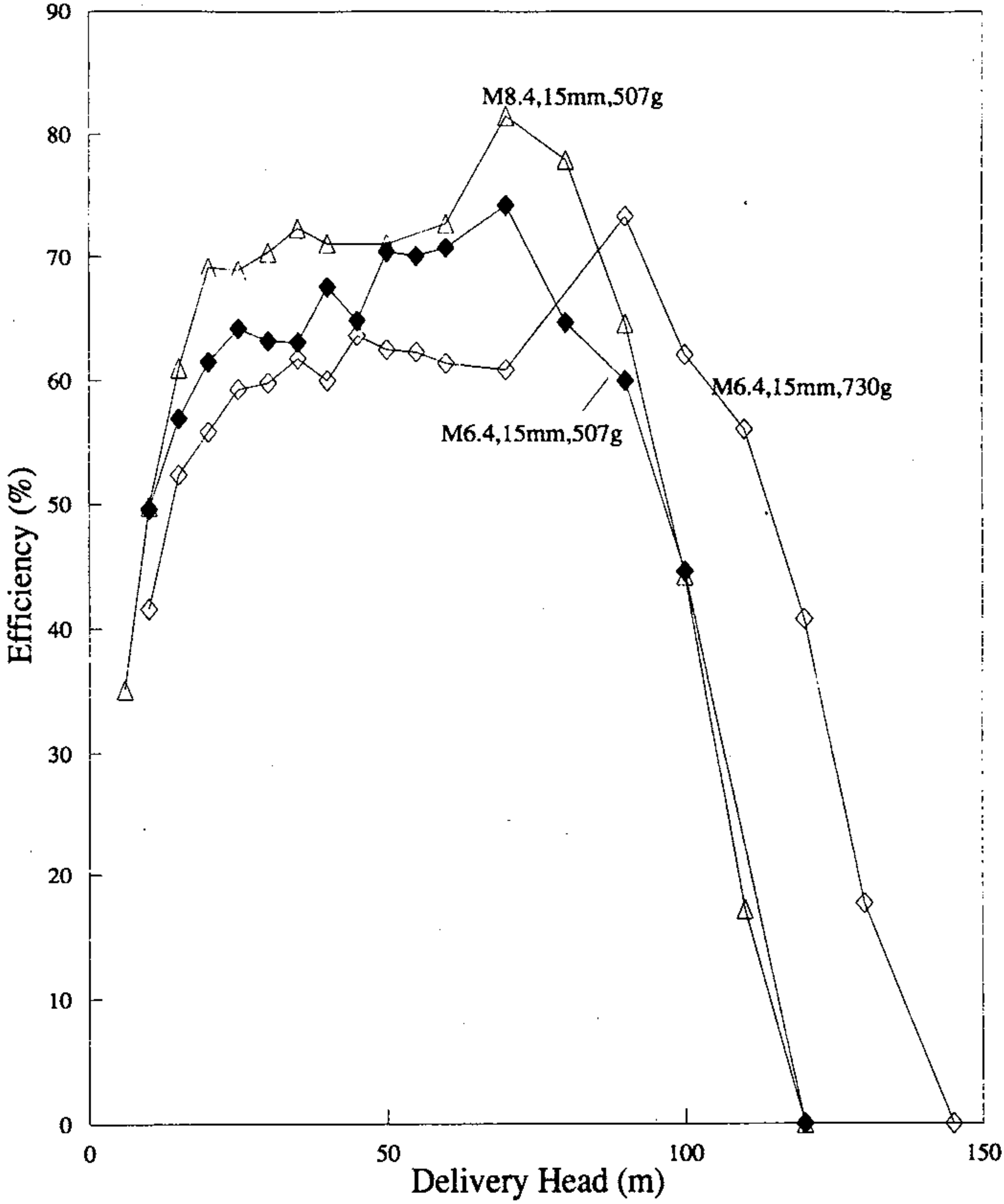

Ratio Comparison of
Commercial & DTU Pumps
Blake 1.5"
SANO 2"
Blake 2.5"
Rife 2"
DTU 15mm,507g
Vulcan 2"
Flow Ratio q/Q
Head Ratio h/H
0.2
0.15
0.1
0.05
0
0
5
10
15
20
25
30

Appendix A **CONTINUING DTU PUMP DEVELOPMENT**

The Mark 6.4 pump was set as a DTU standard in 1990 to allow organisations interested in using the pummp to have a standard model to work with. Since that time a number of developments have occurred in the on-going research programme and new design initiatives produced. One such is the development of the Mark 8.4 that uses the same valves as the M6.4 but replaces the 2" fittings used in the pump body witha body of 4" velded construction. This alteration dramatically reduces the peak overpressure experienced the system, damps out the high frequency oscillations experienced during the cycle and increases both the efficiency and power output for any given set of conditions.

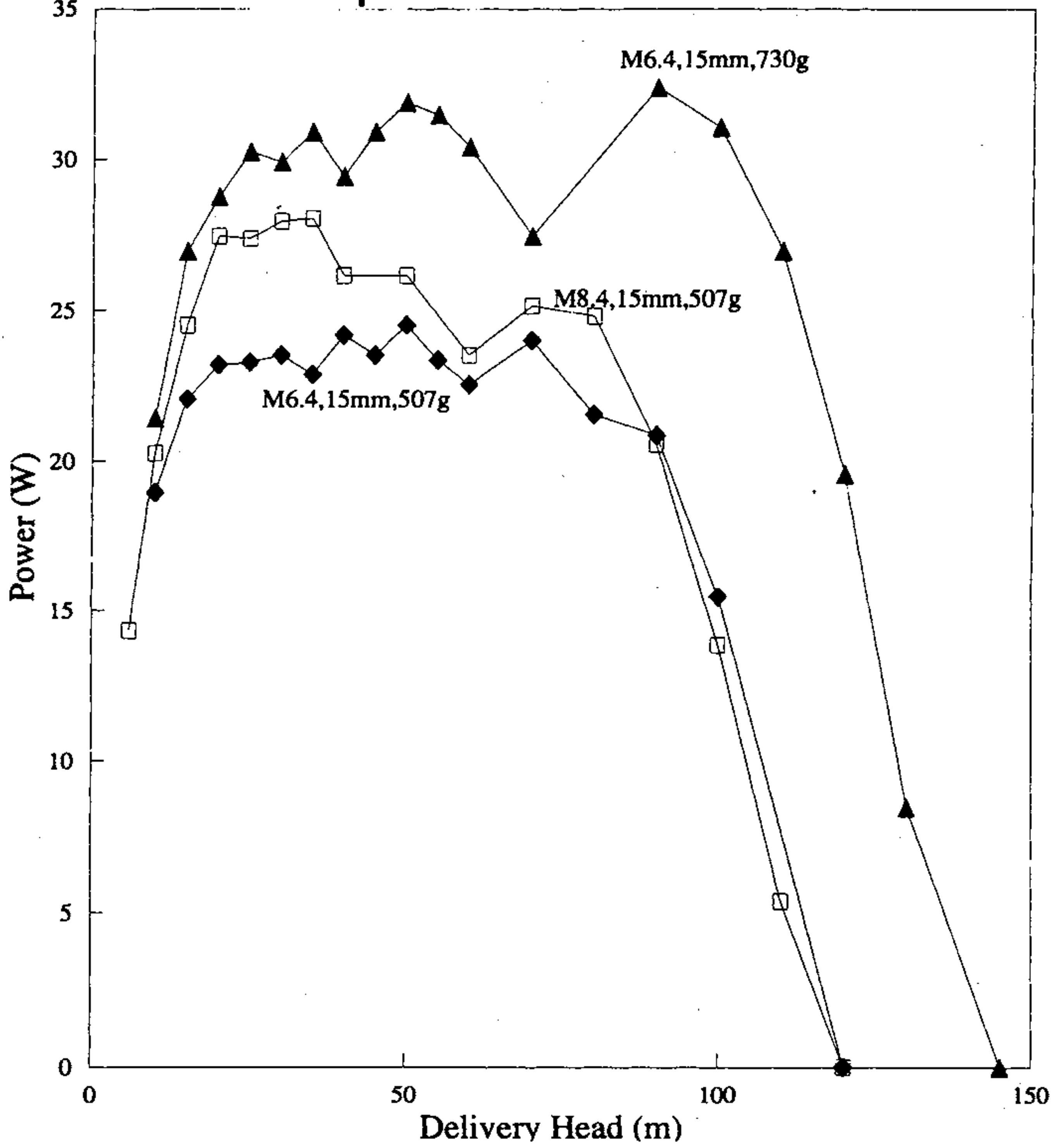
Graphs 10 and 11 allow comparison of the M8.4 with the M6.4 under identical conditions.

Efficiency Comparison of M6.4 & M8.4



GRAPH 11

Power Comparison of M6.4 & M8.4



Appendix B **TABLES OF PERFORMANCE RESULTS**

Tablulated results used in production of Graphs.

The following pages list the information supplied by Delft and that gained under similar conditions on the DTU pump. In all cases the **Supply Head = 3m.**

1) Blake Hydram No2 Drive pipe dia = 1.5" Supply Head = 3.00m

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
11	3.666667	0.686	7.7	36.9	0.208672	55.64589	63.30344	13.84845
12	4	0.686	7.3	37	0.197297	59.18919	65.91422	14.3226
15	5	0.663	6.2	38.45	0.161248	64.49935	69.42889	15.2055
20	6.666667	0.667	4.8	38.3	0.125326	71.01828	74.24594	15.696
25	8.333333	0.669	3.6	38.3	0.093995	68.9295	71.59905	14.715
30	10	0.645	3.1	40.3	0.076923	69.23077	71.42857	15.2055
35	11.66667	0.683	2.5	37.25	0.067114	71.58837	73.37526	14.30625
42	14	0.64	2.2	41.65	0.052821	68.66747	70.23945	15.1074
50	16.66667	0.696	1.6	36.8	0.043478	68.11594	69.44444	13.08
57	19	0.737	1.25	33.6	0.037202	66.96429	68.14921	11.64938
66	22	0.714	1.15	36.2	0.031768	66.71271	67.73762	12.40965
72	24	0.695	1.05	38.4	0.027344	62.89062	63.87833	12.3606
90	30	0.697	0.76	38.9	0.019537	56.6581	57.48865	11.1834
105	35	0.758	0.5	32.6	0.015337	52.14724	52.87009	8.58375
140	46.66667		0		0	0	0	0

2) Blake Hydram No 3.5 Drive pipe dia = 2.5" Supply Head = 3.00m

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
12	4	0.671	20.3	97.6	0.207992	62.39754	68.87193	39.8286
15	5	0.664	16.1	97.5	0.165128	66.05128	70.86268	39.48525
18	6	0.657	13.6	99.2	0.137097	68.54839	72.34043	40.0248
21	7	0.646	11.8	101.9	0.1158	69.47988	72.64732	40.5153
24	8	0.667	10	97	0.103093	72.16495	74.76636	39.24
27	9	0.633	9.15	102.1	0.089618	71.69442	74.02247	40.39268
30	10	0.664	7.85	97.8	0.080266	72.23926	74.30194	38.50425
33	11	0.686	6.9	94.6	0.072939	72.93869	74.77833	37.22895
36	12	0.652	6.6	100	0.066	72.6	74.29644	38.8476
45	15	0.672	5	99.1	0.050454	70.63572	72.04611	36.7875
54	18	0.735	3.5	85.1	0.041128	69.91774	71.10609	30.9015
60	20	0.727	3.1	86.3	0.035921	68.25029	69.35123	30.411
75	25	0.644	2.55	101.5	0.025123	60.29557	61.26862	31.26938
90	30	0.697	1.65	91.7	0.017993	52.18103	53.02625	24.27975
105	35	0.761	0.96	76.1	0.012615	42.89093	43.60239	16.4808
120	40	0.828	0.38	61.7	0.006159	24.01945	24.48454	7.4556
133	44.33333	0.88	0	50.3	0	0	0	0

3) Vulcan 2" Drive pipe dia = 2" Supply Head = 3.00m

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
18	6	0.422	5.4	35.4	0.152542	76.27119	79.41176	15.8922
19	6.333333	0.41	5.05	36.55	0.138167	73.68901	76.88301	15.68783
21	7	0.425	4.44	36.05	0.123162	73.89736	76.75969	15.24474
24	8	0.428	3.9	35.5	0.109859	76.90141	79.18782	15.3036
27	9	0.413	3.45	37.45	0.092123	73.69826	75.91687	15.23003
30	10	0.42	3.05	36.4	0.083791	75.41209	77.31305	14.96025
36	12	0.455	2.2	32.75	0.067176	73.89313	75.53648	12.9492
42	14	0.445	1.78	34	0.052353	68.05882	69.64785	12.22326
48	16	0.44	1.5	34.55	0.043415	65.12301	66.5742	11.772
57	19	0.453	1.07	32.8	0.032622	58.71951	60.02362	9.971865
66	22	0.481	0.66	28.8	0.022917	48.125	49.28717	7.12206
75	25	0.522	0.38	23.2	0.016379	39.31034	40.28838	4.65975
88	29.33333	0	0	0	0	0	0	0

4) SANO No4 2" Drive pipe dia = 2" Supply Head = 3.00m

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
12	4	0.656	10.25	57.1	0.17951	53.85289	60.87602	20.1105
15	5	0.645	8.55	58.4	0.146404	58.56164	63.85362	20.96888
18	6	0.622	7.45	60.3	0.123549	61.77446	65.97786	21.92535
21	7	0.641	6.4	58.3	0.109777	65.86621	69.24266	21.9744
24	8	0.618	5.75	60.15	0.095594	66.91604	69.80273	22.563
27	9	0.633	4.9	58.7	0.083475	66.78024	69.33962	21.63105
30	10	0.648	4.2	56.7	0.074074	66.66667	68.96552	20.601
36	12	0.608	3.7	61.9	0.059774	65.75121	67.68293	21.7782
45	15	0.653	2.8	56.8	0.049296	69.01408	70.4698	20.601
60	20	0.663	1.95	56.1	0.034759	66.04278	67.18346	19.1295
75	25	0.613	1.6	61.45	0.026037	62.48983	63.44171	19.62
90	30	0.665	1.1	54.95	0.020018	58.05278	58.876	16.1865
120	40	0.798	0.45	37.1	0.012129	47.30458	47.93609	8.829
138	46		0		0	0	0	0

5) Rife 20HDU 2" Drive pipe dia = 2" Supply Head = 3.00m

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
6	2	0.965	12.9	72	0.179167	17.91667	30.38869	12.6549
9	3	0.882	9.7	78.7	0.123253	24.65057	32.91855	14.27355
12	4	0.865	8.25	80.8	0.102104	30.63119	37.05783	16.1865
15	5	0.853	7	81.85	0.085522	34.20892	39.39223	17.1675
21	7	0.847	5.7	83.1	0.068592	41.15523	44.93243	19.57095
24	8	0.828	5.2	84.2	0.061758	43.2304	46.53244	20.4048
30	10	0.858	3.9	81.1	0.048089	43.2799	45.88235	19.1295
36	12	0.835	3.5	83.25	0.042042	46.24625	48.41499	20.601
48	16	0.866	2.4	78.7	0.030496	45.74333	47.34895	18.8352
57	19	0.89	1.95	74.9	0.026035	46.86248	48.2108	18.17303
75	25	0.858	1.6	80	0.02	48	49.01961	19.62
90	30	0.84	1.25	82.4	0.01517	43.99272	44.82965	18.39375
105	35	0.888	0.82	75.3	0.01089	37.02523	37.70363	14.07735
120	40	0.947	0.51	66.5	0.007669	29.90977	30.44322	10.0062
135	45	0.999	0.25	55.75	0.004484	19.73094	20.08929	5.518125
154	51.33333		0		0	0	0	0

6) DTU M6.4 2" 1.5" Drive pipe Stroke = 10mm Weight 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
8	2.666667	0.965	10	52	0.238095	39.68254	43.01075	13.08
12	4		8.1	51	0.188811	56.64336	54.82234	15.8922
15	5		6.8	52	0.150442	60.17699	57.82313	16.677
20	6.666667		5.2	51	0.113537	64.3377	61.68446	17.004
25	8.333333		4.2	52	0.087866	64.43515	62.27758	17.1675
30	10		3.4	50	0.072961	65.66524	63.67041	16.677
35	11.66667		3	51	0.0625	66.66667	64.81481	17.1675
40	13.33333		2.4	48	0.052632	64.91228	63.49206	15.696
45	15		2.2	48	0.048035	67.24891	65.73705	16.1865
50	16.66667		2.1	47	0.046771	73.27394	71.2831	17.1675
56	18.66667		2	42	0.05	88.33333	84.84848	18.312
60	20		1.75	42	0.043478	82.6087	80	17.1675
70	23.33333		1.2	42	0.029412	65.68627	64.81481	13.734
80	26.66667		0.6	41	0.014851	38.11881	38.46154	7.848
90	30		0	40	0	0	0	0

7) DTU M6.4 2" Drive pipe 1.5" Stroke 15mm Weight 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
7	2.333333	0.965	14.5	77	0.232	30.93333	36.97632	16.59525
10	3.333333		11.7	79	0.173848	40.56464	42.9989	19.1295
15	5		9	79	0.128571	51.42857	51.13636	22.0725
20	6.666667		7	78	0.098592	55.86854	54.90196	22.89
25	8.333333		5.5	78	0.075862	55.63218	54.89022	22.48125
30	10		4.6	77	0.063536	57.18232	56.37255	22.563
35	11.66667		4.05	77	0.055517	59.21864	58.29735	23.17613
40	13.33333		3.4	74	0.048159	59.39566	58.5702	22.236
45	15		3.1	76	0.042524	59.53361	58.78635	22.80825
50	16.66667		2.8	74	0.039326	61.61049	60.76389	22.89
55	18.33333		2.5	71	0.036496	63.26034	62.35828	22.48125
60	20		2.3	71	0.033479	63.6099	62.7558	22.563
70	23.33333		2.15	73	0.030346	67.77229	66.75538	24.60675
80	26.66667		1.75	68	0.026415	67.79874	66.90562	22.89
90	30		1.35	62	0.022259	64.5507	63.93054	19.86525
100	33.33333		1.15	61	0.019215	62.12754	61.67873	18.8025
110	36.66667		0.7	60	0.011804	42.1023	42.28446	12.5895
120	40		0	58	0	0	0	0

8) DTU M6.4 2" Drive pipe 1.5" Stroke 20mm Weight 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
6	2	0.965	13.5	101	0.154286	15.42857	23.58079	13.2435
10	3.333333		11.8	100	0.133787	31.21693	35.18187	19.293
15	5		9.4	100	0.103753	41.5011	42.96161	23.0535
20	6.666667		7.2	100	0.077586	43.96552	44.77612	23.544
25	8.333333		6	100	0.06383	46.80851	47.16981	24.525
30	10		4.9	98	0.052632	47.36842	47.61905	24.0345
35	11.66667		4.2	98	0.044776	47.76119	47.94521	24.0345
40	13.33333		3.9	100	0.040583	50.05203	50.04812	25.506
45	15		3.5	98	0.037037	51.85185	51.72414	25.75125
50	16.66667		3.25	97	0.034667	54.31111	54.03159	26.56875
55	18.33333		2.9	96	0.031149	53.99212	53.758	26.07825
60	20		2.6	95	0.028139	53.4632	53.27869	25.506
70	23.33333		2.3	94	0.025082	56.01599	55.72863	26.3235
80	26.66667		2.15	95	0.023156	59.43278	59.01527	28.122
90	30		1.7	93	0.01862	53.99781	53.85428	25.0155
100	33.33333		1.3	90	0.014656	47.3882	47.46258	21.255
125	41.66667		0	0	0	0	0	0

9) DTU M6.4 2" Drive pipe 2" Stroke 20mm Weight 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
7	2.333333	0.965	14	108	0.148936	19.85816	26.77596	16.023
10	3.333333		13.3	108	0.140444	32.77015	36.5485	21.7455
15	5		11.3	106	0.119324	47.72967	48.16709	27.71325
20	6.666667		8.7	105	0.090343	51.19418	51.01143	28.449
25	8.333333		7.1	103	0.074035	54.29267	53.73903	29.02125
30	10		5.8	100	0.061571	55.41401	54.82042	28.449
35	11.66667		5.2	102	0.053719	57.30028	56.59204	29.757
40	13.33333		4.4	100	0.046025	56.7643	56.19413	28.776
45	15		3.9	98	0.041445	58.02338	57.40922	28.69425
50	16.66667		3.5	97	0.037433	58.64528	58.04312	28.6125
55	18.33333		3.35	96	0.036158	62.67314	61.81849	30.12488
60	20		3	96	0.032258	61.29032	60.60606	29.43
70	23.33333		2.3	89	0.026528	59.24644	58.78058	26.3235
80	26.66667		2.1	85	0.025332	65.01809	64.29392	27.468
90	30		1.6	86	0.018957	54.9763	54.79452	23.544
100	33.33333		1.3	86	0.015348	49.62613	49.63727	21.255
110	36.66667		0.85	84	0.010222	36.46021	36.73149	15.28725
130	43.33333		0			ERR	ERR	0

10) DTU M6.4 2" Drive pipe 2" Stroke 15mm Weight 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
6	2	0.965	13.3	78	0.205564	20.55641	29.13472	13.0473
10	3.333333		11.6	78	0.174699	40.76305	43.15476	18.966
15	5		9	79	0.128571	51.42857	51.13636	22.0725
20	6.666667		7.1	77	0.101574	57.55842	56.2822	23.217
25	8.333333		5.7	74	0.083455	61.20059	59.59849	23.29875
30	10		4.8	76	0.067416	60.67416	59.40594	23.544
35	11.66667		4	74	0.057143	60.95238	59.82906	22.89
40	13.33333		3.7	73	0.053391	65.84897	64.31986	24.198
45	15		3.2	74	0.045198	63.27684	62.17617	23.544
50	16.66667		3	71	0.044118	69.11765	67.56757	24.525
55	18.33333		2.6	68	0.039755	68.90928	67.51653	23.3805
60	20		2.3	65	0.036683	69.69697	68.35067	22.563
70	23.33333		2.1	66	0.032864	73.39593	71.95301	24.0345
80	26.66667		1.65	68	0.024868	63.82818	63.17301	21.582
90	30		1.42	71	0.020408	59.18367	58.82353	20.8953
100	33.33333		0.95	71	0.013562	43.84963	44.01205	15.5325
120	40							

11) DTU M6.4 2" Drive pipe 2" Stroke 10mm Weight = 507g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
20	6.666667	0.965	4.8	48	0.111111	62.96296	66.66667	15.696
25	8.333333		3.8	48	0.085973	63.04676	65.97222	15.5325
30	10		3.2	48	0.071429	64.28571	66.66667	15.696
35	11.66667		2.7	49	0.058315	62.20302	64.28571	15.45075
40	13.33333		2.35	45	0.0551	67.95623	69.62963	15.369
45	15		2.15	42	0.053952	75.53325	76.78571	15.81863
50	16.66667		2	43	0.04878	76.42276	77.51938	16.35
60	20		1.85	45	0.042874	81.46002	82.22222	18.1485
70	23.33333		1.25	43	0.02994	66.86627	67.82946	14.30625
80	26.66667		0.6	37	0.016484	42.30769	43.24324	7.848
90	30		0	0	0	0	0	0

12) DTU M6.4 2" Drive pipe 2" Stroke 10mm Weight 730g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
20	6.666667	0.965	7.3	77	0.104735	59.34959	63.20346	23.871
25	8.333333		5.9	75	0.085384	62.61457	65.55556	24.11625
30	10		4.9	73	0.071953	64.75771	67.12329	24.0345
35	11.66667		4.1	71	0.061286	65.3712	67.37089	23.46225
40	13.33333		3.9	77	0.053352	65.80027	67.53247	25.506
45	15		3.6	73	0.051873	72.62248	73.9726	26.487
50	16.66667		2.8	67	0.043614	68.32814	69.65174	22.89
60	20		2.45	63	0.040462	76.87861	77.77778	24.0345
70	23.33333		2.3	65	0.036683	81.92451	82.5641	26.3235
80	26.66667		2.1	71	0.030479	78.22932	78.87324	27.468
90	30		1.63	71	0.023497	68.14185	68.87324	23.98545
100	33.33333		1.15	62	0.018899	61.10655	61.82796	18.8025
110	36.66667		0.5	60	0.008403	29.97199	30.55556	8.9925
120	40		0	60	0	0	0	0

13) DTU M6.4 2" Drive pipe 2" Stroke 15mm Weight 730g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
10	3.333333	0.965	13.1	105	0.142546	33.26079	41.5873	21.4185
15	5		11	105	0.117021	48.80851	52.38095	26.9775
20	6.666667		8.8	105	0.091476	51.83645	55.87302	28.776
25	8.333333		7.4	104	0.076605	56.17667	59.29487	30.2475
30	10		6.1	102	0.063608	57.24713	59.80392	29.9205
35	11.66667		5.4	102	0.055901	59.62733	61.76471	30.9015
40	13.33333		4.5	100	0.04712	58.11518	60	29.43
45	15		4.2	99	0.044304	62.02532	63.63636	30.9015
50	16.66667		3.9	104	0.038961	61.03896	62.5	31.8825
55	18.33333		3.5	103	0.035176	60.97152	62.29773	31.47375
60	20		3.1	101	0.031665	60.16343	61.38614	30.411
70	23.33333		2.4	92	0.026786	59.82143	60.86957	27.468
90	30		2.2	90	0.025057	72.66515	73.33333	32.373
100	33.33333		1.9	102	0.018981	61.37196	62.0915	31.065
110	36.66667		1.5	98	0.015544	55.44041	56.12245	26.9775
120	40		1	98	0.010309	40.20619	40.81633	19.62
130	43.33333		0.4	98	0.004098	17.34973	17.68707	8.502
145	48.33333		0	98	0	0	0	0

14) DTU M6.4 2" Drive pipe 2" Stroke 20mm Weight 730g

Delivery Head hd	Head Ratio	Period Time	Delivery Flow	Supply Flow	Flow Ratio	Rankine Eff	D'Aub Eff	Power
6	2	0.965	15.1	140	0.120897	12.08967	21.57143	14.8131
10	3.333333		13.8	134	0.114809	26.78869	34.32836	22.563
15	5		11.4	129	0.096939	38.77551	44.18605	27.9585
20	6.666667		9.4	132	0.076672	43.44753	47.47475	30.738
25	8.333333		8	130	0.065574	48.08743	51.28205	32.7
30	10		6.8	131	0.05475	49.27536	51.9084	33.354
35	11.66667		6	126	0.05	53.33333	55.55556	34.335
40	13.33333		5.5	131	0.043825	54.05046	55.97964	35.97
45	15		4.7	128	0.038118	53.36577	55.07813	34.58025
50	16.66667		4.1	123	0.034483	54.02299	55.55556	33.5175
60	20		3.7	126	0.030253	57.4816	58.73016	36.297
70	23.33333		3.05	126	0.024807	55.40192	56.48148	34.90725
80	26.66667		2.5	118	0.021645	55.55556	56.49718	32.7
90	30		2.3	112	0.020066	60.80219	61.60714	33.8445
100	33.33333		1.9	112	0.017257	55.79776	56.54762	31.065
110	36.66667		1.7	117	0.014744	52.58745	53.27635	30.5745
120	40		1.2	119	0.010187	39.72835	40.33613	23.544
130	43.33333		0.8	118	0.006826	28.89647	29.37853	17.004
150	50		0	116	0	0	0	0

Appendix C **FURTHER NOTES AND COMMENTS ON THE WORK AT DELFT**

The following are a series of points arising from the work at Delft that have not been mentioned in the main text of this paper.

1) Delft managed to record delivery valve movement of rubber type valves by bonding strain guages to the rubber. This technique overcomes the difficulty of recording and analysing delivery valve movement experienced by the DTU.

2) An electronic beat frequency counter was used at Delft to record the operating frequency and period of pumps under test. This is another technique worth pursuing for ongoing tests at Warwick.

3) There is some confusion and lack of consistancy between ram pump manufacturers and users as to how efficiency is measured. Delft produce the following summary

<u>Rankine</u> Drive tank water level is taken as the datum point and supply and delivery heads measured from that point. Therefore the net amount of potential energy in the water delivered $= \rho g q(h-H)$
the net amount of energy input $= \rho g Q H$

$$\text{Hence Rankine efficiency} \quad \eta_R = \frac{q \times (h-H)}{Q \times H}$$

<u>D'Aubuisson</u> The impulse valve orifice is taken as the datum point and therefore

$$\text{work done} = \rho g q h$$

$$\text{energy supply} = \rho g\ (Q + q)H$$

$$\text{D'Aubuisson Efficiency } \eta_D = \frac{q \times h}{(Q + q) \times H}$$

<u>Manufacturers</u>

Most manufacturers take a simplified efficiency that actually produces a higher value of efficiency under any given set of conditions

$$\eta_m = \frac{q \times h}{Q \times H}$$

Delft prefer the Rankine expression which yields the lowest results particularly at low delivery heads.

The DTU currently uses the D'Aubuisson formula as the test rig measures the total flow into the pump (Q + q) rather than just the waste flow. This has been used throughout this report.

There seems little to choose between these two which produce similar results under normal operating conditions. The simplified efficiency used by many manufacturers is more inaccurate but produces more flattering figures!

4) Observation on relative position of delivery and impulse valves concerning air intake during recoil.

During recoil, large quantities of air can be drawn in through the open impulse valve as water flows back up the drive pipe. Only a small quantity of air can be drawn through the snifter valve so section A remains relatively full of water. As acceleration of water towards the pump re-occurs the majority of the air is expelled through the impulse valve and only the small volume entering from the snifter enters the air vessel.

If the recoil is large in this configuration and a significant quantity of air drawn in through the impulse valve, the column may receed past the vertical pipe section leading to the delivery valve. When acceleration occurs a significant pocket of air may be trapped under the delivery valve. This would tend to dampen out the next pressure pulse and reduce delivery flow.

5) The Alto ram from France has an inflatable rubber air compartment in the air vessel to ensure a permanent separation of air and water. This is an attractive option as it reduces the need for a reliable snifter to replenish air lost from the air vessel.

6) The SANO rams are fitted with a 'drip valve' in the air vessel that appears to be located at a point where the air/water interface should be. It presumably operates by passing water if the level in the air vessel rises above it. What is its operating mechanism?

8) Delft draw the following conclusions about pump performance and operation from results produced by their mathematical model.

- given an available source supply the pumping rate (q) is primarily determined by the supply head (H_s)and the delivery head (h_d).
- an increase of the delivery head (h_d) decreases the quantity pumped per cycle (V_d) and by that the pumping rate (q) decreases.
- an increase of the supply head (H_s) increases the pumping frequency and by that the pumping rate (q) increases.
- an increase of u_c (ie. the velocity of the water in the drive pipe at waste valve closure) normally increases the pumping rate (q) while the pumping frequency decreases, but more water (Q) is needed to operate the ram. However, there is a limiation as to this point: an increase in u_c such that the value of the ratio u_c/u_o approaches unity (where u_o is the maximum attainable velocity of the water in the drive pipe) implies a decrease in pumping rate (q) while, as before, the waste flow (Q) increases, a condition to be avoided.
- the larger the size (ie. drive pipe bore) of the ram, the more water (Q) is required to operate the ram and the more water (q) can be delivered to a higher level (h_d).

9) Delft recommend that if the delivery flow from one ramains insufficient a second ram should be placed below the first and utilise its drive water. However if there is sufficient drive head available to be able to do this would a better performance be gained by having two rams each working from a portion of the drive head or one ram working with the total head available?

10) Delft state that the drive pipe length should be approximately 4 to 7 times the supply head. They make no justification for this statement.

11) Delft state that the best pumping results are usually obtained when the cut off velocity is between 60% and 80% of the terminal velocity of the system.

12) Delft produce equations for predicting pump performance, which require certain information about the system to tbe known.

A loss coefficient for the whole system is required which covers all the losses in the drive pipe, pump and delivery pipe. Delft state that this can be found by holding open the impulse valve, measuring the flow rate at terminal velocity and inserting this into the equation

$$\text{max velocity} \quad V = \sqrt{\frac{2gH}{\text{loss coef}}}$$

This does not take into account the delivery system losses and can only be measured once the system is installed, preventing prediction of performance prior to installation. It would be more useful to have a loss coefficient measured for each pump and also to have a simple method of calculating pipe loss coefficient based on diameter and length.

The other requirement for use of Delft's model is to find the cut off velocity for the particular impulse valve setting. They state that this can be found when the maximum delivery head obtainable is known using

$$\text{Cutoff} = (g/c) \times h_{max}$$

They recommend that h_{max} is found by closing the delivery side of the pump and letting the pump run up to its maximum head. This would provide a rather crude approximation producing a low result and could prove to be very dangerous.

DEVELOPMENT TECHNOLOGY UNIT

Working Paper No. 34

The Treadle Pump

1991

Development Technology Unit
Department of Engineering University of Warwick
Coventry CV4 7AL UK
Tel: +44 (0)203 523523 extn 2339 Fax: (0)203 418922 Telex: 311904 UNIVWK G

THE TREADLEPUMP

A human-powered pump for small-scale irrigation in Developing Countries

DTU Working Paper 34

prepared for The Ramat-Warwick Linkage Programme, Sub-Programme 4
under the North East Arid Zone Development Programme
Borno State, Nigeria

The Development Technology Unit, Department of Engineering
University of Warwick, Coventry, CV4 7AL, U.K.

SUMMARY

Using a treadlepump to lift water to a height of 5 metres it is possible for a farmer to irrigate approximately 0.25 hectares of land. The pump can be used for long periods of time because of good ergonomic design. The cost of construction is low and the pump can provide an appropriate technology for smallholders with little capital to spend on irrigation.

THE TREADLEPUMP

A human-powered pump for small-scale irrigation in Developing Countries

1.0 INTRODUCTION

This report has been prepared by the Development Technology Unit of the University of Warwick, Coventry, U.K. It is part of the studies carried out under an EEC funded linkage programme between the University of Warwick and Ramat Polytechnic, Borno State, Nigeria. The linkage is a sub-programme of the North East Arid Zone Development Programme (NEAZDP) which itself seeks to promote the development of rural people in the region and promote long-term sustainability.

The subject of this report is treadlepumps and their use for small-scale irrigation in Developing Countries. The treadlepump is a human-powered water pump that can lift irrigation water from ground or surface sources. The pump consists of two pumping cylinders that are worked reciprocally by the action of the operator standing on two treadle bars. This system gives a near constant flow of water and a comfortable pumping action for the user. The design of the pump allows operators to shift their position along the treadles and so vary the force on the cylinders; in this way a range of pumping heads can be achieved.

The principle advantage of the treadlepump over other forms of human-powered water pump is that it makes best use of the muscles of the body: The treadle action uses the large muscles of the legs, buttocks, and back. This allows sustained use of the pump and the pumping of large volumes of water that are needed for irrigation.

There are currently several designs of treadlepump produced, and a section of the report details their principal features, strengths, and weaknesses. For interested parties there are contact addresses for all the pumps described.

The report goes on to describe why it was thought valuable to develop a new design to cater to the special needs of African countries. The result was the Warwick treadlepump and the original design specification is included with a discussion of the pump's construction and performance. Complete construction details of the Warwick treadlepump, engineering drawings and notes, are given in an appendix. A second appendix discusses the use of the treadlepump in the context of Borno State, Nigeria.

2.0 THE NEED FOR TREADLEPUMPS
A Comparison of Pumping Technologies for Developing Countries

As the human population increases at an ever faster rate the need for increased food production is pressing. The greatest need is in Developing Countries where populations are growing fastest and the land available for the growth of staple crops is shrinking. One of the best ways this can be achieved is by making appropriate irrigation technology available to the small farmer.

In Developing Countries much of the land is family-owned in small plots. It is not uncommon for the majority of farming land to be owned in units of less than 2 hectares. These small lots are often worked solely by the owners and their families, and external inputs, in the form of mechanisation and fertiliser, are low.

The way to increased production has often been seen through incorporating small holdings into larger units and introducing mechanisation and large-scale irrigation. This approach, while successful if carefully thought out, often produces many problems that did not occur with indigenous farming methods; salination, hard-panning, and loss of fertility. Often this type of farming proves to be less efficient than traditional methods.

Modern thinking for the development of agricultural production in Developing Countries looks to retain the traditional ownership of the land and support the indigenous farmer by providing educational and appropriate technical support.

Irrigation is one of the principle ways to increase production in agriculture either by extending the growing season on existing land or by bringing further land into production. Providing small farmers with irrigation water from large schemes can be very expensive: The cost of civil engineering works must be subsidised by the government, or another agency, as the cost is beyond the reach of the rural poor. The collection of water fees from a large number of users is also difficult. Such schemes often achieve only low efficiencies (high water wastage).

The alternative is small scale irrigation using appropriate, locally maintainable technology that is directly controlled by the farmer. As a smallholder can only afford a small capital sum and low running costs the options for technological inputs are limited. A low initial cost eliminates the use of a renewable energy such as solar or wind even though these technologies offer very low costs in the long term. The cost of petrol or diesel engines is also high and although fuel is cheap in some countries, such as Nigeria, its supply can be uncertain and spare motor parts are, at best, expensive, at worst, unobtainable. This leaves the use of animal or human labour for water pumping activities. In areas where draft animals are already used animal power is a good option. However if the animal is used solely for irrigation activities it is unlikely that the extra return gained is sufficient to justify the expense of keeping the animal. In this case joint ownership would seem to be the answer but this brings with it a further set of problems regarding the upkeep of the animal.

Human labour on the other hand can be used for irrigation when not occupied with other tasks, and although the power output of a human is low, when compared to other energy sources, the farmer can decide exactly when and how much to irrigate the fields. This level of control given to the farmer can significantly increase yields.

For these reasons small, human-powered water pumps for irrigation are very useful to the smallholder with few resources. Pumps that can be operated and maintained at a village level also have the advantage reducing the dependence of farmers on outside support services and increasing their self reliance. The treadlepump is designed with these criteria in mind and as will be shown in Section 4.0 makes best use of the capabilities of the human body in providing power for water pumping.

3.0 TREADLEPUMPS- A Description

The treadlepump is a human-powered water pump that is used principally for irrigating smallholdings of land. In all its forms water is pumped by two direct-displacement pistons which are operated reciprocally by the walking/stepping motion of the user.

A general layout of the treadlepump is shown in Fig 3.1. The range of treadlepump designs can be split into two groups: Those that will lift water from a lower level to the height of the pump; and those that will both lift water from a lower level and lift it further to a greater height than the pump. These two types can be described as a suction pump, and a suction and pressure pump, respectively.

The basic components of any treadlepump are described below:

3.1 Pump Cylinders

The two pumping cylinders are operated reciprocally and give a near continuous flow of water. Depending on the operating head, the diameter of the pump cylinders is normally between 75mm (for high heads) and 150mm (for low heads), most often 100mm diameter is chosen. A cylinder diameter of 100mm is suitable for pumping water to heads of between 3 and 8m.

Cylinder materials vary from curved steel sheet to bamboo to pvc pipe. Choice of material is strongly influenced by local availability and cost. Steel is a good choice if there are sufficient skills and machinery to work it; pvc pipe is often more readily available and easier to work. Bamboo is cheap and plentiful in some locations and has the advantage of maintainability at the farm level but suffers from having a short working life: It is not suitable for pressure pumps. The choice of material will also be influenced by the type of pump manifold and how it connects to the cylinders.

3.2 Pump Manifold

The manifold connects the inlet and outlet pipes to the cylinders (in a suction pump it is not necessary). It is separated into two; water enters the pump at the entry side of the manifold, and after passing through the cylinders, leaves on the discharge side. The manifold normally houses the non-return valves which prevent backflow in the pump.

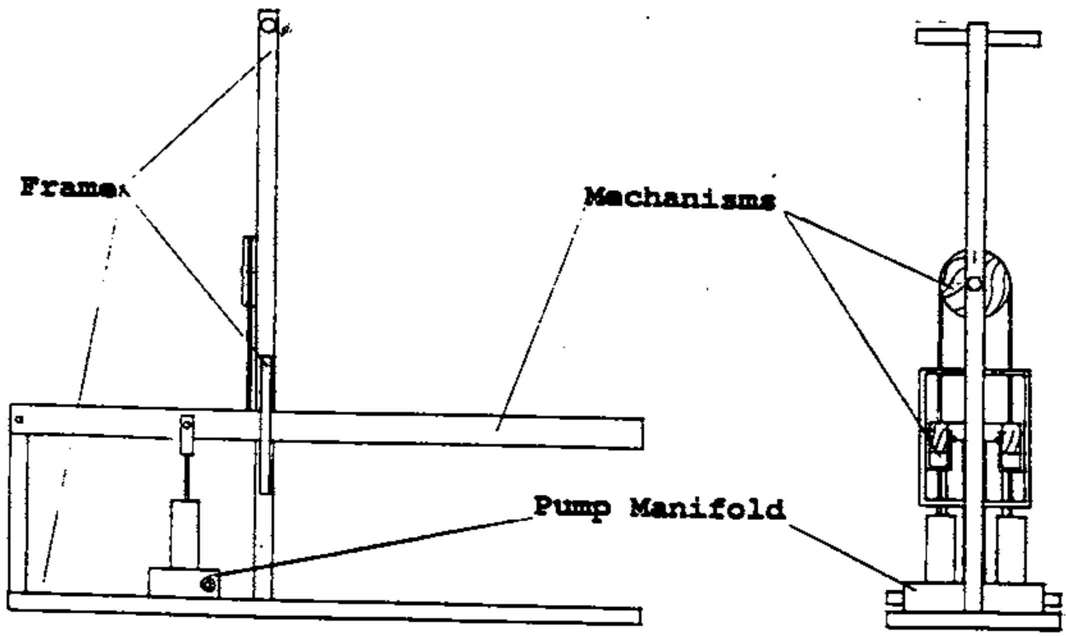
The manifold can be constructed from steel sheet welded into the form of a box, or more easily, in a pump with pvc cylinders, from pipe fittings or straight pipe. The design of the manifold must allow for the removal of the non-return valves for maintenance.

3.3 Non-Return Valves

In a suction only pump the non-return valves are situated in the piston heads and in the inlet pipe. In a suction and pressure pump there are two pairs, one on the inlet side of the pump and one on the outlet. The valves allow water to travel through the pump but prevent its backflow to the source. The valves open and close due to the positive and negative pressures developed during pumping.

Various types of non-return valve are used in treadlepump designs, these include rubber flaps, swinging gates, and poppet valves. The Warwick design uses a new type of valve that relies on the stretching of a rubber flap over a pattern of holes and is unique in that it involves no moving parts.

FIGURE 3.1 Main Components of the Treadlepump



3.4 Piston Assembly

The pistons are driven up and down within the cylinders by the weight of the operator on the treadles. The piston rod is connected to the treadles by a hinge that allows the piston to stay vertical in the cylinder as the treadles are moved up and down. This is important to maintain a good seal between the piston cups and the cylinder.

The piston rods are normally of round steel bar and are threaded at one end to attach the piston cups or rings. The piston cups can be made of rubber or leather but must provide a good water seal and sustain the rigours of continual friction against the cylinder wall.

3.5 Treadles

The pump operator stands on the treadles and pushes them up and down to work the pump. They are normally hinged at one end but in one bamboo version the hinge is between the cylinders and the operator. At the opposite end, the treadles are supported by a pulley wheel and rope or a rocking bar. The range of movement is dictated by the stroke length of the pistons and most importantly by the comfortable operating step of the operator.

The treadles can be made from wood, bamboo, or steel. Steel is only normally used when the treadles are to be cranked to bring the operator closer to the ground.

3.6 Pulley Wheel

The pulley wheel allows the cylinders to move reciprocally and so provides a natural operating style for the user. The wheel has a central axle mounted to a frame; a simple axle, such as a 16mm diameter bolt, is normally used. The wheel can be made from hard wood and boiling the wheel in oil treates the wood and lubricates the bearing for a substantial time.

An alternative to the wheel is a rocking bar which is pivoted in the middle. This arrangement is harder to balance but may be easier to construct in some instances.

3.7 Frame

The component parts of the treadlepump are mounted on a frame or in some designs buried in the ground: A frame allows the pump to be carried to where it is most needed. Although an expensive part of the pump, portability adds significantly to its value in irrigation.

Angle iron or box section are most often used to make a frame that will stand the weight of the user and the rigours of transportation.

If the pump does not need to moved regularly it may be acceptable to dispense with the frame and support the treadle hinges and the pulley wheel on stakes knocked into the ground. This will reduce the cost of the pump. In Bangladesh the Tapak-Tapak pump uses this set-up but it is reported that the pump is carried back to the house each night for security; obviously here the lack of a frame has not stopped the pump being portable!

3.8 Summary of Treadlepump Components

If the parts just described are well designed and matched, the pump will operate smoothly and efficiently. To end this section here is an attempt to describe the action of powering the treadlepump.

The motion of the operator's legs on the treadles is similar to a combination of walking on the spot and of climbing a ladder. Operators can become accustomed to this movement very quickly and in a well designed pump will find that it can be sustained for several hours a day with appropriate rests. A full description of the ergonomic design of the pump is given in Section 4.0.

4.0 THE ERGONOMICS OF TREADLEPUMPS

Using human labour to power a water pump has several advantages: Given a constant water source the farmer can choose exactly when to irrigate. If irrigation is not required the farmer can usefully work at other tasks without having expensive animals or machinery standing idle. However the human body is not very efficient at converting food stuffs into work energy and requires what is, in effect, very high-grade fuel. Also the total amount of power available from one person for manual work is not high- it is in the range 200-300 watts per day. So any machine that uses human power should be efficient to make best use of the scant power available. Another important point is the ergonomic design of the machine that enables it to be used comfortably and for sustained periods of time.

The human body will quickly tire if isolated groups of muscles are used exclusively or if the body is forced to work in an awkward position. Where only a small power input, in human terms, is required, for short periods of time, these points do not need to be considered so carefully. A good example of this is the community handpump where the work done by each individual user is small: The arms are required to do almost all the work, and often the body is bent over, but the water is soon lifted and the activity ceases.

But for irrigation the volumes of water that are necessary to irrigate even small areas of land (0.25 hectare) are very large- several thousand litres when compares to 50-100 litres for household use- and consequently the pump design is paramount. Therefore the human-powered, small-scale irrigation pump must be able to pump large quantities of water and be comfortable to use for several hours a day.

4.1 Utilising Human Power

Perhaps the best way for utilising human power is the bicycle: The pedalling motion of the legs uses the large muscles of the legs, buttocks, and back. A power output of approximately 75 Watts can be sustained for up to 4 hours a day. However the bicycle design requires the use of sprockets and chains which are costly to produce and cannot be easily repaired in rural areas of Developing Countries.

A much simpler arrangement that approximates to the action of pedalling is that used in the treadlepump. The operator stands with one foot on each treadle bar and operates the pump by moving the bars up and down in a reciprocating motion. This is a natural movement for the human body, somewhat akin to walking, and can be sustained for long periods if the parameters of stroke length, cadence, and foot force are matched to the operator.

4.2 Stroke Length

The stroke length is the vertical distance between the feet when one foot is at its uppermost position and the other at its lowest. Too short a stroke and the leg muscles will tire quickly, too high and the leg muscles are straining. On a bicycle a crank of approximately 170mm gives a comfortable and efficient action; this is equivalent to a stroke length of 340mm on a treadlepump. In the case of the stepping action of the treadlepump this is a long stroke and results in a low pumping cadence (the speed of pumping). The relationship between stroke and the cadence is inversely proportional: The stroke lengths is governed by what is a comfortable speed to operate the pump. Another point to consider is the difficulty of sustaining a high foot force when pumping to high heads if the stroke length is long.

Bearing in mind these factors a pump that offers stroke lengths in the range of 100-350mm will be comfortable to operate. The operator will normally choose a short stroke length at high heads and a longer stroke with decreasing head.

4.3 Cadence

The cadence is the frequency with which the treadles are moved up and down: One cycle is completed when a treadle bar is pushed down and rises once again to its starting position. A cadence of up to 60 cycles per a minute is comfortable.

With increasing foot force (at higher heads) the cadence will tend to drop and the stroke length increase. The opposite is true for low heads where the stroke can be very short and fast; this can be tiring. In this case moving along the treadles nearer to the hinge will reduce the mechanical advantage and so improve the pumping action of the operator.

4.4 Foot Force

The pumping head is overcome by the force on the piston acting against the water in the cylinders. The piston force must also overcome the friction losses is the pipes.

The force exerted on the pistons by the operator can be varied by the operator moving along the treadles: This varies the mechanical advantage between the operator and the cylinders. This is one of the most useful features of the treadlepump. Not only can the operator find a comfortable position for different pumping situations but also the pump is adaptable, without modification, to a wide range of people of different strengths and weights.

For extended periods of pumping the foot force should not exceed 50% of the operators body weight, the maximum for short periods of time is approximately 70%. Therefore to be suitable for men , women, and children a pump should be designed to give foot forces of 15-50kg across the range of pumping heads.

If a pump is to be used almost exclusively for a particular head application then the cylinder diameter can be chosen to obtain suitable foot forces. The most common cylinder diameter is 100mm and is suitable for heads of between 3-8 metres. Below 3m a 150mm cylinder is suitable and above 8m a 75mm can be used. However a 75mm cylinder reduces the quantities of water pumped significantly and therefore pumping irrigation water to heads of more than 8 metres using human power is not normally economic.

4.5 Mechanical Advantage

Moving up and down the treadles varies the mechanical advantage of the treadlepump. In this way the operator can vary the force exerted on the pistons while maintaining an approximately steady foot force. This is much less tiring than a pump where the force required is fixed according to the head of water.

To allow the pump to be used by the widest range of operators and across varying pumping heads it can be calculated that the mechanical advantage must vary between 0.5 to 1 and 3 to 1 for a 100mm cylinder. The low values of mechanical advantage, less than 1 to 1, can be easily obtained by the pump operator moving on top of the pistons and even past them, next to the hinges. For high mechanical advantage, of 3 to 1, the distance between the hinge and the end of the treadles must be three times the distance between the hinge and the piston. Care must be taken to ensure the pump does not overturn when the operator stands at the extreme end of the treadles.

4.6 Treadle Spacing

A comfortable spacing of the treadles is between 175-200mm. Slight variations can be achieved by the operator moving the feet to one side of the treadle bars. In practise the spacing of the cylinders will probably dictate that the treadle spacing is close to 200mm.

4.7 Summary of Ergonomic Features

A pump that provides the maximum range of adaptability while retaining a comfortable pumping action will have the following features:

Stroke Length	100-350mm
Cadence	as chosen by operator
Foot Force	15-50kg
Mechanical Advantage	from 0.5 to 1 upto 3 to 1
Treadle Spacing	175-200mm

5.0 CURRENT TREADLEPUMP DESIGNS

5.1 The Bellow Pump

The Bellow Pump design evolved at the International Rice Research Institute (IRRI) in the Philippines. However although having some characteristics in common it is not a true treadlepump and so only a brief description is given here. The pump has two sets of flap valves and is both a suction and pressure pump.

Construction

The pump utilises a pair of flexible bellows fitted together on a base plate; see Figure 5.1 for a general layout. The pumping action is a rocking one with the bellows being operated reciprocally. The material for the bellows is principally canvas which is reinforced with steel sheet with a rubber bladder inside. The bellows are stitched together to achieve the desired shape and considerable care is needed to ensure a water-tight seal. The discharge box, a kind of manifold, is made from steel sheet folded into shape. The non-return valves are a rubber flap with a hard rubber stiffener; the valve assembly bolts over a hole in the discharge box.

In general the construction requires accurate marking out of the component parts and skillful stitching to produce a durable pump. However few machine tools are needed; there is a small amount of welding. The materials used are many: steel, canvas, two types of rubber, bolts, rivets, steel and plastic pipe.

Performance

The output flow of the bellow pump is large by human-powered pumping standards- nearly 90 l/s at 1.5m head falling to 40 l/s at 3.5m head. The pump testing was carried out by pumping for 15 minute periods and so even the quoted maximum efficiency figure of 40% at 2.5m, calculated using a human power input of 75 Watts, could be over optimistic. Pumping to heads of greater than 3.5m is not suitable since the foot force the operator must apply is excessive; unlike a treadlepump there is no way to vary the mechanical advantage of the bellow pump.

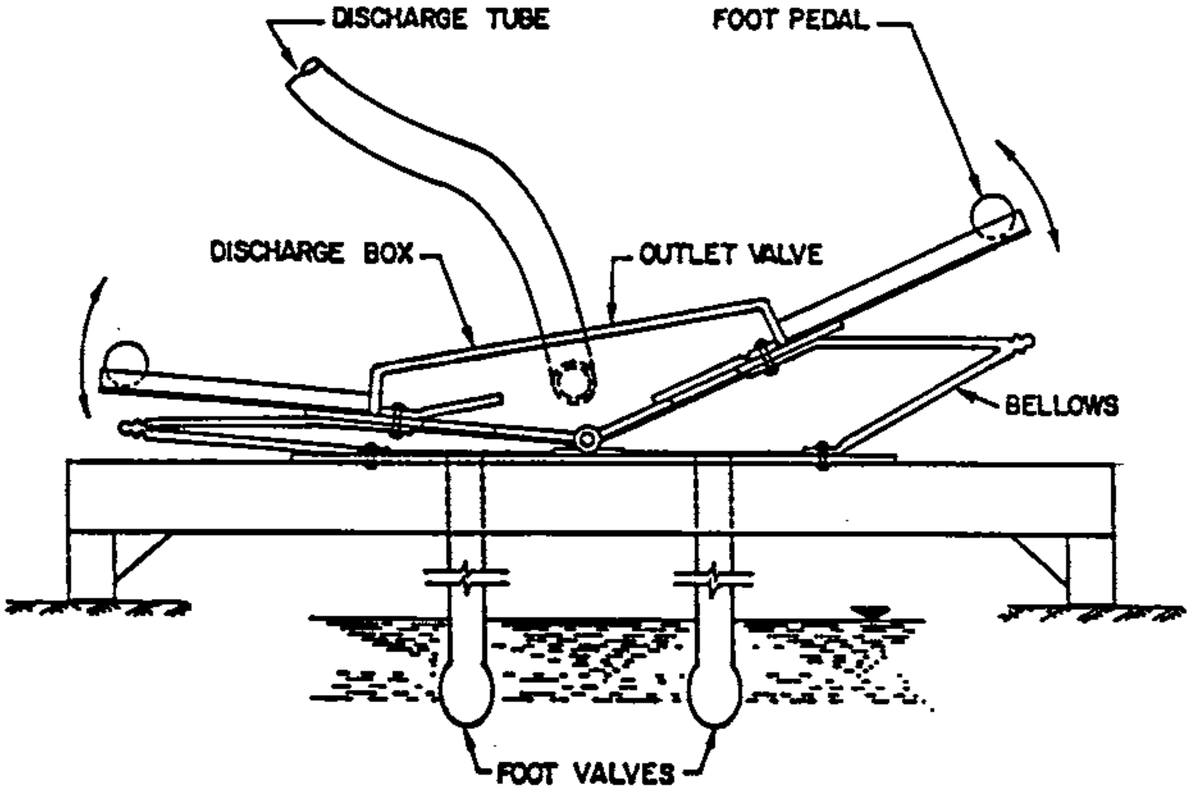
Summary

The Bellow Pump requires considerable skill in its manufacture although few tools are required. It can pump large volumes of water to small heads but is, at best, only 40% efficient. Reports of field trials call into question the durability of the pump and it appears that, now, few are still used or manufactured.

Further details from:

International Rice Research Institute
P.O. Box 933
Manila
Philippines

FIGURE 5.1 The Bellow Pump



MANUALLY OPERATED BELLOW PUMP
FOR RURAL COMMUNITY WATER SUPPLY

5.2 The Tapak-Tapak, Bangladesh

The Tapak-Tapak Treadlepump, or TT Pump, is almost certainly the most widely used of treadlepump designs. The design originates from northern Bangladesh, where there are reported to be 50,000 pumps in use, and has also been adapted by IRRI for use in the Philippines.

The pump is for suction only and so is suitable only for pumping from shallow wells (upto a maximum 7m, but more normally 5m) or from surface water. In Bangladesh this is ideal since the water table is close to the surface even in the dry season.

Construction

The original version of the TT Pump was made predominantly of bamboo which kept its cost very low- US$8 in 1986. Modifications to the pump have used sheet steel and even cast iron cylinders. A layout of the TT pump as built in the Philippines is shown in Figure 5.2

Water is sucked up to the pump manifold and through non-return valves into the cylinders. The valves are simple flaps of car tyre. The cylinder material is bamboo or a more durable material in the improved pumps. The piston heads contain bucket valves that allow water to flow through the piston on the downstroke and drain into a channel that carries the water to the field. The bucket valves use prefabricated piston cups that are made for handpumps; these are commonly available in some countries but can be very expensive in Africa.

The layout of the pump is typical of treadlepumps in general: The treadles are supported at one end by a rotating hinge and the cylinders are placed approximately 1m from the hinge to avoid excessive twisting of the piston cups in the cylinder. The operator stands at the far end of the treadles next to the pulley wheel that supports the treadles. In the Bangladeshi "Dheki" design this has been modified to eliminate the pulley wheel and supporting rope: The hinge has been placed between the operator and the cylinders. This arrangement has several ergonomic disadvantages but the cost savings appear to override these as the design is now the best seller. The frame and the pipe work for the original TT pump are very cheap since they are all made from bamboo by the farmer.

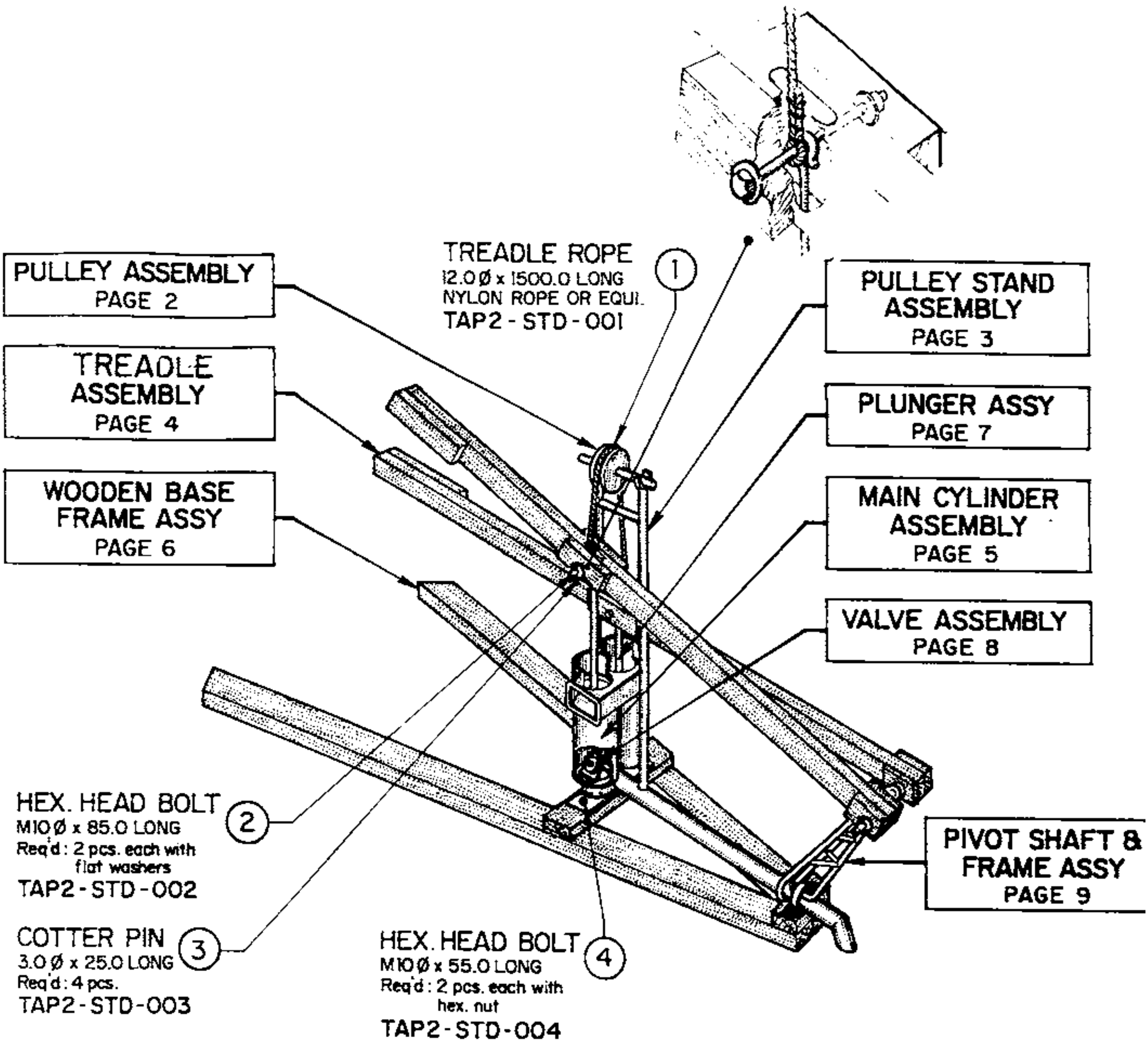
In the improved version of the pump developed in the Philippines the construction is much more complicated: Part of the frame is welded steel and the cylinder unit is of intricately folded sheet steel; the piston bucket valves need some machining; steel pipe is used for the suction pipe. This increased the cost of the pump to US$25 in 1987 but the pump is a very good product. However the materials and skills used in the construction of this pump are prohibitively expensive in some countries, many parts of Africa especially.

Performance

The figures available for the TT pump suggest that efficiency is quite high. At low rates of exertion the water power delivered by the pump is in the range of 40-50 Watts for lifts of between 3.4-5.5m. From this an estimate of the pump efficiency is approximately 50% Surprisingly the results for the modified or "Dheki" design of pump show only a slight reduction in efficiency even though the pumping action and mechanical advantage are poor. This could possibility be due to the operators familiarity with the "Dheki" as the same mechanism is used in rice pounding devices.

As the pump is suction only it is unlikely that it is suitable for use where water is at a depth of more than 5m; below this the quantity of water required for irrigation is very hard to pump using human power.

FIGURE 5.2 The Tapak-Tapak Pump



NOTE: DETAILED PARTS IN THE FOLLOWING PAGES ARE SERIALIZED BY REFERENCE NUMBERS AND ARE ENCLOSED IN HEXAGONAL FIGURE ⬡ TO INDICATE FABRICATED PARTS. THOSE ENCIRCLED ○ DENOTES PURCHASED STANDARD PARTS.

PERSPECTIVE VIEW

Summary

The Tapak-Tapak Pump is the most widely used treadlepump and is suitable for pumping from wells and surface water where the lift is less than 5m. Construction can be very cheap if bamboo is used for the principal parts but the introduction of more durable materials and commercial piston rings increases the cost. Increased levels of skill are required to make the improved versions.

Further details from:

RDRS
GPO Box 618
Ramna-Dhaka 1000
Bangladesh

and,
International Rice Research Unit
P.O.Box 933
Manila
Philippines

5.3 The USAID treadlepump, Ivory Coast

The USAID treadlepump is another design based on the Tapak-Tapak Pump (TT Pump) from Bangladesh. The design has been adapted to use commercially available pvc pipe and fittings. There are also two versions of the pump; suction only, and suction and pressure.

Construction

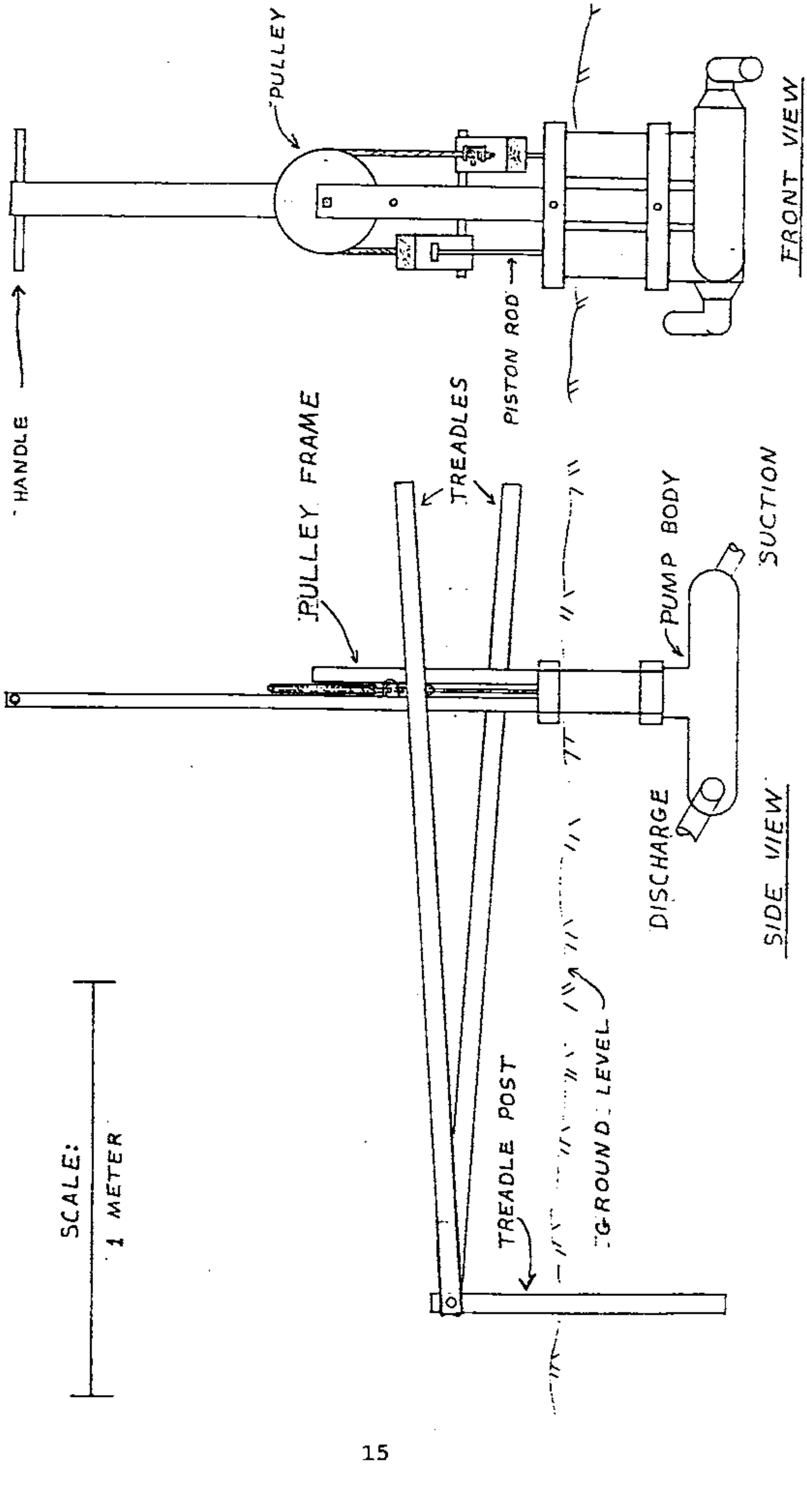
The design is shown in Figure 5.3. It has no frame and is buried in the ground on site. This is a disadvantage if the pump is to be moved frequently. The treadles and pulley wheel are similar to the TT Pump but the cylinders and the manifolds are made from pvc pipe and pipe fittings. The manifolds contain the non-return valves and so are fixed together using a non-hardening joint compound to allow for removal during maintenance. This compound might be difficult and expensive to obtain. There are two designs of non-return valve offered for the pump, a simple rubber swinging gate valve or a more complex flap valve that closes over a metal disk perforated with drill holes. Both systems require some machining to obtain a good fit with the inside of the pipe. A suction only version uses valves in the piston cup and it too requires machining.

The use of pvc fittings and pipe makes the manifold and cylinder assembly extremely easy to construct. However in many countries fittings such as the T-pieces and elbows required are both difficult and expensive to obtain. The piston cups are also commercial products although leather could be substituted.

Performance

The pumps were tested over 15-30 minute intervals and so using a human power output figure of 75 Watts maybe misleading: For short periods such as these a fit person can sustain a higher power rating. Even so the figures give by the designers of approximately 1.5 l/s at 6m head and 2 l/s at 4m head indicate that efficiency of the pump must be quite high- perhaps 75% This is not so surprising considering the quality of materials and construction in the pump.

FIGURE 5.3 The USAID Treadlepump



Summary

This is a good design of pump if pvc pipe and fittings are available and there are the necessary machines and skills to construct the non-return valves. Its lack of portability might also restrict its appropriateness. The use of many bought materials will make this an expensive pump in many parts of the world.

5.4 The Harare treadlepump, Zimbabwe

This pump has been designed with the aid of Loughborough University in the U.K. and has been called the Harare treadlepump for the purposes of this report because that is where it is manufactured (in the Loughborough University literature it is called the Pressurised Discharge Treadle Pump). It is a suction and pressure treadlepump which has a familiar layout.

Construction

Like the USAID pump pvc pipe is used for the cylinders but the manifold is made from welded sheet steel. Access to the manifold is via a cover plate that bolts to a flange plate in the manifold. Internally the manifold is divided in two to provide a separate valve chamber for each cylinder. The non-return valves are of a novel design that allows the whole valve assembly to be removed for maintenance: The valve is not attached to the manifold but is restrained from "popping out" by a stop of thick car tyre. Diagrams of the manifold and the valve design are given Figure 5.4.

The frame is of welded angle iron and allows the pump to be transported easily. The treadles are supported by a pulley wheel and rope and the frame includes an adjustable handle for the operator to hold.

Performance

Only very limited performance data is given for the Harare treadlepump; for test periods of 15 minutes the pump delivered 0.4 l/s to a head of 14m. This is a power output of 60 Watts and suggests that the operator must have been working hard. Efficiency is hard to estimate in this case.

Summary

The Harare treadlepump uses pvc cylinders and a steel manifold and as a suction and pressure pump can lift water to high heads. The construction requires a good deal of welding. The cost is high at US$275 in 1989 and this is due most probably to the amount of steel and the welding involved.

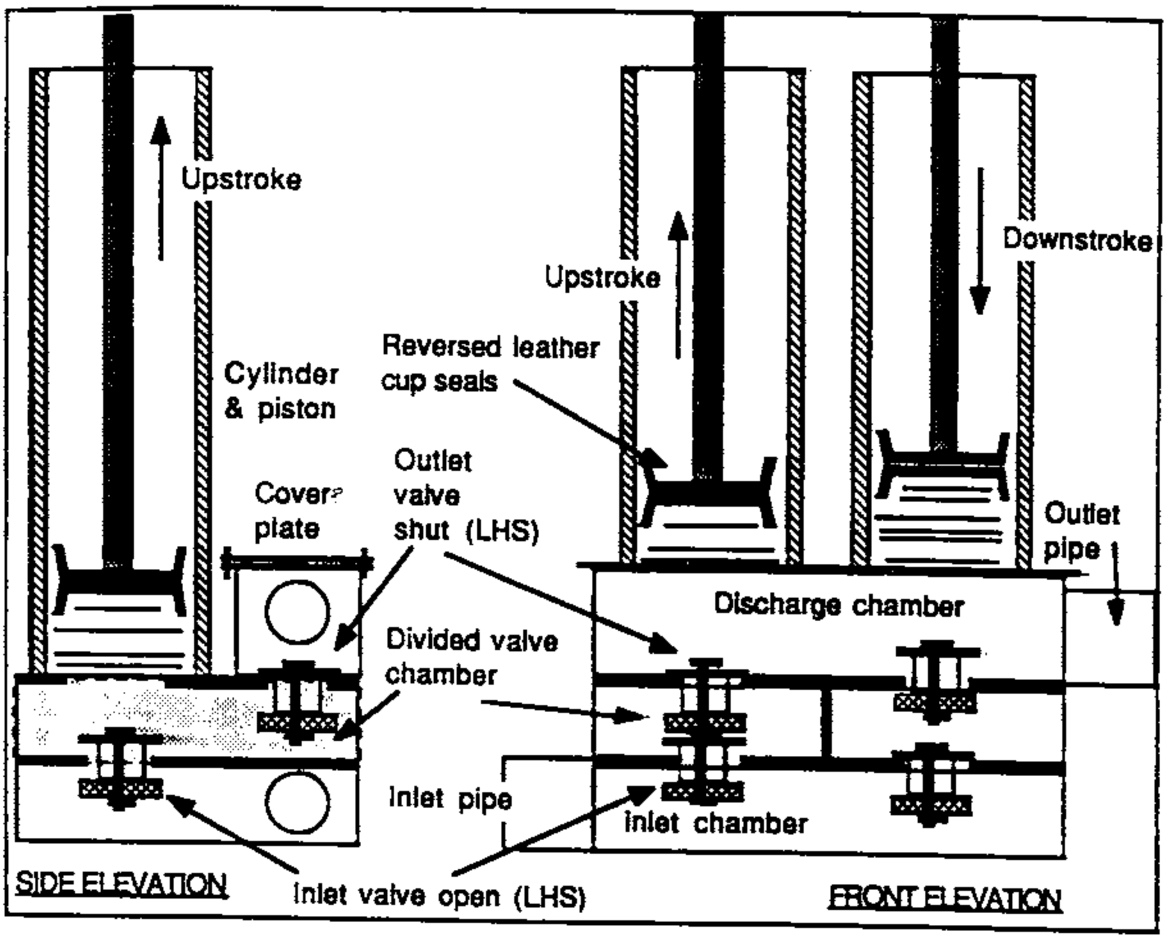


Figure 4: The pressurised discharge manifold box

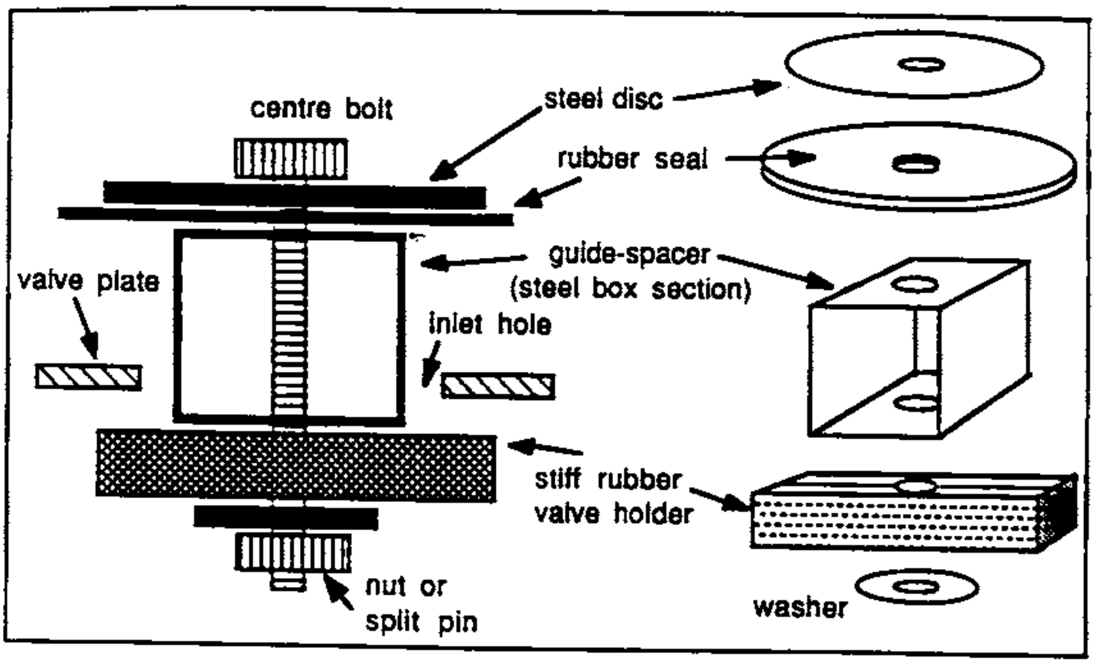


Figure 5: Valve details

5.5 The Warwick treadlepump, (Nigeria)

The Warwick Treadlepump is a suction and pressure treadlepump that has been designed to reduce the cost and complexity of other currently available designs particularly in African countries. At the time of writing, October 1991, it is about to undergo field trials in the North East Arid Zone of Borno State, Nigeria. A picture of the pump is shown in Figure 5.5.

Construction

The unique feature of the Warwick treadlepump is the use of pvc pipe to form both the cylinders and the manifold. The intersection of the cylinders to the manifold is made by cutting the pipe using a metal jig: No pipe fittings are needed in its construction. The intersection is sealed with a rubber gasket which also acts as a non-return valve by stretching away from the pipe wall during pumping. As the non-return valves have no moving parts, and are made completely from rubber, they are easy to make and cheap to replace.

The pump has a frame of angle iron and treadles of wood. The piston rods are made from round steel bar and the piston cups of leather. The parts of the pump can all be made using hand tools, although the frame needs some welding and the pipe intersections must be cut using a special jig.

During assembly much care must be taken to align the valves and intersections correctly otherwise the pump will not seal properly. Dismantling the pump for maintenance is very straightforward.

The engineering drawings and construction notes of the Warwick treadlepump are presented in Appendix A of this report.

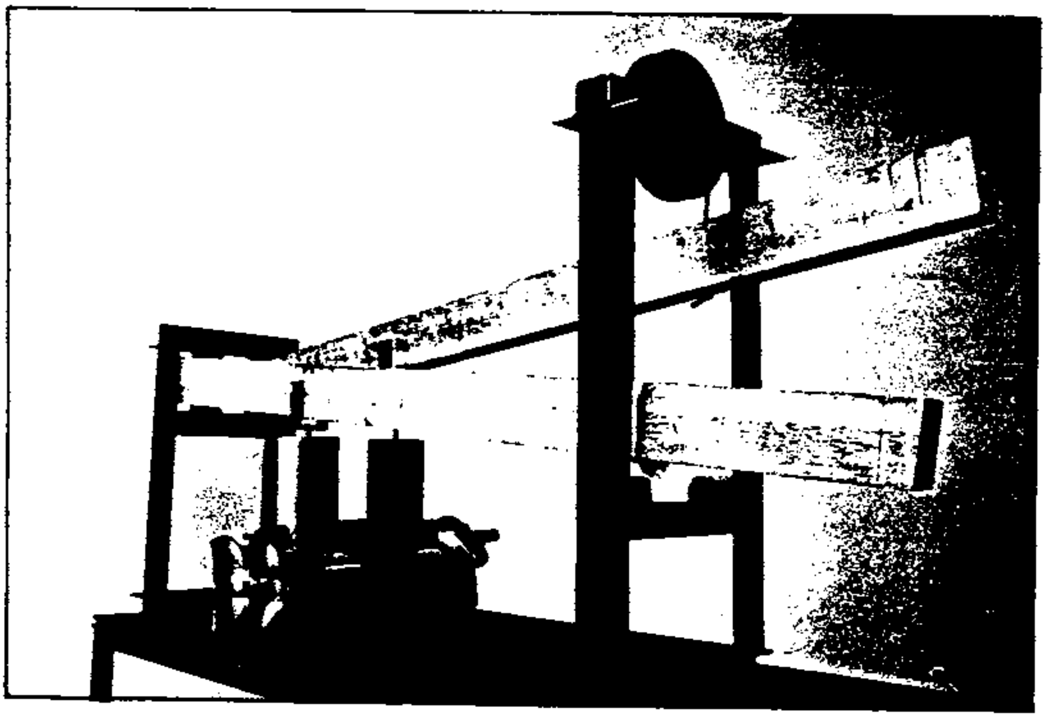
Performance

The performance of the Warwick treadlepump is given in detail in Section 6.0

Summary

The Warwick design of treadlepump uses straight pvc pipe and rubber flap valves to create a cylinder and manifold assembly that can be cheaply made. It is most suitable for lifts of between 3-8m. It is a design that is well suited to countries where steel prices are high and machine tools and metal working skills scarce.

FIGURE 5.5 The Warwick Treadlepump



6.0 THE WARWICK TREADLEPUMP

Although some of the pump designs discussed in Section 5.0 have good qualities, the Development Technology Unit (DTU) at the University of Warwick felt it was worthwhile to develop a further design that:

(a) was ergonomically effective as the best existing designs,
(b) was well suited to the needs of African smallholder irrigation in terms of flow rate, range of heads, portability, and ease of maintenance,
(c) was simpler to make than other designs and considerably cheaper,
(d) used materials fairly readily available in African countries and avoids components (such as pipe fittings) which are not.

With any technology for rural application the DTU tries to serve not just the user but the artisan interest group as well- it seeks to identify products and designs suitable for income generation via manufacture in villages and small towns. Sometimes user and manufacturer interests coincide, as they appear to in this case; sometimes the user (e.g. farmer) is best served by equipment made by large-scale urban industry. The Warwick treadlepump is intended for production in small workshops with little equipment: The materials specification was determined based on the availability of materials in Zimbabwe, Zambia, and Nigeria in 1990-91. The use of scarce pipe fittings was deliberately avoided.

For reasons of simplicity and standardisation only one size of pump has been developed using a nominal 100mm diameter piston. This size is well suited to heads of 4-10m at full "adult" power but has a poor efficiency at lower heads, and hence higher flows, for which a larger diameter pump would perform better. Since the "head" supplied by the pump has to overcome friction losses in the pipes as well as lift the water, the pump is suited for lifting water in excess of 3m. At "child" power levels the optimum range of lifts is lower and the efficiency at any given head higher. The table overleaf shows the specification to which the pump was finally designed.

Figures 6.1 to 6.4 shows performance test results. These tests were undertaken in August 1991 at relatively high powers, namely those which could be sustained by a fit and healthy young man of weight 80kg for periods of 15 minutes. From Figure 6.1 it can be seen that the input power exceeded 80 Watts, giving flows higher than the specification and, in consequence, lower efficiency than the specification (the main source of inefficiency is the friction in the valves which becomes substantial at flows greater than 1 l/s). Figure 6.2 shows the effect of increasing head: Namely a fall in flow rate and a strong rise in efficiency. The rise in efficiency, combined with a slightly better ergonomic match with the test operator, explains the increase in "water watts out" with the increase in head, shown in Figure 6.1. Water Watts is a measure of the pumping power achieved.

The penalty for operating the pump in suction is shown in Figure 6.3. For example, with a total head of 5m, having all that head in suction results in a flow, 0.73 l/s, which is only 70% of the flow, 1.06 l/s, obtained when all the pump is in pressure. Thus when lifting water from a 5m high river bank, a pump close to the water level will lift significantly more than a pump situated on the bank top. It is recommended that the pump is not normally used higher than 5m above the source of the water.

In tests the pump was found to be efficient and easy to use with a number of speeds, heads, and people: The performance specification was met. However the tests were performed on a new pump; long term testing may indicate changes in efficiency over time, especially if the piston or valve wear is severe.

The materials specification was met provided pvc components are painted or well shaded to protect them from degradation by prolonged exposure to sunlight.

FIGURE 6.1

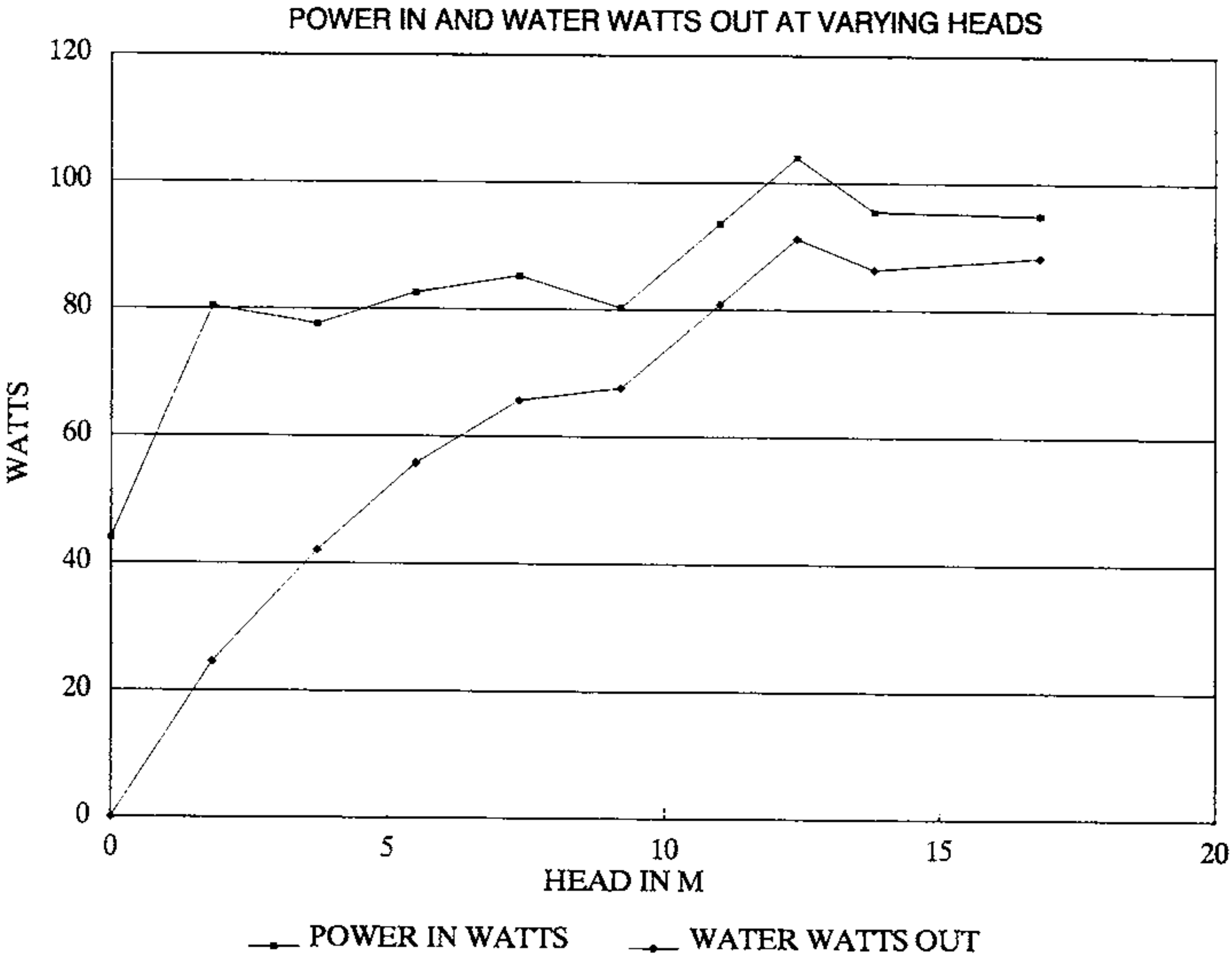
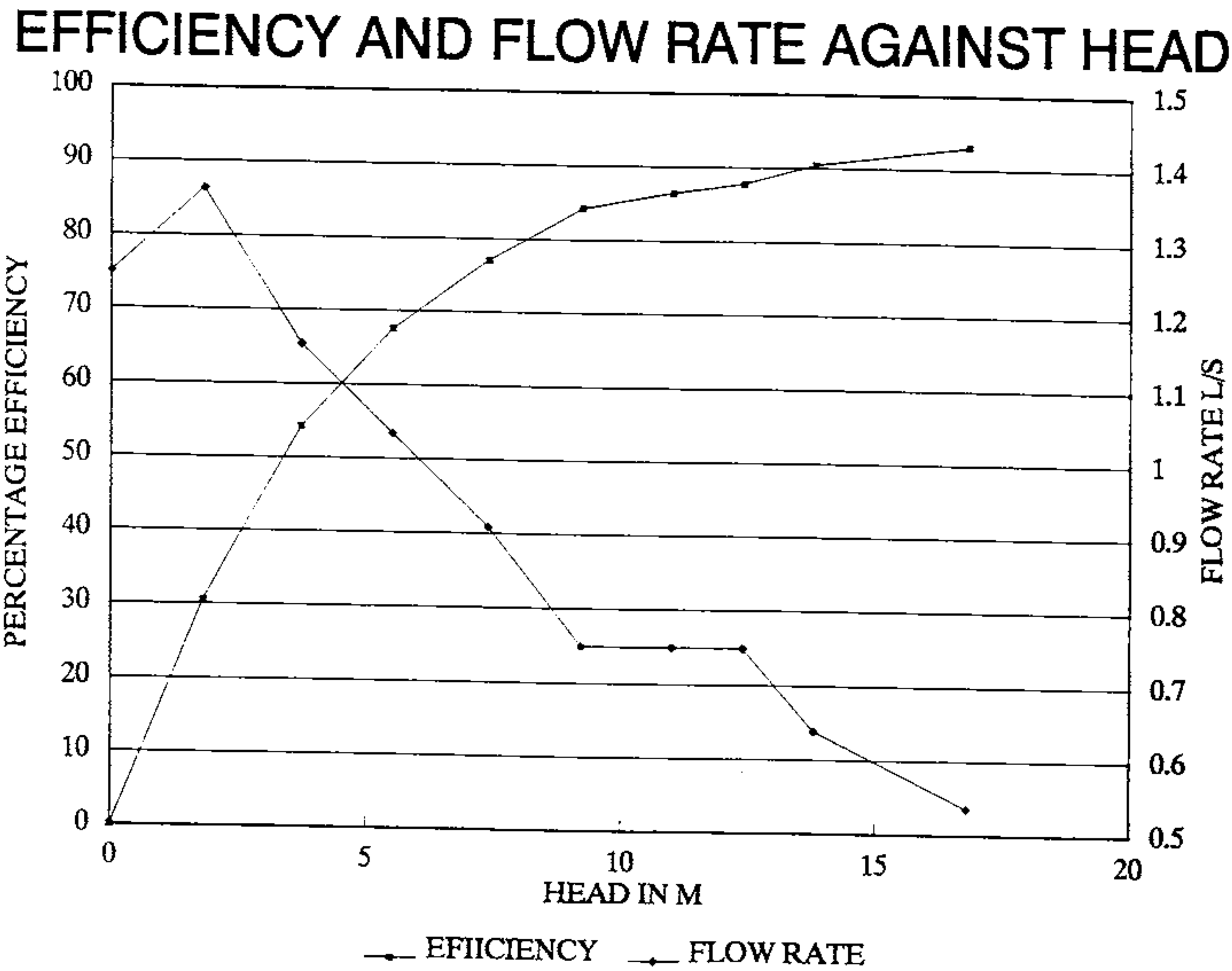
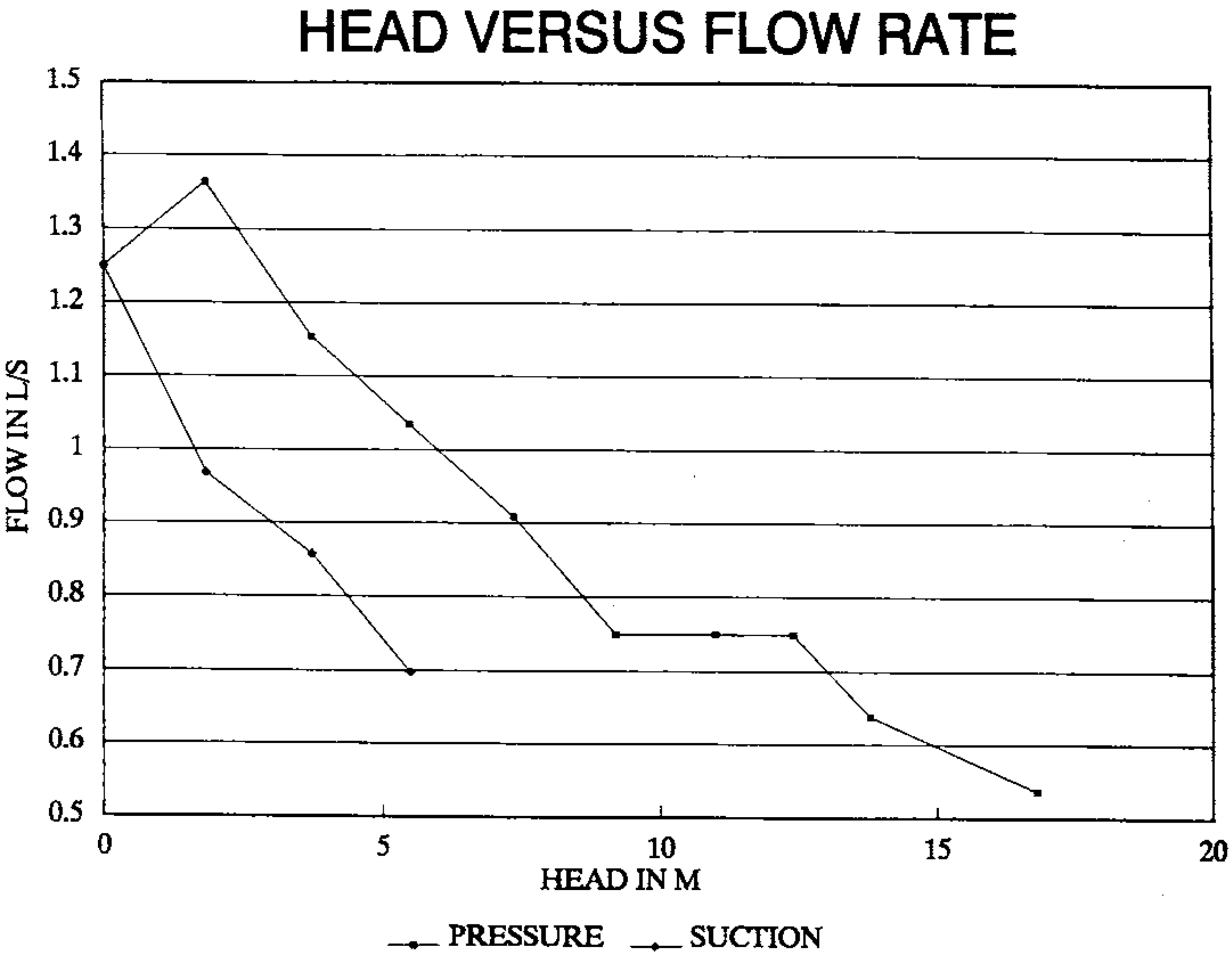

POWER IN AND WATER WATTS OUT AT VARYING HEADS
120
100
80
60
40
20
0
WATTS
0
5
10
15
20
HEAD IN M
POWER IN WATTS
WATER WATTS OUT

FIGURE 6.2



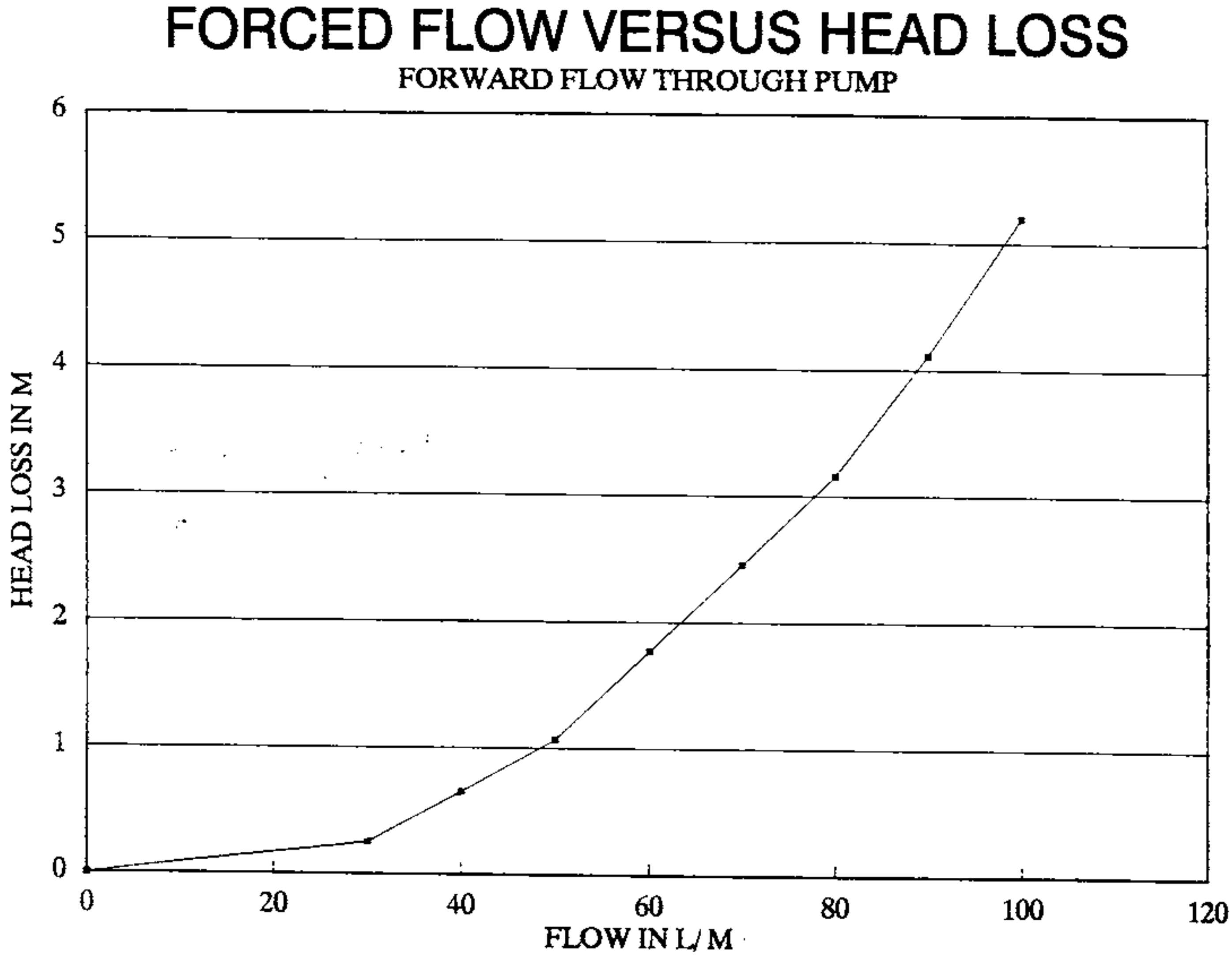
EFFICIENCY AND FLOW RATE AGAINST HEAD
PERCENTAGE EFFICIENCY
FLOW RATE L/S
HEAD IN M
0
5
10
15
20
0
10
20
30
40
50
60
70
80
90
100
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
1.5
EFIICIENCY
FLOW RATE

FIGURE 6.3



HEAD VERSUS FLOW RATE
FLOW IN L/S
1.5
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0
5
10
15
20
HEAD IN M
PRESSURE
SUCTION

FIGURE 6.4



FORCED FLOW VERSUS HEAD LOSS
FORWARD FLOW THROUGH PUMP
HEAD LOSS IN M
FLOW IN L/M
0
1
2
3
4
5
6
0
20
40
60
80
100
120

The required case of manufacture was achieved in that only hand tools and downhand welding are required. However cutting of the manifold intersections needs to be quite accurate, within 1mm, and is best is achieved using a purpose-made jig. The production of the jig itself requires skills higher than for the manufacture of the pump.

6.1 Table of Specification: The Warwick Treadlepump

Performance Criteria

a) *Operating Efficiency*
i) Normal Range 3-7m, 75% efficient
ii) Extended Range 1-3m and 7-10m, 50% efficient

b) *Head Range*
i) Adult 1-10m
ii) Child 1-7m (up to 10m if two children)

c) *Treadle Height*
The height of the treadles to be as low as possible to avoid instability

d) *Pumping Force*
i) Adult 50kgf maximum
ii) Child 15kgf maximum

e) *Mechanical Advantage*
A variation of between 0.5:1 to 3:1

f) *Flow Rate*
For an adult operator producing 40W at 3m head: 1l/s

Materials used in Manufacture

a) Only materials fairly readily available from provincial merchants have been used.
i.e. 50x50x5mm angle iron, 110mm OD pvc pressure pipe

b) Resistant to degradation by moisture, weather, and ultra-violet radiation.

Manufacture, Maintenance, and Durability

a) The purchase cost of the pump should be less than one profits from one dry-season crop (DTU Working Paper 32)

b) No machining

c) Minimum fabrication

d) Low tolerances

e) No requirement for specialised tools

f) The pump should be maintainable with a few common tools. Wearing parts should be locally made and should have a working life of at least one crop season. Non-replaceable components should have a service life of over five years.

6.2 Status of the Warwick Treadlepump

The design of the Warwick treadlepump has evolved over the course of three years, 1989-1991. The primary feature that sets it apart from other treadlepump designs is the unique manifold and cylinder joint that does away with the need for pipe fittings, such as t-pieces and elbows. The intersection of the cylinders and the manifolds also houses a stretch rubber non-return valve that has the advantage of no moving parts and simple manufacture. The rubber valve also acts as the gasket for the pipe intersections.

This valve and manifold design was perfected in a BSc dissertation by Rebecca Scott in 1989-1990 and was incorporated into a treadlepump design by Simon Lucas during 1990-1991. Following testing of the pump at the University of Warwick in the summer of 1991 design changes were made to improve the clamping arrangement that holds the pipe intersections together. A jig was also constructed to aid the manufacture of accurate pipe intersections and so reduce the possibility of the rubber gasket failing at high pumping heads.

At time of writing, October 1991, the treadlepump design is about to be constructed at Ramat Polytechnic, Borno State, Nigeria as part of the Ramat-Warwick Linkage Programme supported by the EEC and part of a large development programme in the North East Arid Zone of Borno State aimed at the development of rural areas and the increased security of rural people.

After initial testing of the pump at Ramat Polytechnic and, possibly, design changes to suit locally available materials, the treadlepump will be introduced to selected villages in the North East Arid Zone for field testing. This testing will take place with the close cooperation of village development representatives and the field staff of the development programme.

If the field trials of the treadlepump are successful training courses will be offered to local artisans in the construction of the pump. The credit facilities of the development programme will be offered to artisans wishing to start production. It is hoped that the treadlepump will provide a valuable contribution to irrigation technologies in the region and a chance for smallholders, who cannot afford more expensive pumps, to grow a dry season crop and protect main crops from drought.

BIBLIOGRAPHY

Water Pumping Devices- A Handbook for users and choosers
Peter Fraenkel
IT Publications, U.K
1986

Bicycle Science
Whitt and Wilson
The MIT Press, Cambridge, Mass., USA
1974

The Bellow Pump
anon
International Rice Research Institute, Los Baños, Laguna, Philippines

Human-Powered Pump for Low-Lift Irrigation
R.E.Stickney, C.Abrina, et al
International Rice Research Institute, P.O.Box 933,
Manila, Philippines
June 1987

USAID Universal Treadle Pump for Human Powered Irrigation
Dan Jenkins
USAID/Abidjan
December 1985

Manually Operated Irrigation Pumps
R.A.Lambert and R.D.Faulkner
WEDC and Civil Engineering Department,
Loughborough University of Technology, U.K

The Warwick Treadlepump
Simon Lucas
Final Year thesis, BSc
University of Warwick,
Coventry, U.K

APPENDIX A

The Warwick Treadlepump
Engineering Drawings and Construction Notes

CONSTRUCTION NOTES

The Frame

The frame is constructed of 50x50x5mm Angle Iron and welded at joints. The choice of angle is an economic one being the cheapest section per kg and the most widely available. A lighter frame could be made from box section if the cost is permissible.

Care should be taken in the orientation of the angle as this is critical for the fixing of some components. Holes for bolts (for bolt axles and pulley wheel) should be drilled before welding. Good welds will add considerably to the robustness of the frame.

The Treadles

The treadles should be made from well-seasoned soft wood and be straight along their length. The holes drilled in the treadles for the hinge and the piston rod hangers should be as accurate to the dimensions given so as little wood is removed from the treadles as possible. If treated wood is not available then the durability of the treadles can be improved by, preferably, soaking the wood in old engine oil, or painting on the oil.

The Pulley Wheel

The pulley wheel is to be of hard wood as locally available. After the pulley is finished it is boiled in oil for a day; this not only treats the wood but provides lubricant for the bearing. Take great care when boiling oil as there is a danger of the oil catching fire. The centre hole is 16mm and to achieve a running fit the bolt axle will need to be sanded down. Washers are used on the axle in the gap between the pulley wheel and the frame. A 50mm diameter piece of tin sheet can be used as a bearing cover for the end of the bolt; it can be held in place by a rivet drilled into the frame. This cover will stop dust entering the bearing.

The Cylinders and Manifolds

Once the cylinders and manifolds are cut to length the first task is to fit the end caps. This will help the pipes retain their shape during the cutting of the intersections.

The end caps are made from flattened pieces of plastic pipe. To flatten a piece of pipe cut it along its length and place it in a oven at 120°C for a few minutes. The time necessary will depend on the type of plastic pipe, so experiment. (if there is no oven the pipe can be softened in boiling oil but take care). Once the pipe is soft it can be clamped between two flat sheets and allowed to cool.

The other part of the end cap is a ring of plastic, again cut from a piece of pipe. Cut a ring of plastic 15mm long and remove a small piece from it so that it can fit just inside the cylinder pipe. This can now be glued in place in the position shown. A circular piece of plastic with a diameter equal to the internal diameter of the pipe is cut from the flat sheet and glued into place on top of the plastic ring inserted before. The end cap is then finished off by gluing another ring of plastic on top of the flat plate. Apply plenty of glue to ensure a complete seal.

On one of the end caps of each manifold pipe there is a smaller diameter pipe to allow the attachment of the inlet and outlet hoses. These fittings are made from 37.5mm diameter pipe that is belled out at one end. The belled end is fitted inside the end cap and prevents the fitting pulling out. The bell is formed by heating the small diameter pipe in the oven, and when soft, pushing the pipe over a wooden former or other suitable forming tool. The smaller diameter pipe must be glued into place in the end cap <u>before</u> the cap is fitted in the manifold.

The end caps are now fitted and the pipe intersections in the cylinders and the manifold must be cut. They are cut using a metal template and, to achieve an accurate finish, a metal jig. First place the pipe to be cut in the jig and mark a line along its length that will pass through the centre of the two intersections. Now place the template on the pipe and mark round where the two intersections will be cut (the template is 5mm undersize to allow for errors in cutting). Cut out the two intersections with a jig-saw or similar and place the pipe back in the jig. The intersections can be accurately finished using the pipe-shaped sanding tool.

Once all the intersections are cut the valves can be made. The valves are made from car innertube and marked using the valve template. It is important that the valves are cut so that the rubber that is used curves in the same direction when fitted in the pump as it did in the innertube. This is achieved by lining up the valves so that their short axis is along the length of the innertube.

Piston Cups

The cups are made from leather that is approximately 5-10mm thick. It is necessary to form the cups into shape using a steel mould. The leather is soaked for a day and an oversize piece put in the base of the mould. A fly press or similar is used to force to top of the mould down into the base. Leave the mould overnight and then cut off the excess leather before removing it from the mould. The mould can then be separated and the cup removed.

The cups are fitted to the piston rod by a nut and washer each side. The washers can be made from flat plastic sheet, as the end caps, to save materials.

Assembling the Pump

The first job when assembling the pump is to put together the cylinders and the manifolds. A lot of care is needed at this stage to line up the valves very accurately with the intersection holes. If the valves are not put in straight then the pump will blow a seal very quickly and must be dismantled once more.

Lay the manifold pipe that has two holes on a desk and place two valves in the holes. Align the valves very carefully in both directions (this is made much easier if the centre lines of the intersection holes are marked on the pipe) and then press the two cylinder pipes into place. Again align very carefully the drilled holes in the cylinder pipes with the intersection holes. You can check if the alignment is approximately right by looking into the cylinders and seeing if all the drilled holes are covered by the rubber valve. Also check that the rubber valve sticks out a little way from the intersection all the way round. If anything is looks wrong take it apart and try again- it is easier to put it right at this stage!

The next stage is to lay the other two valves in the intersection holes of the two cylinders, again taking a lot of care, and push the other manifold pipe into place. Once again the drilled holes must be perfectly central to the holes in the manifold. Now the cylinders and manifolds are together it is a good idea to hold them in place with a thick rubber band at each end.

The cylinders and manifold can now be placed in the frame and the clamping pieces fitted. Tighten the bolts evenly and be careful not to crush the pipe. Fit the pistons in the cylinders and the treadles and then attach the treadles to the frame at the hinge. Put the pulley rope over the pulley wheel to support the treadles at the other end. Connect the inlet and outlet hoses with jubilee clips- don't get them the wrong way round! The pump is now ready for use.

At certain heads it may be necessary to prime the pump by pouring water into the top of the cylinders. Also the pistons will not seal properly until the cups are thoroughly soaked. After a few strokes the pump should fill with water and the operator can find a comfortable position on the treadles.

APPENDIX B

Treadlepumps for North Borno State, Nigeria

The North East Arid Zone Development Programme (NEAZDP) concerns the northern part of Borno State in Nigeria. The area straddles the Sudan Savannah and Sahel climatic zones having both open savannah and stabilised sand dunes. It is crossed by rivers whose winter flow is slight but whose flow during the summer rains creates extensive swamp lands and permits basin (fadama) irrigation. The basic crops away from "fadama" lands are sorghum, millet, and dates. The nomadic Fulani fraction of the population move large herds of oxen, goats, and camels through the area. The population of the region is in excess of one million.

Water lifting in the Zone can be divided into: (a) The raising of water from village wells for domestic needs and livestock; (b) Lifting irrigation water from streams, swamps and (in the oases) shallow wells; (c) Pumping into urban water supply systems.

Treadlepumps are non-motorised and therefore unsuitable for pumping tasks requiring powers above 250 Watt-hours per day or 100 Watts over short periods. Moreover petrol and diesel fuels are extremely cheap in Nigeria and although engine-driven pumps are not cheap, they are readily available. For these two reasons treadlepumps are unsuitable for the last application listed above or for any large scale irrigation.

For normal village wells, treadlepumps have three disadvantages over handpumps or rope-and-bucket. Firstly the wells are normally deeper than the 6 metres any surface-mounted pump can efficiently draw up from: treadlepumps are only able to operate at ground level. Secondly treadlepumps do not have the proven robustness that one would look for in selecting any pump that is to be used by many different people. Thirdly the physical method of working a treadlepump- somewhat akin to cycling- may be culturally unacceptable for women in North Borno in places as public as a village well. Of these three disadvantages, the second and third may be perhaps overcome or accepted, given the much greater efficiency of treadle pumping over arm-powered pumping. The first disadvantage is more fundamental and rules out use of treadles at most village wells in the Arid Zone since they are over 6 metres deep.

It is primarily for low-lift irrigation of small plots that treadlepumps may find application in the Zone. For this activity both adequate output and low capital cost are required.

Treadlepumps can deliver (for a given human effort) nearly twice as much water as handpumps and three times as much as rope-and-bucket methods; except at lifts of less than 2 metres. Treadlepumps are likely to be more efficient than all other human-powered water-lifting devices. Even the smallest engine-driven pumpset has an output greater than 10 treadlepumps; however for small scale irrigation this high capacity of motorised pumps cannot be made use of. Worked for 4 hours a day and lifting water 3 metres, for example, a treadlepump will irrigate about 0.4 hectares and a motorised pump at least 5 hectares. The latter figure is much larger than one family could intensively cultivate using manual methods, so motorised irrigation implies either mechanical agriculture or multi-family cooperation.

Treadlepumps have a <u>running</u> cost that is lower than that of handpumps or buckets but higher (in Nigeria, because of low fuel costs) than that of motorised pumps. In <u>capital</u> cost, the treadlepump is dearer than the handpump and (for small scale irrigation) much cheaper than the motorised pump. For peasant scale irrigation, the form as well as the size of the costs is often important: Family labour costs may be more easily borne than external money costs. There are other important considerations in pump selection besides running and capital costs- availability in the market, maintainability and portability are examples.

Treadlepumps have two potential niches in irrigated agriculture in the Arid Zone. They might replace existing, less efficient, manual devices. They might open to irrigation new lands for which the capital costs and complexity of motorised irrigation are unacceptably high. As no treadlepumps are at present in use in North Borno or apparently on sale in Nigeria, it is timely to access which, if any, of the designs available world-wide it might be worth introducing. Introduction is taken to mean selection, demonstration, acceptance by users, and establishment of local manufacture.

DEVELOPMENT
TECHNOLOGY
UNIT

Working Paper No. 35

Materials for Low-Cost Building in North East Nigeria

1991

Development Technology Unit
Department of Engineering University of Warwick
Coventry CV4 7AL UK
Tel: +44 (0)203 523523 extn 2339 Fax: (0)203 418922 Telex: 311904 UNIVWK G

Building Materials Survey

W. Lawson, BSc, PhD

Introduction:

The survey was undertaken in two parts. In the first, visits were made to a variety of educational and government institutions and building material manufacturers in Northern Nigeria in order to assess present practices, availability and cost of materials, and their advantages and disadvantages. In the second, a visit was made to the North East Arid Zone and several villages and other sites were visited to assess present building practices.

Samples of some typical masonry building materials were taken and their compressive strengths evaluated at the Department of Building and Civil Engineering at RAMAT. The results are contained in Table 1, and show that there is considerable variation in quality and that some, concrete blocks and the stabilised soil cement blocks used by the Borno State Housing Corporation, are of a consistently low quality.

Suggestions for improvements in building and building materials were made particularly with respect to RAMAT and RAMCAT involvement with NEAZDP.

Prompt action on there recommendations would ensure that they could be incorporated in planning for 1992.

Relevant references for additional information have been included where appropriate.

Table 1 Compressive strength of some Northern Nigerian masonry building materials

	Clamp fired bricks [MPa]	Kiln fired bricks [MPa]	Concrete blocks [MPa]	Stabilised earth blocks [MPa]
	2.42	3.70	2.62	0.62
	3.11	5.46	2.86	0.99
	4.18	6.04	3.10	1.23
	4.69	6.24	3.34	2.09
	4.79	6.63	3.58	2.46
mean	3.84	5.61	3.10	1.48
standard deviation	1.04	1.15	0.38	0.77

NB. No aspect ratio correction has been made.

Earth Blocks

Outside the largest cities earth blocks are the traditional and most common building material.

The earth blocks are made from a blending of the soil profile (relying on feel and experience to assess its suitability), mixing it with sufficient water to make a soft mix, and forming the blocks in simple wooden moulds.

The blocks are sun dried and turned regularly to minimise cracking or warping through uneven drying.

At Kaska and Yin in the North East Arid Zone a considerable quantity of fine dry grass is also added to the mix.

The mortar and render which are more durable than the blocks, are made from the same soil with cow dung added. Split Azara palm was used for reinforcing. Roofing, either flat or domed was formed by short lengths approximately 1m long, of split Azara palm, covered with Azara palm woven mat and further covered with a layer of earth of the same composition as the walls. Internal walls were protected with a white clay whitewash with no additives. Durability of the earth walls and roofs was the major problem identified. Some roofs had collapsed.

Sadly, the Emir's palace at Yorgoram which was in excellent restored condition just six months ago was now in a serious state of disrepair, with walls and roofs collapsed, including a portion of the entrance gate. A significant factor in this deterioration apart from the poor durability of the soil, would appear to be termite attack of lintels, arch supports, reinforcing and other timber parts of the structure.

On some sections of the Emir's palace coatings of bitumen (roof of toilet) had been used to significantly improve durability. Another technique was to take the shell of the fruit (Kuba) of the Dorawa tree, soak it in water and use this water mixed with earth to make a hard, black protective coating.

At Yin, termites also appeared to be a significant factor in the deterioration of earth buildings. Bitumen disolved in kerosene and added to the earth render was reported to greatly improve its durability. The bitumen costs about N45 per 50kg bag and with the kerosene would be comparable in cost to cement stabilised render. Bitumen was reported to not always be easy to obtain in the market.

Used unstabilised and unprotected from rain earth blocks need regular attention to maintain their structural integrity. Renders are often employed to protect the earth blocks. These may be of a similar composition to the block itself, a gypsum(?) plaster or a sand/cement render. Studies conducted at ABU by Solanke (1984) have shown that the sand/cement render is the most durable but has poor adhesion to the earth blocks. Mud and gypsum plaster/renders adhere better but are less durable. It would appear that few attempts have been made to make stabilised blocks thus largely negating the need for a render. Similarly, no studies have been done to assess the cost difference between cement stabilised blocks and un-stabilised blocks with a cement render.

Earth blocks may be improved by attention to the mix composition and the mixing process, compression, stabilisation and curing, by rendering, and by attention to the design and construction of the building in which they are used (eg. overhanging eaves or verandah, damp course, positioning of windows and doors to minimise structural stresses). Siting of the building and choice of roofing materials may also imporve its comfort and usefulness. Secure attachment of roofing to earth buildings seems to be poorly understood. A brief description of various suitable techniques is given by Middleton (1987;3).

Palm Building Materials

Some buildings utilised Goruba palm in Y shaped post and beam construction with Azara palm woven panels for walls and roofs. Sometimes a thinner bent frame was used. The woven Azara palm lasts 2-3 years before it has to be replaced. The Goruba palm is very durable and is termite resistant. It is recycled from old buildings when they fall into disuse. Small quantities of galvanised iron were used, usually for doors and for open spouts to conduct water from earth roofs.

Recommendations

Essentially there are two main problems to address: the poor durability of the earth material and the termite attack of timber in the buildings.

1. Earth Durability

The durability of the earth material itself can be improved in various ways; by attention to the making of the blocks, by the development of effective protective coverings, by design changes to the buildings or by a combination of these.

Making blocks.

Some improvement in the durability of the earth blocks is likely with more thorough dry mixing of the materials, i.e the earth should be dried and crushed to a powder, mixed, then water added slowly as it is returned to a mouldable condition. Whether the extra work involved is justified by the improvement in durability achieved should be determined by some simple tests conducted in the villages and under RAMCAT/NEAZDP supervision.

The binding quality of the clay may also be improved by ensuring the water used is acidic. This can be simply achieved by testing with pH indicator strips and adding citric fruit juice. RAMCAT/NEAZDP should again supervise.

Further and much more significant improvements in the durability of the blocks may be achieved by stabilisation of the mix with small quantities (approximately 5% - 10%) of cement or bitumen and by forming the blocks in a simple hand operated compression machine. Research has shown that compression is most important in improving water resistance of earth blocks (Ola, S.A. and Mbata, A; 1990).

However, an initial inspection of the earth block making machine (made by RAMAT and based on an existing design) which was used by the Borno State Housing Corporation in their recent Maiduguri hosuing project, indicates some serious deficiencies. It appears that the machine applies very little compression to the blocks, particularly as it becomes clogged in use.

This was supported by the general "loose", uncompacted appearance of the blocks and also by their very poor compressive strength (mean 1.48 MPa), despite containing 10% by volume of cement. They are not likely to be very durable although fortunately most of the buildings are rendered. It is of great concern that in the first really significant application of compressed, stabilised, earth block building that such poor results have been obtained. There is an urgent need for RAMCAT to be further involved in this project in order to monitor the performance of the buildings, to review the block making and construction process (very quick but poor quality) and to evaluate other block making machines, of which various types are available. It is likely that the most appropriate block making machine will be of a simple and uncomplicated design, robust construction and able to be used effectively by unskilled labour.

Some other points to examine include:

. the adequacy of the mixing procedure
. the use of the coarse laterite material which was transported a considerable distance
. variations in cement content
. Cost comparisons of stabilised blocks with unstabilised, rendered blocks and other building materials/construction methods
. the residents' response to the houses and housing development in general. (RAMCAT and RAMAT Dept of Architecture and Town Planing) This is known as post occupancy evaluation and provides useful feed-back for future improvements.
. pressed blocks made with a cinva ram type device could be tried with interested block makers to assess the response to this technology. (RAMCAT/NEAZDP)
. a joint project with ABU could examine the problems associated with render durability and adherence.

The durability of blocks can be assessed using a water erosion test rig for which Warwick University has a design. Durability of blocks should also be assessed but the project should not be delayed while the water erosion test rig is built. The previously mentioned evaluation of block making machines and construction processes is more urgent at this stage.

Protective Coverings.

Earth walls can also be made durable with a protective render or other protective covering. Various possibilities exist which should be evaluated by RAMCAT in association with NEAZDP and preferably with builders rather than in the laboratory.

i) cement stabilised render
ii) bitumen stabilised render (also more likely to deter termites)

iii) render made from termite mound soil

iv) protective coating of vegetable oil or vegetable oil/clay mix. Palm oil and ground nut oil should be evaluated. Linseed oil has been found excellent in Australian conditions but does not appear to be available. Other possibilities for earth wall protective finishes have been tried in Ghana (Schreckenbach, H, 1991;21).

Design Changes.

Protection of earth walls can also be achieved by simple design changes in buildings;

i) use arches for doorways and windows as this reduces the tendency for cracking and hence water ingress. It also conserves the wood that would have been used in lintels and reduces the possibility of termite damage. Arches require wooden form-work for their construction but this can be reused. A technique using car tyres as arch form work has been used elsewhere (Stulz, R and Mukargi, K. 1988) and should be tried with Nigerian earth builders.

ii) Fix panels made from Azara palm woven mat to the walls to keep them dry.

iii) Extend the roof to have approximately 1m eaves or surround the building with roofed verandahs.

iv) Make the roof structure independent of the earth walls using termite resistant Goruba palm posts and long Azara palm roof timbers and woven mats. An earth coating could be optional. (see Figure 1)

v) Give roof structures sufficient slope (pitch - greater than 10°) to ensure water run off. (see Figure 2)

vi) Site buildings on raised ground and place drainage channels to keep surface water away from the walls. (see Figure 3)

vii) A radical suggestion is to build underground but thorough assessment of the suitability and stability of the soil, particularly in the wet season, is essential before this is done.

RAMCAT/RAMAT/NEAZDP should initiate and conduct courses on site in villages building model buildings to illustrate some of the above points. In particular basic design and construction techniques appear to be the least costly for builders to implement yet should result in significant improvements.

2. Termite Resistance

Termites are attracted to wood or biomass materials which contain cellulose which is their food. Some woods contain oils and other chemicals which repel termites making them termite resistant. Termites live underground, from where they gain entry to wood in contact with the ground. They perfer dark moist conditions. It follows that if timber parts of buildings are not in contact with the ground, and the amount of timber or other biomass they contain is small, then the chance of termite damage is greatly reduced. Use of termite resistant timbers also reduces the possibility of termite damage. Periodic inspection of exposed timber sections of buildings is also a good preventative measure as any termite conduits found can be destroyed before damage becomes serious. Reducing the amount of non-termite resistant timber in buildings and grass (biomass) in earth blocks will reduce the attractiveness of the building to termites. The use of termite resistant timber where ever possible, and particularly in hidden applications, is highly desirable.

The use of chemical barriers (soil impregration with chlordane or similar chemicals) must not be used because the water supply will become contaminated. Chemical impregration of timber or coating with waste oil, creosote or similar materials is likely to be costly and not necessarily very effective.

The introduction of some of the other recommendations, such as stabilisation and design changes, should enable durable buildings much less susceptible to termite damage to be built. RAMCAT and the Department of Agriculture at RAMAT will soon be commencing a project to improve grain storage structures. This would provide an ideal opportunity to trail the above recommendations in a practical and small scale manner.

The association with NEAZDP should provide access to well organised field offices and villages, for constructing the trial structures.

Fired Clay Bricks

Fired clay building products are made by techniques ranging from extremely simple, through intermediate to highly sophisticated. At the extremely simple end of the continuum clay and sand which occur naturally together in river banks are mixed with water and kneaded by foot to a soft plastic state. The bricks are formed in simple wooden moulds, sun dried and fired in a wood fired clamp kiln. The clamp kiln is very inefficient and costly to operate and the bricks produced are of a very variable and low quality. In particular, they appear to be neither very strong nor durable. They sell for N0.30 per brick. These brickmakers are disheartened by the lack of follow-up from earlier studies of brickmaking and lack capital to improve their production facilities.

The British built Government brickworks at Maiduguri is over 50-years old and represents an intermediate level of brickmaking technology. Similar materials and preparation as described above are used by these brickmakers. The bricks are made on a moulding table in simple wooden moulds and dried slowly under cover before firing in a downdraught wood fuelled kiln. This kiln is well maintained (internal walls are replastered with clay/sand mix after every 3 firings) and relatively efficient. Each firing contains 25,000 bricks of which 3,000 are rejects (ie 12%). Some of these are salvageable as half bricks.

The bricks sell for N0.20 per brick and are of a fairly consistent quality. About 30 people are employed and live on site with their families. Brickmaking appears to have become a family tradition and they are proud of their work and achievements.

During the early 1980s a continuous clay product extrusion plant was built in Maiduguri by a German Company. This plant only operated for about five years. The major reasons for the failure would appear to be inadequate assessment of the raw materials, poor quality control of the materials in the factory and lack of adequate technical training of personnel to operate the plant, including the gas/oil fired tunnel kiln, when the German management left.

About the same time a similar plant costing N5m was set up nearby by the Federal government. The plant utilised two large oil fired Hoffman kilns. This operation also failed reportedly due to a lack of marketing management expertise; the company being unable to adjust successfully to fluctuations and competition in the market.

The critical need for precise technical control in continuous production operations makes such systems inappropriate where the requisite skills are lacking or not developed. They are also inflexible manufacturing systems.

Other large scale brickmakers were observed; some closed and others operating. The viability of such operations depends on many factors which can vary, sometimes quite quickly (eg. markets) and so require skilled and flexible management as well as technology. Capable, well trained management able to plan and adapt to changing conditions is an important aspect of the success of such plants.

Recommendations

Further improvements to intermediate technology brickmaking plants may include:

* improved mixing and moulding with a drier mix and simple hand press.
* Use of simple kiln monitoring techniques such as pyrometric cones to identify cool/hot spots in kiln and to reduce these to give a more consistent product.
* Sorting and pricing of bricks according to quality and intended use (likely to be difficult while bricks are in short supply).
* Setting up of similar intermediate technology brickmaking in other areas.

* Reduction of fuelwood use by incorporation of waste oil burners of the flat plate drip feed type.

RAMCAT should make contact with the Maiduguri brickmakers to discuss the above points and plan action where necessary. Specific technical backing may be provided by Warwick University and other useful information is provided by Beamish, A and Donovan, W (1989).

RAMCAT could also conduct basic bricklaying courses and investigate the need for multi-skilled builders courses.

Concrete Blocks

Small and medium scale concrete block makers are common throughout Northern Nigeria.

A typical small scale concrete block maker employs three people and produces about 320 blocks per day from seven bags of cement and 35 barrows of sand. Cement costs 65-75 Naira per 50kg bag, sand about N200 for $10m^3$. The price of cement is rapidly escalating at present, from N65 to N90 in less than a month.

The mix used is usually quite lean, 1:10 by volume, and the blocks are often dried in the sun and sold after 3 or 4 days. The concrete is mixed by hand but a mechanical press is used to form the blocks. Waste oil or similar material is used as a release agent. Water may often be contaminated with organic matter. The blocks sell for around N4.30 each.

A larger plant with a multiple mould machine was seen at Maiduguri. This plant was designed to produce three times the present production but as machinery became in-operable it was scavenged for spare parts. Minor maintenance and repair did not apppear to be of concern. (A simple repair to a mould, which would have been easily within local capacity, was not attended to even though it produced a defective block). It appeared that because defective blocks could still be sold at the full price there was no perceived need to repair the production equipment. At this larger plant mixing was mechanical and the mix was transported to the block making machine by a mobile skip. Blocks were also wet down each day. A sample of blocks from where the blocks were wet down four times a day for three days were tested. Their compressive strength was poor, averaging 3.10MPa. (See Spence, R.J.S. and Cook, DJ 1983;89). It should also be noted that there blocks had undergone the best curing regime seen; others would be much worse.

The effects of cultural differences in perceiving and understanding production problems with concrete blocks has been discussed by Harder (1991) who suggests that special training is probably necessary.

Recommendations

Improvement to the quality of concrete blocks made could be achieved through:

* proper mix ratios/water content
* proper moist curing
* maintenance of machinery and equipment.

RAMAT could organise short courses held in afternoons, on simple concrete block making technology, including maintenance of equipment. RAMAT could sponsor a block making industry group to discuss common problems in the industry with the aim of improving standards of production.

Roofing Materials

Traditional thatch materials are used in some areas but have largely been replaced by galvanised iron. Light gauge galvanised iron costs approximately N520 for 20 standard 8ft x 4ft sheets. Corrugated fibre cement sheeting is slightly more expensive, costing almost N27 per m^2, for the cheapest sheet.

Galvanised iron is easy to transport and fix, but has a limited life. Fibre cement sheet requires some special skills to fix, but is more durable than galvanised iron. Fibre cement sheet also keeps the building cooler than galvanised iron.

The sole Nigerian owned fibre cement sheet manufacturer, Turners Building Products Limited at Kaduna, is currently trying to eliminate the use of asbestos in its products.

Wooden roofs with earth cover are also used occasionally but require significant maintenance.

Recommendations

* RAMCAT to conduct short practical courses in rural area, on fixing fibre cement sheets.
* RAMCAT to develop and run courses in rural areas on the fixing of roofs to earth building.
* RAMCAT to explore the possibility of a joint research project with Turners on the elimination of asbestos from fibre cement sheet. Apart from the obvious health benefits such a project also means import replacement as asbestos is currently imported from Canada.
* RAMCAT/NEAZDP to investigate the potential for intermediate technology fibre cement roofing tile manufacture in rural areas.

Miscellaneous

Some general points of interest;

* products developed by students and staff at the Institute of Agriculture Research at ABU are advertised in the local press to find manufacturers. The manufacturers have the right to the design in return for a donation of money or equipment to the Institute. Manufacturers may also sponsor further product design and development.
* Some University staff and students see development oriented projects, particularly those of a practical nature, as being of low prestige. Reward systems to alter this perception should be instituted in tertiary educational organisations.
* During travels in Nigeria it has become apparent that there are many geographic similarities with parts of Australia. Problems of development and the role of appropriate technologies in remote communities lacking in resources are also common to Australia. Study tours or other means of Nigerians living and working in selected parts of Australia may be very useful training and educational experiences. The Centre for Appropriate Technology in Central Australia may be a useful model on which RAMCAT and NEAZDP can improve their mode of operation.

Acknowledgements

The author would like to thank Ibrahim Mshelia, Head, Department of Civil Engineering and Building at RAMAT, Habu Yalwa of RAMAT, John Higham and Alistair Dunn of Warwick University for their assistance in the collection of the data on which this report is based.

Thanks are also due to the Rector of RAMAT Alhaji Baba Gara Tijani, Bellar Mamza, Head of RAMCAT and Gaban Tembo of NEAZDP for their support and the Warwick/RAMAT linkage for making this study possible.

The author is also very grateful to those individuals and organisations which gave freely of their time. Without their contribution the study would have been superficial and of little real value.

References

HARDER, L (1991) Cultural Differences and the Problems in Curing and Production, Basin-News, No 2.

MIDDLETON, G.F (1987 rev. ed.) Earth Wall Construction, Bulletin 5, National Building Technology Centre, Chatswood, Australia.

OLA, S.A and MBATA, A (1990) Durability of soil-cement for building purposes - rain erosion resistance test, Construction and Building Materials, vol. 4, No. 4.

SCHRECKENBACH, H (1991) Wall Finishes for Earthen Walls - based on studies in Ghana. Basin-News No 1.

SOLANKE 0. (1984) An application of Borassus Flabillifar to reinforce the superficial leaf of Mudwall. PhD Thesis, ABU.

SPENCE, R.J.S. and Cook, D.J (1983) Building Materials in Developing Countries, Wiley, Chichester.

STULZ, R and MUKARGI, K (1988 3rd ed.) Appropriate Building Materials, SKAT/ITDG.

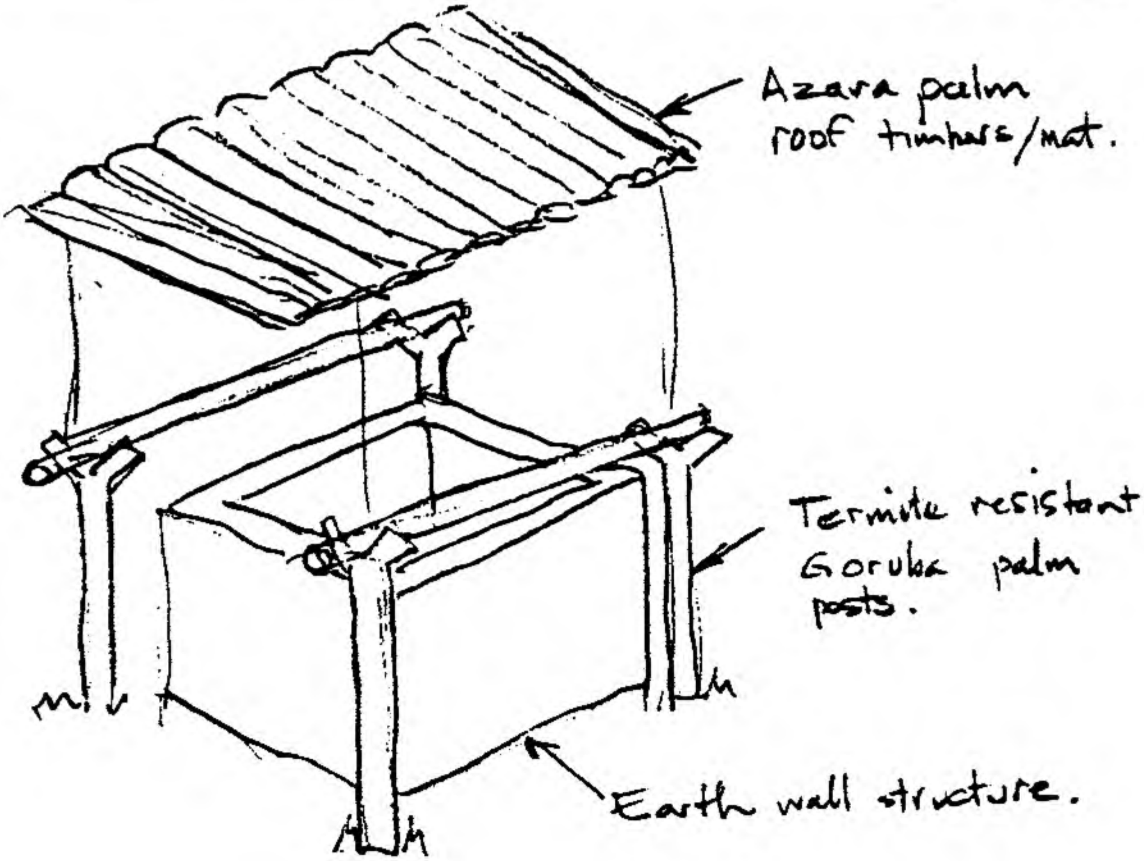


Fig 1. Schematic representation of structural separation of earth walls and roof.

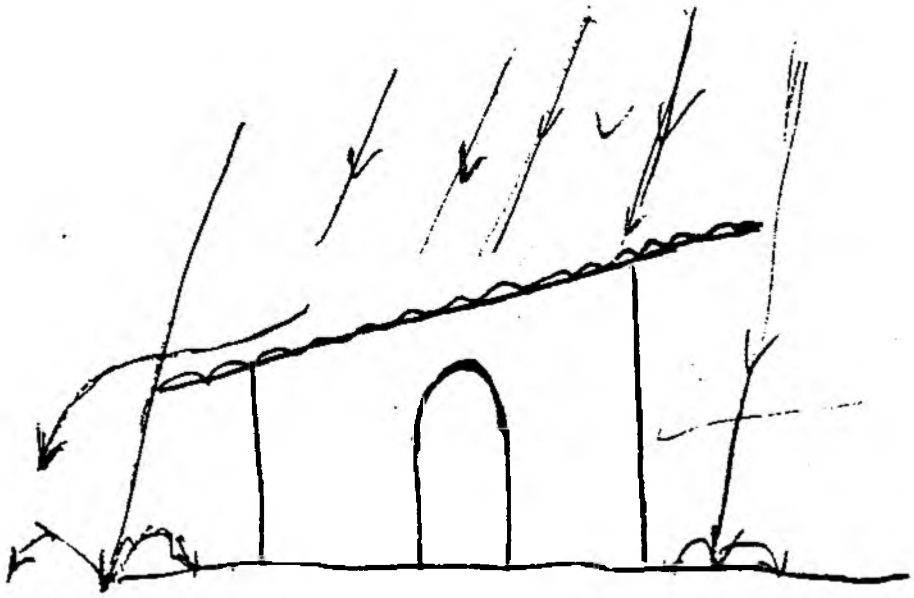
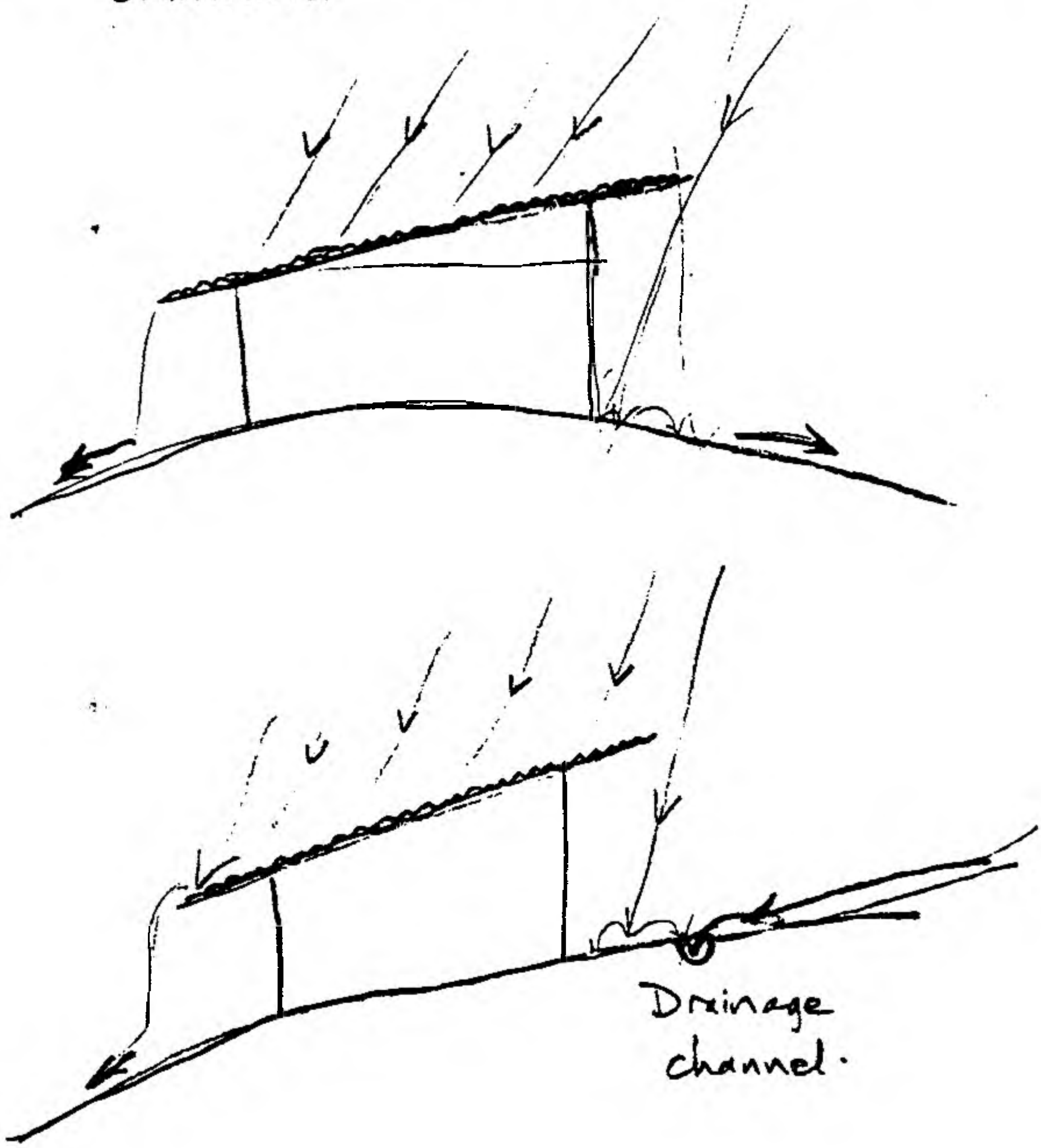
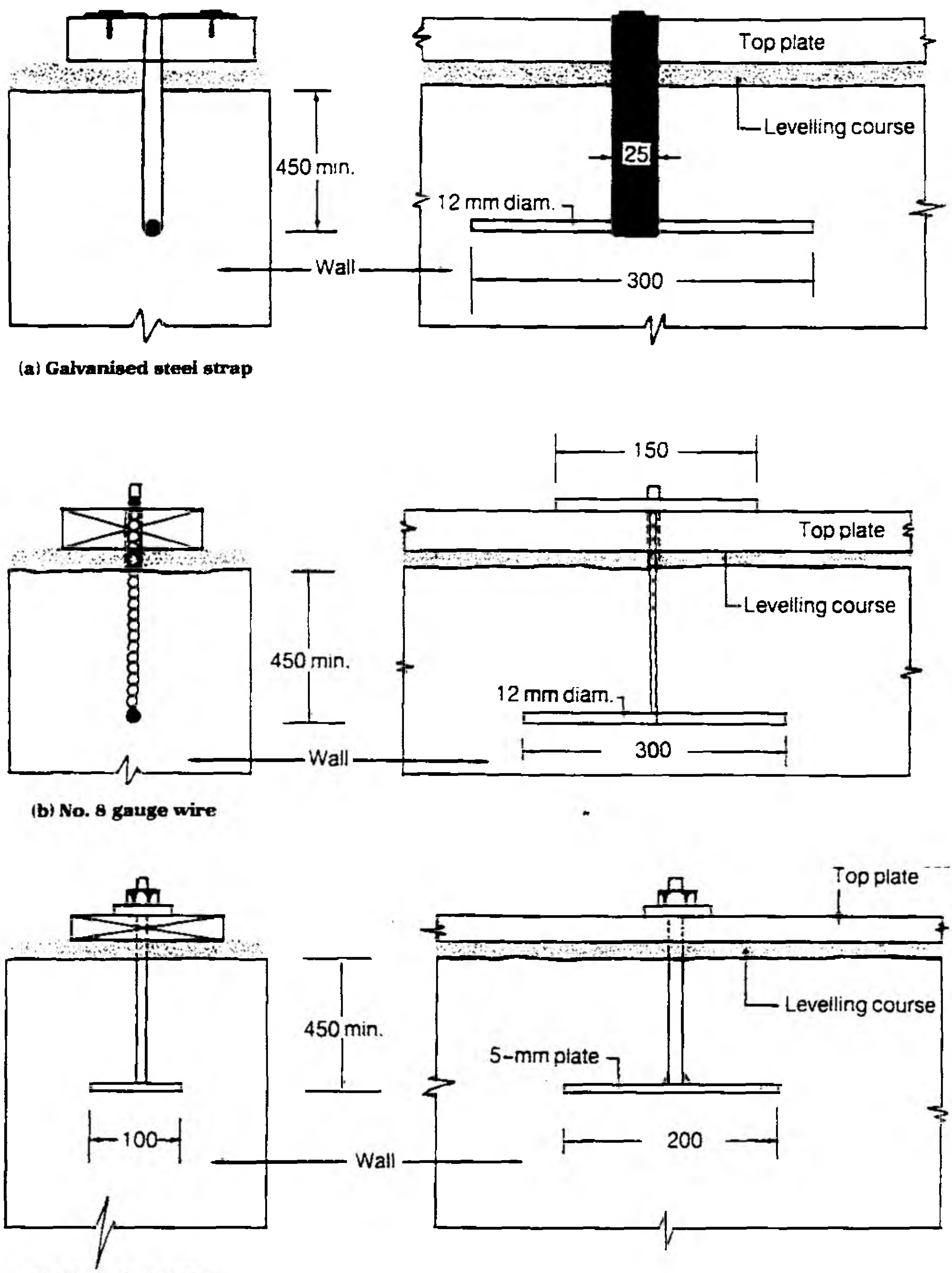


Fig 2. Sloping roof to ensure water run off. Note also extended eaves and arched door.

Fig 3. Siting on raised ground and drainage channels.



All dimensions in mm



~~Fig. 1.4~~—Methods of securing the top plate

From MIDDLETON, G.F (1987)

DEVELOPMENT
TECHNOLOGY
UNIT

D
T
U

Working Paper No. 38

Soil Testing for Soil-Cement Block Preparation

1993

D.E.M.Gooding

Development Technology Unit
Department of Engineering, University of Warwick
Coventry CV4 7AL UK
Tel: +44 (0) ~~1203~~ 523122 Fax: (0) ~~1203~~ 418922
email: dtu@eng.warwick.ac.uk

2476

DTU Working Paper No.38 January 1993

Soil Testing For Soil-Cement Block Production

by Dr D.E.Gooding

ABSTRACT:

This working paper describes how to test soils to determine their suitability for use in soil-cement building blocks. Several reports covering this topic have been published over the last twenty years by a variety of organisations. This paper provides a brief description of the effects of soil properties on the handlability of blocks during moulding and the performance of the blocks after curing. It then undertakes a practical critique of the published tests for selecting soil, and for determining how much cement should be added to them, identifying a number of ambiguities, difficulties of performance and actual error in them. It concludes with recommended testing plans and three appendices. Appendix A describes selected procedures for field-testing soils to be used for block making, Appendix B describes laboratory test procedures: in both appendices the shortcomings identified earlier have been corrected. Appendix C is a bibliography.

CONTENTS:

INTRODUCTION

The purpose of this review is :

- to accurately describe the main soil selection tests and assess their usefulness
- to recommend test procedures for block makers under different circumstances.

The following report examines the process of soil selection for the purpose of soil-cement block production. Initially a brief explanation is given concerning the susceptibility of unstabilised earth constructions to water damage. This is followed by an introduction to soil-cement as a building material giving reasons for its use. Soil suitable for soil-cement construction is then considered from a particle grading and plasticity viewpoint, with due consideration to the underlying mechanisms responsible for strength and durability. Factors characteristic of a generally suitable soil are then put forward followed by a review of more specific criteria, previously published by other authors. The testing of soils for use in soil-cement is discussed with sections highlighting some misleading and inaccurate statements present in some of the soil testing literature. A coherent plan is then given to show the order in which the tests should be used. The main soil selection tests are fully described in Appendix A as field tests and Appendix B (simple) laboratory tests.

1. SOIL BUILDING

Some form of soil covers virtually the whole land surface of the Earth. This soil is usually readily processed with simple hand tools into an easily mouldable material which possesses good compressive strength when dry. Given soil's widespread availability, it is not surprising that it was traditionally widely used as a building material.

The major drawback to building with soil is its susceptibility to water. A soil wall may be considered as a load bearing skeleton of silt and sand glued together by clay. This glue-like behaviour when dry is caused by micro-droplets of water which exist at clay particle interfaces. Clay particles are usually electrostatically charged as a result of surface ion substitution. The charge tightly bonds a thin adsorbed layer of water to the particle's surface. The bonding is sufficiently strong for some adsorbed water to remain even at oven drying temperatures (105 -110°C). At the point of contact between two adjacent particles, a micro-droplet of water can exist where the two adsorbed water layers come into contact. These micro-droplets generate both surface and capillary tension forces which hold the clay particles together. However when any significant quantity of water is absorbed into empty soil pores, the droplets increase in size and the capillary and surface tension forces reduce, causing the soil to quickly soften and subsequently swell. On repeated wetting and drying the outer surfaces of a soil wall expand and contract more quickly than the main body. In a comparatively short time this leads to cracking and spalling of the outer surfaces and low durability for the wall. Moreover if the wall becomes saturated with water the compressive strength may fall sufficiently to allow complete collapse.

There are many methods to reduce a soil's susceptibility to weakening by water. These fall into the following broad categories: protecting the wall from exposure to water, reducing the permeability of the wall by increasing the soil density, making the soil water-repellant by the addition of a water-proofing agent and providing a secondary cementitious-type strength mechanism which is largely unaffected by water. I propose to centre the following report on the final category namely cementitious additives and concentrate on soil-cement.

2. WHAT IS SOIL-CEMENT AND WHY USE IT ?

Soil on its own can be used for construction, but unless it is protected from water the resulting building will not be very durable in any but the driest climates, as has been described above. Cementitious stabilisation in combination with densification gives soil both wet strength and erosion resistance. Densification or compaction reduces the soil's permeability and enhances the secondary cementitious bonding mechanism. Ordinary portland cement is the most commonly used stabiliser and at present usually the cheapest. Lime and lime-pozzolan stabilisation are growing in popularity because, unlike cement, lime may be produced economically by small-scale batching kilns. However, at present the quality of lime produced by such small-scale kilns is highly variable and liable to change from one batch to another. Moreover, a system of price subsidy exists in many countries so that despite cement relying on lime as a raw material and being more expensive to manufacture, it still remains cheaper than lime in the market place. The higher cost and variability of lime have led to the current dominance of cement.

Soil-cement is produced by dry-mixing a suitable soil[1] with a small quantity of cement and re-mixing the product with a specific quantity of water. The resulting damp soil is normally compressed in a mould, ejected and subsequently wet cured for 3-4 days then damp cured for at least two weeks before incorporation in a building. In many ways soil-cement may be seen as a simpler version of sand-cement, not requiring the sand to be first separated from other soil constituents. Sand-cement is widely used, though variable in quality as a result of poor curing.

---

[1]. The criteria for a suitable soil will be examined in detail later but it should be noted that two or more unsuitable soils may be combined by simple mixing to produce one more successful soil.

Soil-cement blocks produced with compression are in general more dense and hence less porous than sand-cement. The resultant reduction of moisture loss during curing leads to a greater consistency in quality for soil-cement.

The minimum amount of cement required to stabilise a block depends on the type of soil, the degree of compression and the final application for the blocks. Generally the interest is to minimise the cement content to below 10%. Given suitable conditions, contents as low as 3% are possible.

The exact mechanism by which a small content of cement may stabilise a large mass of soil is not fully understood. Ordinary Portland Cement is made up of 45% tricalcium silicate (C_3S)[2] and 27% dicalcium silicate (C_2S). In the presence of damp soil these components hydrate to form mono and di-calcium silicate hydrate gels (CSH and C_2SH, see equation below). These gels then slowly crystallise into an insoluble interlocking matrix throughout the soil voids binding the soil particles together. As the matrix is insoluble it gives a strength mechanism which works to restrain the softening and swelling of the unaffected soil, thereby dramatically reducing the weakening effect of water. The interlocking calcium silicate fibres may be seen when a cured soil cement sample is examined under an electron microscope. The hydration of the calcium silicate also results in the release of free lime (CH) according to the reaction:

$$C_3S + 2H = C_2SH + CH$$
$$C_2S + 2H = CSH + CH$$

The free lime then reacts further with the clay fraction (pozzolanic reaction) by the removal of silica from the clay minerals and subsequently forms more calcium silicate gel which also gradually crystallises.

In summary soil cement is a building material which has superior strength and erosion resistance compared to unstabilized soil, without incurring the cost of the large quantities of cement found in concrete.

---

[2]. C and S represent Calcium and Silicon respectively, not carbon and sulphur. This is in keeping with most of the published concrete literature and is acceptable, allowing these simple equations to be given as illustrations instead of the more complicated fully balanced chemical equations.

3. SOIL FOR SOIL-CEMENT

3.1 GENERAL PROPERTIES

Using a suitable soil for soil-cement block production will result in:

- strong blocks, namely those that after curing possess high wet strength and erosion resistance.
- handleable blocks, that immediately upon demoulding can be transferred to a curing area without a high breakage rate.
- blocks which will not seriously distort or crack during curing.
- blocks which will not expand and contract excessively in the building if subjected to wetting and drying cycles.

Specifically disqualified soils are:

- those containing organic matter.
- those which are highly expansive.
- those containing excessive soluble salts e.g gypsum and chalk.

For building purposes soil can be generally characterised in two ways, by a particle size distribution analysis and by a plasticity index. The particle size analysis will give information on the soils ability to pack into a dense structure and the quantity of fines present (combined silt and clay fraction), while the plasticity index gives an idea of the cohesion of the fines.

3.2 PARTICLE GRADING

The British Standard and MIT classification of soil particle sizes is given below:

coarse gravel.................. 60 to 20 mm.
medium gravel.................. 20 to 6 mm.
fine gravel................... 6 to 2 mm.

coarse sand................... 2 to 0.6 mm.
medium sand................... 0.6 to 0.2 mm.
fine sand..................... 0.2 to 0.06 mm.

coarse silt................... 0.06 to 0.02 mm.
medium silt................... 0.02 to 0.006 mm.
fine silt..................... 0.006 to 0.002 mm.

clay.......................... < 0.002 mm.

Gravel is not usually used in soil-cement production, as the large particle size may lead to a poor (rough) surface finish. A suitable soil will contain a mixture of sand, silt and clay-sized particles. The proportions of each of these three fractions influences the properties of the block and will be discussed below.

A particle size analysis will determine the fraction of a soil's particles that fall within each of the above size bands. If a dense block is to be produced, it is important that the soil used is "well graded". The theoretical distribution of particle sizes to provide a perfectly packed structure is called the Fuller curve. The Fuller curve is based upon the assumption that all of the particles are spherical and that the largest particles just touch each other, while there are enough intermediate particles to fill the voids between the largest, but without holding them apart. The intermediate sized particles are also similarly arranged with progressively finer particles filling the voids between larger ones. The Fuller distribution is an ideal model and never occurs naturally. However, a natural soil which has an even distribution of particle sizes, termed well-graded, is a good approximation.

The value of a well-graded soil for soil cement is that such a distribution of sizes gives a dense structure with a low specific surface area. A dense structure is important for several reasons. A densely packed arrangement will have a higher number of contacting particles, giving a better load bearing skeleton. The number and size of the inter-particle voids will be reduced, as will the number of linked voids. This will reduce the porosity of the soil and hence also its permeability, thereby reducing susceptibility to water penetration. As the interlocking calcium silicate matrix extends through the soil voids, a more compact void system requires less cement to provide a matrix of equal efficiency. Similarly if it is imagined that cement coats the soil particles' surface, a high specific surface area will lead to cement blinding, or a lower specific surface area soil will require less cement to provide the same particle surface coverage and consequently the same strength and durability.

The upper and lower limits to the soil's grading also need to be considered. A soil may be considered well-graded with a uniform distribution of particles from fine silt to coarse sand (coarse soil) or with a distribution from clay to fine sand (fine soil). The coarse soil will have a lower specific surface area than the fine soil as the same mass of soil will contain fewer and larger particles.

From the above consideration of specific surface area, it might be concluded that the more coarse soil would produce strong blocks with a lower cement content than that needed for the fine soil. This is however only the case when the blocks are kept within the mould to cure. A coarse soil containing no fines (silt and clay) is non-plastic and will not have sufficient cohesion to retain its shape on ejection from the mould or to allow easy transportation to the curing area. If the blocks are left to cure in their moulds (and the moulds are made strong enough to withstand a significant compaction pressure) then the machinery costs escalate unacceptably. The coarse soil could be considered to be a form of sand-cement containing large voids (a result of the lack of fines). Large voids would increase the porosity of the block and lead back to the common sand-cement problem of rapid drying before the cement has had time to adequately cure. Such a soil would be considered well-graded but still be unsuitable for soil-cement block production. Conversely a well-graded fine soil, containing little sand but a high clay content, would have a high specific surface area and expansivity (see below). The high clay content would give the soil cohesion and stability on ejection from the mould, but the high specific surface area would require a large amount of cement to provide a reasonable particle coverage.

A suitable soil will be well-graded but certain other limits should also be imposed: The largest particle size present should not be sufficiently large to cause a poor surface finish. Sufficient fines (silt and clay) should be present to allow handleability on demoulding but not enough to blind the small quantity of cement to be used.

3.3 PLASTICITY (FINES CONTENT)

The silt and clay content of a soil are responsible for soil cohesion and it is these fines which provide the fresh blocks with handleability until the initial set of the cement has occurred. The degree of cohesion provided to the block is dependant both on the fines present and the degree of compaction used to form the block. In general terms, a low-pressure moulding process will require a higher fines content than a high-pressure moulding process. This is because increased compaction will force the soil particles into more intimate contact, thus strengthening the fresh compact.

However, the fines and in particular the clay fraction can also lead to blinding of the cement as a result of their high surface area (see above). Head (Ref 2, Head 1980) reports that the approximate surface area of fine sand and medium silt are 0.023 and 0.23 square meters per gram, while for three major clay groups, kaolinite, illite and montmorillonite this increases to 10, 100 and 1000 square meters per gram respectively.

The fines also affect the final cured block's expansion on wetting. Clay usually exists in small agglomerations which expand in three dimensions on wetting as water penetrates some of the numerous individual particle boundary fissures. The expansion of the clay fraction must be largely restrained by the calcium silicate matrix in order to minimise expansion and contraction of the cured block, on repeated wetting and drying. Hence for durability the clay fraction should be as small as possible to allow the lowest cement content. It might be expected from the large difference between the specific surface areas of the three clay types mentioned above that different clays have significantly differing expansion characteristics on wetting. This is the case, in general as the surface area of the clay fraction rises, so does the amount it will expand on wetting. As a result the type of clay as well as the quantity present will affect the block.

The fine fraction can be seen to be helpful to the block production process but to adversely affect the wet strength and durability of the final cured block. The quantity and type of clay should therefore be considered important soil parameters. The quantity of fines may be measured by using one of the sedimentation tests described later, however the clay type present is very difficult to determine without highly complex tests. In fact it is not necessary to know the clay type present but it is important to know the properties exhibited by the clay. The Atterburg tests defining liquid limit, plastic limit and plasticity index are used to quantify the plasticity of the finer fraction of a soil (only particles less than 0.425 mm are tested). These tests measure the percentage water contents at which the soil passes from a liquid state to a plastic state (liquid limit) and from a plastic state to a solid state (plastic limit). The numerical difference between the liquid and plastic limit (the plasticity index) thus gives the range of water content over which the soil may be considered plastic. As plasticity is dependent on the soil cohesion, it has been found that this index reflects the cohesive characteristics of the soil. Furthermore as cohesion is largely dependent on the specific surface area of the fines, these plasticity limits also reflect the expansivity of the soil. A soil with a low plasticity index will display low cohesion and usually low expansion on wetting, while a high index soil will display the reverse.

3.4 SUITABLE SOILS

A suitable soil should not contain organic material or excessive soluble salts which would interfere with the setting of the cement. It's sand fraction should be well graded to provide a densely packed load-bearing skeleton for the block and it's largest size particle should be small enough to give a smooth surface finish. The fine fraction should be just sufficient to provide enough cohesion to the fresh block to prevent damage on ejection and transportation from the mould. Too large a fines content will either require a large cement content for adequate stabilisation or will reduce the durability and wet strength of the final cured block. The cohesion of the fresh block will depend on the compaction pressure used and the type as well as the quantity of clay present in the fines.

From the above it should now be possible to see the role that each of the soil's component fractions plays in a soil-cement block and the importance of selecting a suitable soil. If the soil available on site appears unsuitable, it should be remembered that natural soil exists in distinct strata with differing compositions. If the different strata are adequately tested then it is a comparatively simple operation to mix suitable masses of two or more strata to produce an acceptable soil. Given the need to select at least a broadly suitable soil then the case for adequate soil testing should be clear.

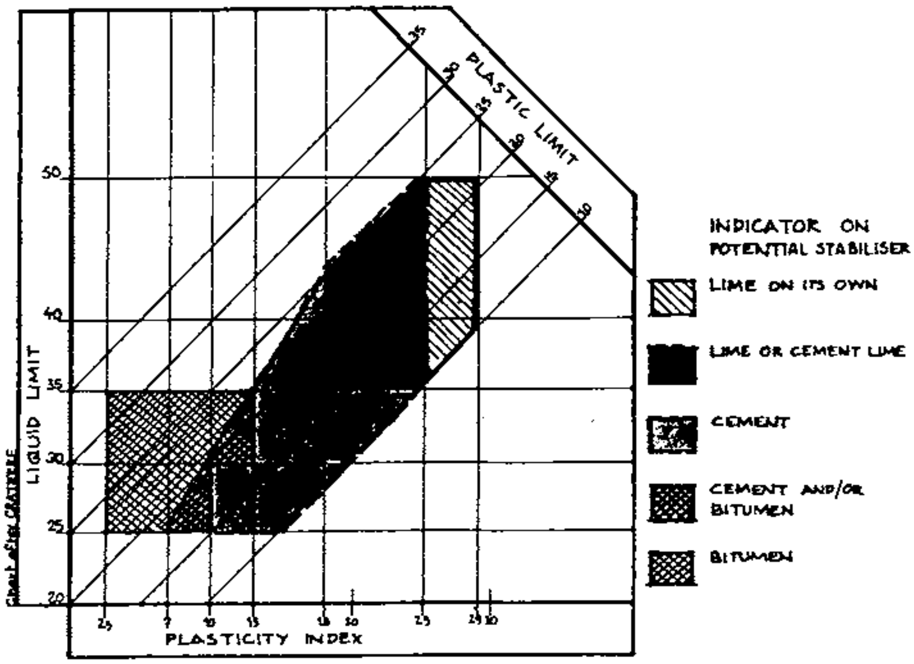
3.5 SURVEY OF CURRENTLY AVAILABLE CRITERIA FOR SOIL SUITABILITY.

The following is a brief review of published selection criteria from other authors. It is not an exhaustive review but rather included as an indication of the variation between authors and as a warning that such criteria should be used as a guide in initial soil selection rather than as a rigid set of rules. This variation is not surprising given the enormous variability of soil itself and the variation in production methods used by the different authors working in different climates. Some of the authors recommend criteria based only on particle size while others use criteria based solely on the Atterburg limits (Plasticity Index). In general it would be wise to consider both.

Ref No.3, Norton 1986. **Building With Earth, a Handbook**.

Atterburg limit criteria for stabilisation:

<u>Interpretation of Atterburg limits</u> (reproduced unmodified by Norton from a CRATerre original, Ref No. 7).



Particle size criteria for soil cement:

Optimum: no specific optimum, "should have a high sand content".

Limits :	sand/fine gravel (<5-6 mm)	45 - 75 %
	silt	15 - 30 %
	clay	10 - 25 %
cement :	variable	8 - 16 %

Not mentioned whether above is by weight or volume.

Ref No.6. United Nations 1964. **Soil-cement, its Use in Building**.

Particle size criteria for soil-cement:

Optimum: 75% sand. 25% silt and clay, of which more than 10% is clay.
Limits : minimum of 45% sand. 55% silt and clay.
maximum of 80% sand. 20% silt and clay.
cement : variable, between 4.75 % and 12.5 % by volume.

Ref No.4. International Labour Office 1987. **Small-scale Manufacture of Stabilised Soil Blocks**

No criteria is explicitly mentioned. Instead it is said that "Ideally, there should be an even distribution of each soil fraction in order to manufacture good-quality stabilised soil building blocks. If this were to be the case, about five per cent cement would be needed as a stabilising agent." The five fractions mentioned are: greater than 6 mm (coarse and medium gravel), greater than 2 mm (fine gravel), greater than 0.2 mm (coarse and medium sand), greater than 0.06 mm (fine sand) and less than 0.06 mm (combined silt and clay).

Ref No.11. Fitzmaurice, Robert 1958. Contained in Spence 1983. **Manual on Stabilised Soil Construction for Housing**

Atterburg criteria for soils most suitable for stabilisation:

liquid limit :	less than 40 percent
Plasticity index :	less than 22 percent and greater than 2.5 percent
Fitzmaurice's note :	primarily derived from temperate soils and only of limited application to tropical soils particularly laterites.

Ref No.2. Stulz, Roland 1983. **Appropriate Building Materials**.

Atterburg criteria for portland cement stabilisation.

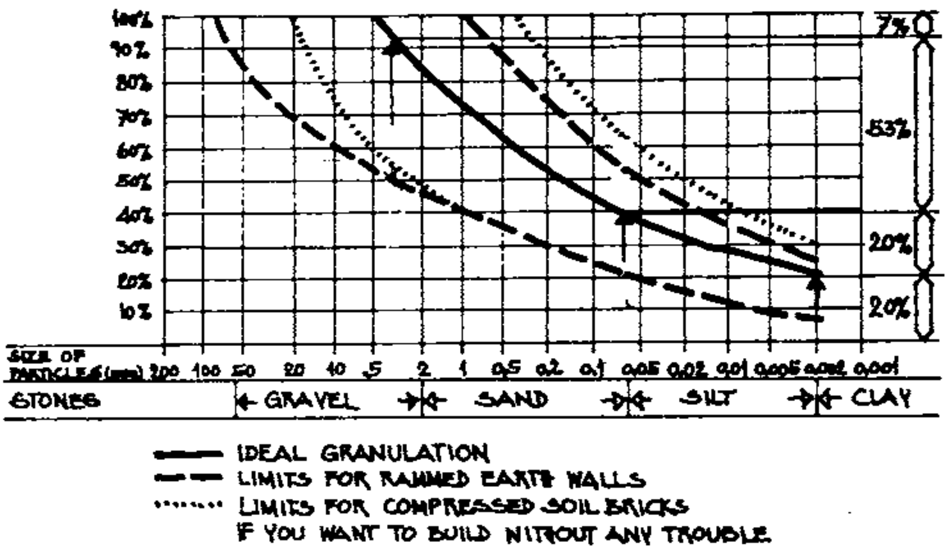
Plasticity index : 0 - 12
Cement content : 6 -10 % (down to 3 % for sandy soils).

Also "cement stabilisation of clayey soils (like red cotton soil) seems not to be useful."

Includes atterburg three-axis graph by CRATerre. Identical to that used by Norton (as shown above) but without a similar key.

Particle size criteria for compressed soil bricks:

<u>Particle size criteria granulation curve</u> included in Stulz after CRATerre (Ref No.7)

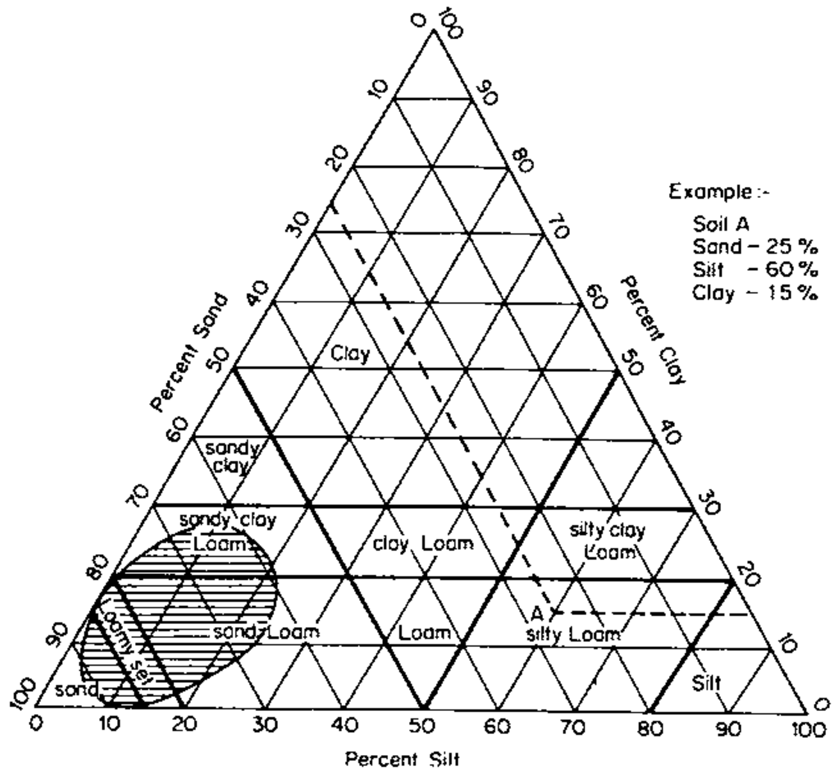


Ref No.11. Spence,R.J.S & Cook,D.J. 1983. **Building Materials in Developing Countries**.

Particle size criteria for soil-cement:

Spence and Cook include a graphical plot on a triangular U.S. Bureau of Public Roads particle-size graph roughly between the limits:
sand:90 - 60 % silt:25 - 0 % clay:25 - 0 %

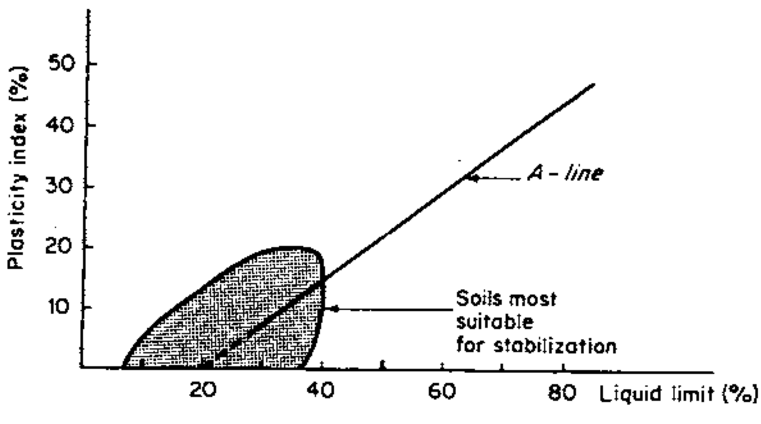
Triangular chart for particle size classification of soils:
(shaded area indicates soils most suitable for stabilisation)



Atterburg limit criteria for stabilisation:

Applicable only to the fraction of soil finer than 0.4 mm, roughly between the limits; plasticity index 0 - 22 %, liquid limit 7 - 40 %.

Plasticity chart showing soils most suitable for stabilisation



It can be seen from the above that there have been a number of criteria put forward for soil selection based on particle size or Atterburg limits or both. In broad terms these criteria are in agreement. A soil suitable for cement stabilisation should have a significant sand content (at least greater than 50%, preferably closer to 75%) and a low plasticity index & clay content (typically less than 25% clay). These criteria are however intended for use as a broad initial guide for soil selection. It must be emphasised that the testing procedure is not complete until the soil or soils selected have been used to produce, cure and test a trial set of blocks. Only after a trial set of blocks have been tested and proven to be acceptable should the main production run begin.

4. TESTS FOR SOILS.

4.1 TYPES OF TEST

Prior to soil-cement block production there are three main types of test which may be conducted:

First, field tests can divide the soils into broadly suitable and unsuitable categories and if suitable into potential high and low cement classes.

Second, laboratory tests can be used to characterise the soils by particle size distribution, plasticity or other numerical measures for relation to the selection criteria (see section 3.5) and enable simple soil modification by blending. Most small-scale manufacturers of blocks, especially those producing blocks at a rural building site, have little or no access to laboratory facilities and in particular accurate mass measurement to 0.01 g. For these block makers, judicious use of the field tests, the shrinkage test, production trials and past experience has to suffice. The laboratory tests are appropriate where medium or large-scale production is planned, where minimising cement content is especially important or when soil-cement block making is moving into a new area.

Third, trial production tests can be carried out on manufactured blocks to check that the final block properties required (dry strength, wet strength and durability) can be achieved. This paper is concerned with tests which are carried out on the unformed soil. It will therefore not cover the trial block testing procedures; for information on compression and durability testing of formed blocks, "Small-scale Manufacture of Stabilised Soil Blocks" should be consulted (Ref 4, ILO 1987).

The field and laboratory test procedures reported in the literature have been conducted by the author and evaluated using a carefully characterised soil (of a type suitable for soil-cement block making). For each published test he observed the accuracy of its description, its ease of performance and the accuracy of its results (in terms of internal consistency and agreement with British Standard Tests). A number of the tests examined were found to be misleading and incorrect in parts. The following sections are concerned with highlighting these problematic areas in an attempt to improve testing procedures as a whole.

4.2 FIELD TESTS

Field tests are for preliminary site surveying, to identify the soils most likely to be suitable and so restrict the number of soils to be more rigorously assessed by laboratory tests or trial production. The tests (described in appendix A) will provide a rough idea of a soil's grading and plasticity and also indicate whether a soil contains significant organic matter (reject outright), a predominance of gravel, a predominance of sand or a predominance of fines. They may also be able to distinguish whether silt or clay is the more significant fraction of the fines. They are generally fairly easy to perform and often require little or no experimental equipment, making them very cheap to implement.

However field tests are frequently reported without acknowledging the reliance they place on the operator's senses: although the methods employed are generally simple, the interpretation of the results is a skilled operation. Consider for example the dry strength test. The prepared soil sample is crushed between the fingers and the ease of crushing is taken as a measure of the soil's clay content. For a novice operator the ease of crushing is difficult to assess and as a result so too is the clay content. A skilled operator may compare the ease of crushing with that of soils he/she has previously tested and hence arrive at a more precise conclusion. Tests which rely on personal judgement are open to differing interpretation between operators and depend on the operator's skill for their accuracy. With training and experience these tests may provide a fast, quite accurate determination of the soil's characteristics, however for a novice they can only be expected to provide a more basic picture.

Table 4.2 (below) shows which tests are reported by which publication. The glass-jar sedimentation test will be discussed under laboratory tests (section 4.3) as it contains problems in common with the syphon sedimentation test. The remaining field test methods are generally in agreement and as such no further detailed comments will be made. The test descriptions and notes included in appendix A have been compiled by the author and are a combination of earlier reported methods and the author's own modifications. Each test begins with a brief resumé by the author giving comments on the use to which the test may be put, the accuracy which may be expected from the results, the time taken for completion and the limitations of the test.

All of the test results observed (both the good and the bad), plus the location and depth of the soil samples in question should be recorded in case it is later necessary to use a soil for blending which on preliminary examination had been rejected.

TEST NAME	REF.1	REF.2	REF.3	REF.4	REF.5	REF.6
SMELL	√	√	√	√	√	X
VISUAL-TOUCH	√	√	X	X	X	√
THREAD	√	√	X	√	ND	X
RIBBON	√	√	√	X	ND	√
SHINE	√	√	√	√	√	√
SEDIMENTATION GLASS-JAR *	X	√	√ a	√ b	√ c	√
DRY STRENGTH	√	√	√	X	X	√
SURFACE WATER	√	√	X	X	√	√

Table 4.2. Reported Field Tests

ND : Mentioned but not adequately described.
* : These tests are described in the laboratory test section covering sedimentation test.
a : Ill advised recommendation to add salt, ignores flocculation.
b : Over-concentrated solution causes inaccurate estimation of sand and fines content, salt added ignoring flocculation
c : Over-concentrated solution, test not intended to discriminate fines into silt and clay

4.3 LABORATORY TESTS

4.3.1 GENERAL CLASSIFICATION

The laboratory tests establish numerical values for certain soil parameters, primarily the percentage distribution of the different sizes of soil particle present and the plasticity limits. These values are subsequently used to determine the best available soil or combination of soils. All of these tests rely on accurate weighing and or some form of laboratory equipment. Scales with a resolution higher than one thousandth of the chosen sample weight are desirable.

There are four main types of test:

The sieving tests separate the different size fractions of the soil into discrete parts thereby indicating the soil's particle grading. The silt and clay fractions are too small to be easily separated by sieving and as such are normally reported as a combined fraction. The larger particles may be separated into a number of size fractions, depending on the number of sieve seizes available, according to the MIT and British Standard particle classification boundaries, given in section 3.2. A full laboratory analysis would give the percentage by weight of each of these size bands.

The sedimentation tests if correctly conducted have the ability to separate the larger sand and gravel size fractions from the combined fines fraction and under favourable circumstances to further distinguish the combined fraction into separate silt and clay fractions. However the simplest test, the glass-jar sedimentation test, is usually included under field tests because visual discrimination of the silt/clay boundary may not be possible. In this case the test can only be used to give an idea of the general relative proportions of sand and fines. In its coarsest form the glass-jar sedimentation test provides no more information than a sieving test and although less accurate, it does not require any mass measurement. Further, although the sedimentation time is long the operator time required to conduct the test is less than that for a sieving test.

The Atterburg or plasticity tests define the soil's liquid limit, plastic limit and plasticity. The test methods included are simplified versions of the more rigorous British Standard methods after Norton (Ref 2, Norton 1986). The Atterburg limits allow the soils plasticity characteristics to be related to the criteria given above in section 3.5.

The shrinkage test is a test of the soil's contraction on drying and gives a combined measure of the soil's particle grading, plasticity and clay type. It gives an overall idea of the soils behaviour and suitability for stabilisation. The degree of contraction may be thought of as a measure of the expansive force which the soil stabiliser will have to withstand when a manufactured block is exposed to water. The degree of contraction is then taken as a measure of the quantity of stabiliser required. The shrinkage test may be used as a straight-forward method of determining a soil's suitability for use where more complex testing is not possible or not justified for small-scale production. However it must be remembered that this test gives no direct information on the soil's constituent parts and as such will not allow easy soil modification. It was empirically designed for use with the Cinva Ram, a low-pressure (2MPa) manual-compaction moulding machine developed by VITA. It was intended to gauge the amount of stabiliser required for a given soil compacted with this machine[3]. It is very suitable

[3]. It should be remembered from the above discussion of soil suitability that the compaction pressure used to compact the block does affect the soil requirements. The shrinkage test was empirically calibrated for the Cinva Ram (2MPa) and is not directly applicable to a machine operating at a different compacting pressure. In general if the

for small-scale production if soil modification is not considered cost-effective but it must be used in conjunction with tests on trial blocks.

If the results from these tests are to be useful, a great deal of time and care must be taken. This point is seldom mentioned. These tests appear simple to carry out and they produce numerical values which are relatively easy to interpret, but they are not fool-proof and will produce misleading results if not carefully performed. The sedimentation tests in particular are very delicate, requiring time and practice to perfect. In general soil tests are subject to two accuracy limitations, experimental care and measurement resolution. The following four sections deal with each of the four main test types, giving a simple theoretical background and examining certain misleading and inaccurate aspects contained in earlier reported test methods.

4.3.2 SIEVING TESTS

The sieving tests may be conducted wet or dry, on a complete natural soil sample or on the residue from a syphon sedimentation test. In order to appreciate which of these is the more suitable for any given circumstance a brief consideration of the underlying theory should be given. A sieve test separates the soil fractions by allowing particles with a diameter slightly smaller than the diameter of the sieve holes to pass and retains those which are slightly larger. For an accurate determination of the size fractions present the soil particles must be separate i.e. the soil should be in distinct particles not agglomerations of particles. The ease with which any given soil may be broken up into separate particles determines which method of sieving is appropriate. It should be noted here that dry sieving is only recommended by the British Standards Institute (BS 1377) for clean sands and gravels (i.e. without any significant quantity of cohesive material).

A sieve test conducted on oven-dry soil particles (Dried to constant weight at 105-110°C) should be preceded by a breaking-down operation where the particle agglomerations are broken into separate particles. For low cohesion soils, those with only a small clay content, this is quite readily done with a pestle and mortar; however for soils with a high clay content this may be very difficult. If the soil is not adequately broken down then an overestimate of the larger sizes and an underestimate of the combined silt and clay fraction is likely. This is particularly so for lateritic soils which become very hard on drying. In this case a significant quantity of clay-sized particles may remain trapped with the larger sand sized particles. If on examination it appears that the soil has not been completely broken down then

---

machine compacts to a higher pressure then the cement content may be reduced for a given soil shrinkage, or alternatively the range of acceptable soil shrinkage values may be increased.

the soil is unsuitable for dry sieving and should be wet sieved or sedimented and subsequently dry sieved (see below).

For wet sieving a measured weight of oven dry soil is soaked in a large quantity of water or preferably water and a suitable dispersing agent. By soaking the soil any particle agglomerations soften and subsequently break up if the resulting suspension is adequately stirred. In order to successfully sieve this soil suspension a large quantity of excess water is required both to wash the particles through the sieves and to separate those particles which loosely adhere to each other as a result of the water's surface tension. Moreover a number of particles slightly smaller than a given sieve's diameter may be retained by water tension across the sieve holes. As a result an improvement in accuracy will be found if the retained samples are dried and resieved.

If the soil is first subject to a syphon sedimentation test, which removes the clay fraction, then a dry sieve test may be conducted on the settled soil residue. This soil residue will be cohesionless, if sedimentation separation has been successful, and therefore very easily broken down into separate particles.

ILO (Ref 4, ILO 1987) reports the dry sieve test as a "further soil testing procedure" without any mention of the necessity to break down the lumps of soil which are usually formed on drying and the consequent inaccuracy. The ILO also includes a section on laboratory testing methods which are "briefly discussed". A wet sieve test is mentioned[4] but without discussing when or why it should be used in preference to the dry sieve test, indeed the only sieving test method contained in the publication is the under-explained dry sieve method.

Norton (Ref 3, Norton 1986) does not report a dry sieve test on soil in a natural condition but rather only a dry sieve test on the residue from a syphon-sedimentation test. This is acceptable providing that the sedimentation test is correctly carried out. However the syphon-sedimentation test as reported by Norton may lead to flocculation (see Sedimentation Tests below) and consequently lead to subsequent further inaccuracy in the dry sieving.

A wet sieve test has also been reported by Norton. He states that "it should be used for analysing lateritic soils in order to ensure that clay particles trapped in fissures on larger particles are washed out." However he does not advocate soaking of the soil to facilitate this but rather to "mix the soil sample with water, and wash it through the sieves. " If the soil sample is not soaked before mixing then significant quantities of clay will remain adhered to the larger particles. The wet sieve test

---

[4]. A reference is given to a Road Research Laboratory paper, West,G & Dumbleton, M. J. "Wet sieving for the particle size distribution of soils" (Crowthorne, United Kingdom, Road Research Laboratory, 1972).

relies on water to disperse the soil grains, if sufficient soaking time is not allowed for this dispersion to take place then the test will be subject to the same inaccuracies mentioned above for the dry sieve test.

Norton does not mention that the initial soil sample to be tested should be carefully weighed out nor does he state whether the sample should be oven dried, air dried or damp. Rather he suggests that all of the material remaining on the sieves should be dried out and weighed and that the material carried by the wash water should be collected, dried out and separated with the syphon sedimentation test. In order to sieve such a wet sample a considerable quantity of water is required to wash the particles through, frequently tens of litres (several gallons). To collect and dry such a large quantity of water is both time consuming and impractical without very large collection vessels. If this method is employed for soils containing significant quantities of combined silt and clay then concentration problems will be encountered with the syphon sedimentation test. For example if 1kg of a fine soil containing 40% combined silt and clay fraction is wet sieved either 400g of material will have to be sedimented (four times the recommended concentration) or the dried material will have to be re-wetted, thoroughly mixed (to evenly re-distribute the silt and clay fractions) and subdivided before re-drying to ascertain the new dry weight of the smaller samples.

The wet sieving test, as reported by Norton, will not give reliable results unless the soil is left to soak adequately and will be very time consuming if the wash water is collected and dried. A more sensible method would be to accurately weigh an oven-dried soil sample, soak this in water or preferably water and a dispersing agent and allow the wash water to go uncollected. The weight of the separate dry retained materials may then be related to the original dry sample weight to give the percentage of each size fraction and the combined silt and clay fraction may be assumed to be the difference between the original total dry sample weight and the sum of the dry fraction weights. The clay content may be determined by a separate syphon sedimentation test and the silt fraction assumed to be the difference between the original sample weight and the combined sand, gravel and clay fractions.

4.3.3 SEDIMENTATION TESTS

The sedimentation tests are based upon Stoke's law of sedimentation which predicts the velocity in free fall of any diameter spherical particle of known specific gravity in a fluid of known viscosity at low concentration. For sedimentation testing it is assumed that the specific gravity is the same for each soil particle, each particle is approximately spherical and each soil grain exists as a separate particle. Hence the rate of fall is dependant only on the diameter of the particle. Larger diameter particles will fall more quickly than smaller diameter ones and hence the settled material will be graded with large particles at the bottom and fine particles at the top. One problem which may be thought of with this method of separation concerns the distances that particles have to fall. A small particle initially close to the bottom of the vessel falling slowly may settle in the same time as a large particle initially at the top of the vessel, leading to contamination. This does occur, but as the velocity of fall is proportional to the square of the particle diameter larger particles fall significantly faster than small ones and the contamination is only minor.

At high concentrations the particles interfere with each other, leading to "wipe-down" whereby small particles are carried down by larger ones. Similarly if the soil sample is not sufficiently dispersed agglomerations of particles will fall more quickly than would be the case for separate particles. Particle agglomeration may occur as a result of two separate factors; firstly, as the result of insufficient soaking whereby particles, primarily clay and silt, remain bound together or bound to larger sand particles and secondly when the silt and clay particles, initially dispersed, reassociate as a result of electrostatic interaction to form flocs (flocculation). In either case settlement will be affected and the measured fractions incorrect.

The glass-jar sedimentation test uses the differential settlement phenomena to give an idea of the relative proportions of different sized particles. A suspension of soil is allowed to settle undisturbed in a parallel sided vessel. As a result of the differential settling and a usually slightly discontinuous range of particle sizes, the material forms settled layers of gravel, sand, silt and clay. The height of each different soil layer formed is measured relative to the total settled height and taken to be the relative proportions of each discernable size fraction. The formation of layers is readily visible in light coloured soils which do not contain a perfectly continuous range of particle sizes but for other soils the layers may be less visible. For most soils it is possible to determine the boundary between the sand and silt layers as sand grains may be individually discriminated while silt grains appear as a homogenous mass. However it is frequently difficult to see the boundary between silt and clay as both material's grains are too small to be discerned. More complex timed methods have been put forward to attempt to overcome this discrimination problem but there are problems with these too (see below).

The syphon sedimentation test also uses the differential settlement phenomena. Rather than attempting to use the settled layers as indicators of the sample's different size fractions it attempts to separate the clay fraction by allowing heavier soil fractions to settle out of a suspension so that the remaining fluid, containing the clay particles, can be dried separately. If the initial dry soil mass is known then the percentage of clay may be found. This test depends on the clay remaining in suspension longer than any other heavier soil component and as such relies on the initial suspension being dilute and effectively dispersed. If "wipe-down" or agglomeration occur then the material syphoned off in suspension will be less than the true clay fraction. Moreover if flocculation occurs it is frequently not possible to discern the level of the settled material and hence the correct level for the separation disk. The flocs, containing both silt and clay particles, interfere with each other and slowly settle en masse in a loosely packed arrangement rather than as discrete particles. In this condition silt does not settle significantly faster than clay and hence cannot be distinguished. The formation of a flocculated suspension is usually readily apparent as a pronounced clear layer of water will form quite rapidly above the remaining suspended material during settlement. If the syphon sedimentation test is to be used the soil suspension must be both fully dispersed and deflocculated, a point frequently neglected by the literature.

The "sedimentation bottle test" (glass-jar sedimentation test) of ILO (Ref 4, ILO 1987), reports that "the bottle is first filled to one third with clean, uncontaminated water. Approximately the same volume of dry soil (which has passed through the 6 mm sieve) and a teaspoon full of common salt are added. Salt facilitates the dispersion of soil particles." Using equal volumes of dry soil and water will give a highly concentrated suspension of soil and lead to significant wipe-down of the fine fraction (see above). The diagram included with this description actually worsens this situation with mistaken captions. Diagram 1 shows "1. Bottle one third filled with water." diagram 2 then shows a full bottle with a one third volume of water resting on a two thirds volume of soil stating "2. Add one teaspoon full of salt and fill bottle with soil." Filling a bottle containing one third of water with dry soil will produce an intensely concentrated suspension.

Having shaken the soil bottle it is stated that "Two or three minutes later the water will start clearing....Two or three distinct layers will be observed, with the lowest layer containing fine gravel, the central layer containing the sand fraction and the top layer containing the combined silt and clay fraction....The individual percentages can be determined by direct measurement of the depth of each layer." This is most misleading. The above implies direct measurement of the layer height after only two to three minutes. Only the sand sized fraction would have settled in this short time, silt and clay particles fall much more slowly (clay falling at approximately twelve millimetres per hour) and would still be in suspension. Moreover unless the soil were to be predominantly clean gravel and sand, a high concentration of fines would be present which would not enable "distinct layers" to be seen rather the entire depth would appear muddy. This sedimentation bottle test has failed to produce any distinct layers when performed by the author on a known well-graded soil containing seventy six percent sand, fifteen percent clay and nine percent silt. The solution was too concentrated, the whole appearing as thick flocculated liquid. The relative volume of soil to water should be reduced to at least one quarter to three quarters.

Furthermore, Grimshaw (Ref 9, Grimshaw 1971) has reported that salt is a clay flocculant causing these particles to agglomerate into larger flocs, not to disperse as mentioned above. The addition of salt has been put forward, by Webb (Ref 5, Webb 1988) reporting that "salt will speed up the final settlement of particles". This is correct as the flocs formed are larger and heavier than individual clay particles and hence fall more quickly. However Webb puts forward this glass-jar type sedimentation test with salt to quickly and roughly determine the relative sand and fines content. It must be remembered that the fines are not separated into silt and clay fractions and may not be distinguished when flocculation occurs (see above). More suitable dispersants which do not cause flocculation are listed in appendix A page 39.

Norton (Ref 3, Norton 1986) suggests a "Simple particle separation by sedimentation" test which uses a timed observation system rather than visually discriminating settled layers. This test advises that a jar should be one third filled with slightly compacted dry soil and this height (h), measured from the base of the jar, be recorded. Water and a pinch of salt is then added to fill the jar to three quarters full. The jar is shaken, soaked for one hour and re-shaken. After the final shaking the jar is left to stand and a stopwatch is started. "When one minute is up, mark on the side of the jar....This amount (T1) is fine gravel and sand....After 30 minutes mark again....(T2) is fine gravel, sand and silt together. After 24 hours mark again....(T3) includes fine gravel, sand, silt and clay. The depth of clay = T3-T2. The depth of silt = T2-T1. Divide each depth by the total (h)...." and so gain the percentage proportion of each particle size. As has been mentioned earlier, a dry soil will expand on wetting. If the settled heights are related to the initial compacted height of the soil as described, then in general the sum of the soil fractions will exceed one hundred per cent. A more satisfactory solution is to relate the measured heights to the total settled soil height of the soil after twenty four hours. Again it is recommended that "a pinch of salt" should be added. This is not correct, in this test Norton is proposing to separate the silt and clay fraction and has apparently ignored the flocculating effect of salt. If flocculation occurs the level of the fully settled material will be obscured by the semi-settled flocculated layer. If the top of the flocculated layer is taken to be the settled height nonsensical results will follow as the settled height apparently reduces as the floc settles. Again flocculation will be apparent as a marked clear layer quickly appearing above the remaining suspended material. If flocculation occurs the suspension must be deflocculated.

Soil flocculation may occur without the addition of salt (chlorinated water among other things may have this effect), if it does then the suspension must be treated with one of the compounds listed in appendix A, page 39, to deflocculate and redisperse it.

4.3.4 ATTERBURG TESTS

The Atterburg or plasticity tests define the moisture content at which the soil passes from a liquid state to plastic state and from a plastic state to a solid state; these boundary points are the liquid and plastic limits respectively. The transition from liquid to plastic to solid is a gradual process, viscosity and shear resistance increase as the water content decreases. The precise boundaries between the states are defined by the tests themselves and not as a result of theoretical analysis or an intrinsic soil property, e.g. the plastic limit of a soil is the moisture content at which a thread of soil with particles greater than 0.425mm removed will break when rolled down to 3mm. Because of this reliance on the testing method different test procedures and even different test operators will give varying results. It is therefore most important that care is taken to follow the method given and to have the tests conducted by the same operator. The most important piece of equipment for the plasticity tests given in appendix B is the hand of the operator.

The plastic limit test described in appendix B is that used by the British Standards Institute (BS1377). The liquid limit test reported is a simplified version of the Casagrande liquid limit test. The full Casagrande test requires the use of a piece of specialised equipment which mechanically taps the curved dish by vertically dropping it a set distance at a set rate. In the simplified version of the test the curved dish is tapped horizontally by the operators hand. The simplified test given was first reported in "Handbook for Building Homes of Earth" (Ref No.1) and subsequently repeated unaltered by Norton (Ref 3,Norton 1986) and Stulz (Ref 2, Stulz 1983). It should be remembered when using this variant of the test that it is likely to give more variable results than the original. The force which is used to manually tap the curved dish depends on the operator, so it is desirable that the same operator conducts each test if comparisons are to be valid.

Other authors either describe very similar tests to those given in the appendix or refer the reader to the British Standard tests (BS1377). Concerning those tests which they describe two points need to be mentioned. Firstly sample preparation may be incompletely specified, it is not always clear that the soil sample to be tested must have all particles larger than 0.425 mm removed prior to testing for both the liquid and plastic limit tests. Secondly, the soil mixing operations should be very thorough. The soil should be mixed for at least ten minutes (up to 30 or 40 minutes for heavy clays). Mixing should continue for several minutes even after the disappearance of any wet or dry spots. For the liquid limit test it is not sufficient to add and mix soil or water to the sample while it is still in the curved dish. The sample should first be removed from the dish to allow mixing in a larger, more suitable container.

Stulz (Ref 2, Stulz 1983) suggests that "If you already know that you are going to add a stabiliser to your soil, then add the same proportion of stabiliser to your sample as you intend to use in your house". This is misleading as the term stabiliser is normally used to include cementitious compounds; cement, lime and pozzolanas etc. I believe that by "stabiliser" Stulz is referring to soil modifiers i.e. sand and clay rather than stabilisers. The modified soil should be tested but without the addition of cementitious stabilisers which will dramatically change the plasticity of the soil. In the case of cement the hydration reaction begins immediately the cement contacts water and initially progresses quickly. As a result the plasticity of the soil will change quickly with time and not allow any meaningful results to be obtained.

4.3.5 SHRINKAGE TEST

There is a large number of shrinkage-type tests which have been reported. The test which will be discussed here is a linear shrinkage test conducted on natural soil which has had particles larger than 6mm removed. This test has been included as a laboratory test because it requires a large mould and up to seven days of drying. The shrinkage test gives an idea of the gross behaviour of the soil on drying. The change in length of the soil sample may be considered to represent the expansion force which the soil stabiliser will have to resist when the final block becomes wet. In general the smaller the soil's contraction on drying the smaller the quantity of stabiliser required.

This test has been reported with two different but broadly similar experimental techniques. The method included here requires the soil mix to be at or near its liquid limit, while the other method frequently reported requires the soil to be at its optimum moisture content for maximum density moulding. The near-liquid-limit method has been chosen as this mixture of soil will contain more water and hence give slightly higher shrinkage values. The greater variation in liquid limit moisture content (between soils) compared to the more similar optimum moisture content will give a broader range of shrinkage values for different soils and hence will allow better discrimination. Again the recommended cement addition given by this test are only a guide and must be verified with trial block production.

The test has been calibrated by VITA for use with the Cinva Ram compacting machine (details are given with the test in appendix B) but not for other machines. Webb (Ref 5, Webb 1988) has suggested a very similar set of values for the Brepack machine which operates at five times the compaction pressure of the Cinva Ram, however it appears that the two sets of data are not comparable as the set given for the Brepack will produce blocks to a higher strength standard than that for the Cinva Ram. The cement saving appears small unless blocks of the same strength are compared. For instance, Webb cites blocks produced in Kenya from "Murram soil containing about 16 per cent clay stabilised with 4 per cent cement by weight under a compaction pressure of 10 MPa" and states that these "compared favourably with blocks made on a block press machine which used 18 per cent cement as a stabiliser. In this case the compacting pressure was 2 MPa". In this case the 18 per cent cement content used with low-pressure compaction was apparently equivalent to a 4 per cent content at high-pressure compaction. This is an extreme example but does illustrate the trend which is not apparent from the table included with the shrinkage test in Appendix B.

One final point to mention with respect to the cement content table from Webb is that for shrinkages of less than 15mm (in 600 mm) the soil should not be automatically rejected. It is not clear why Webb has chosen to reject this class of shrinkage. If the soil does have some plasticity, sufficient to allow adequate green strength for demoulding, then a low shrinkage soil should produce admirable blocks when compacted to high pressure. Lower shrinkage on drying will reflect the soil's potential to produce blocks which will be less prone to expansion on wetting and hence more durable. It is the requirement for green strength on demoulding which governs the minimum cohesion and hence shrinkage value. It would be expected that a high pressure machine should be able to handle soils with lower shrinkage than would a low pressure machine; from the VITA table the reverse would appear to be the case. A better guide would be that at either pressure soils with shrinkage down to a nominal value of 5mm should be investigated but zero shrinkage materials (0 - 5 mm) should be rejected.

This test is most useful where the scale of production does not justify the use of more elaborate tests or where it has been initially decided that soil modification will not be used. It does not give useful information for predictive soil modification but may be used to check the effectiveness of soil modification by trial.

5. COHERENT SOIL TESTING PLANS

In general the literature concerned with soil testing provides a number of suitable tests but does not provide a logical testing plan for their implementation. The following section discusses the soil-testing needs for differing project sizes and purposes. From this discussion it is hoped that the reader may be able to appreciate the need for different scales of soil testing. The large variation in scale of production, climatic conditions and use to which the final structure is put does not lend itself to specific recommendation, however certain generalisations are possible and would appear helpful as these are usually lacking elsewhere. The section is completed with an example of a coherent testing plan, comprising a testing tree for the field tests and a set of coherent laboratory tests, suitable for a medium scale producer.

In general there are two paths which may be followed by a soil-cement block producer, to use the available soil in its natural state or to use a modified soil (one produced by the combination of two less suitable soils). The decision whether to modify the natural soil is a complicated one. If the available soils are quite unsuitable for block production then either the soil must be modified or an alternative site must be found. Often however, although the available soil is acceptable for production in its natural condition, if it is modified blocks may be produced which are either of better quality or cheaper. The former is achieved by maintaining the cement content while improving the soil hence increasing block strength, the latter by maintaining the strength while reducing the cement content. The difference in cost or block properties resulting from modification depends on the degree of improvement which would be possible. The further away from the "ideal" soil the natural soil lies then the greater the improvement possible and hence the greater the justification for modification.

In small-scale block production, for example for a single building (self-built unit), the savings made through soil modification of an acceptable soil[5] are likely to be small. In this case the additional cost in terms of time and equipment required to perform all of the laboratory soil tests may not be justified. If the soil appears suitable from the field tests and the simple shrinkage box test, it would generally be more appropriate to use the natural soil, increasing the cement content if greater wet strength is required. If none of the available soils are suitable then modification will be necessary. In this case modification may be done by trial and error, checking the results with the simple shrinkage box test. This will then not require the grading or plasticity to be known but will take a significant time to perform adequately (the shrinkage test may take up to 12 days to complete). If the equipment is available it will always be beneficial to conduct the laboratory tests but adequate blocks may be produced without. The most fundamental piece of equipment required for laboratory testing is an accurate weighing balance, ideally capable of weighing to one thousandth of the sample weight.

In medium-scale block production, for example local village/community building programmes, the economies resulting from modification may be more significant and hence justify the increased testing costs resulting from a more complete laboratory testing program. Such a programme would include the determination of the soil's grading and plasticity characteristics. A more complete testing program enables faster more reliable modification processes to be used. The soil may be predictively modified to meet the criteria mentioned earlier in section 3.5, rather than imprecise modification by trial and error. The choice between modifying or not modifying should be based on the relative cost of the cement to that of the labour or machinery required to perform the additional soil blending

[5]. "acceptable soil" here means one which may be stabilised without modification even if quality improvement is possible through modification as opposed to an unacceptable soil which will not allow adequate stabilisation unless it is modified.

operation. If the relative cost of cement is high and significant cement saving is possible through modification, then in general it will be economically beneficial to modify such a soil to minimise its cement content. However if labour costs are high then it may be preferable to accept a high cement content and not modify the soil. Each case should be judged on its own merits.

For large-scale block production, involving considerable capital expenditure, then a full laboratory analysis including soil grading, plasticity and chemical composition may be justified. This type of test programme is not feasible without a well equipped, dedicated soil testing laboratory. (These are usually available through the government department dealing with road building). In this case several soil samples considered suitable from the field test selection process would be sent away to a soil laboratory to be tested, either so that the best can be identified or so that an optimum soil-blending formula can be devised. Even after full laboratory soil testing, trial block production testing must be carried out with the modified soil and local on-site laboratory testing is desirable to monitor the soil used throughout the project.

The above argument assumes that the final properties required of the soil-cement blocks are known. This is frequently not the case and deserves a brief consideration. Numerous standards have been developed for fired clay products and concrete blocks, especially in the developed world. However in general building material standards are much less advanced in under-developed countries and in the case of soil-cement blocks frequently non-existent. One draft specification for stabilised soil building blocks backed by the United Nations Commission for Human Settlements (UNCHS) in Nairobi, Kenya 1990, was based on a report presented by the Building Research Establishment (Ref 14, Webb 1991). This specification requires that water absorbtion after 24 hours of soaking should not exceed 15% of the original mass and that the minimum unconfined wet compressive strength after 24 hours immersion should not be less than 1.5 MPa (N/mm^2). It further suggests that the wet compressive strength should not be greater than 50% of the dry compressive strength. This specification may be used as an initial base standard for simple single storey buildings constructed with soil-cement blocks in arid or semi-arid regions. However it might be as well to remember that, provided enough strength is present to allow the wall to be self-supporting, durability is the factor which governs the building's life. A wet strength of 1.5 Mpa may be sufficient to prevent building collapse but might be inadequate for reasonable durability in less arid regions. The field of building standards relating to stabilised-soil building blocks is one which requires a large amount of further work. The wide variation in climatic conditions throughout the world necessitates regional or national building standards rather than global ones. At present these standards do not exist and a degree of judgement must be used when deciding the final block properties required. It would seem that the above specification can be taken as the minimum acceptable standard but that for areas with high rainfall the wet strength requirement should be increased to 2.8 Mpa or an external render applied to the wall. In such conditions any economic analysis carried out to asses the viability of soil modification should include due consideration of the cost of this external render or lack of it.

It may now be seen that the soil testing programme should be tailored to the scale of the project and the available testing equipment or testing funds. Soil testing is a supplement to reduce the number of trial blocks which need to be produced. A thorough testing plan should identify soils which are likely to be suitable and disqualify the unsuitable ones. All the above scales of production should utilise the basic field tests to reduce the number of soils to be subsequently considered. The simple laboratory tests will then further simplify the selection and modification of the soil. Full-scale laboratory testing will provide more accurate values for the grading and plasticity of the tested soils, although this accuracy improvement should be very minor if the simple laboratory tests are carried out correctly. Full-scale testing will also provide information on the chemical composition of the soil. The chemical composition may reveal the presence of soluble salts, primarily sulphates, which can attack the hardened cement's calcium silicate hydrate matrix and possibly lead to a reduction in strength with time. This reduction in strength with time may take several years to become apparent and therefore cannot be tested practically by trial block production.

The following sections show how the soil tests given in Appendices A and B may be used to provide a coherent soil test plan without undue duplication. Not every test mentioned in the Appendices need be conducted in every case; a number of the tests are alternatives which may be used according to the equipment available or can be used as cross checks if required.

5.1 PRELIMINARY ON-SITE SOIL TESTING PLAN

The initial field tests should be conducted on-site to assess the gross suitability of the available soils, arranging them into one of the categories listed below. Fines in this categorisation refers to the combined silt and clay content it should be noted that a soil containing clay-free fines regardless of the quantity of fines should be reported as very low clay and considered unsuitable.

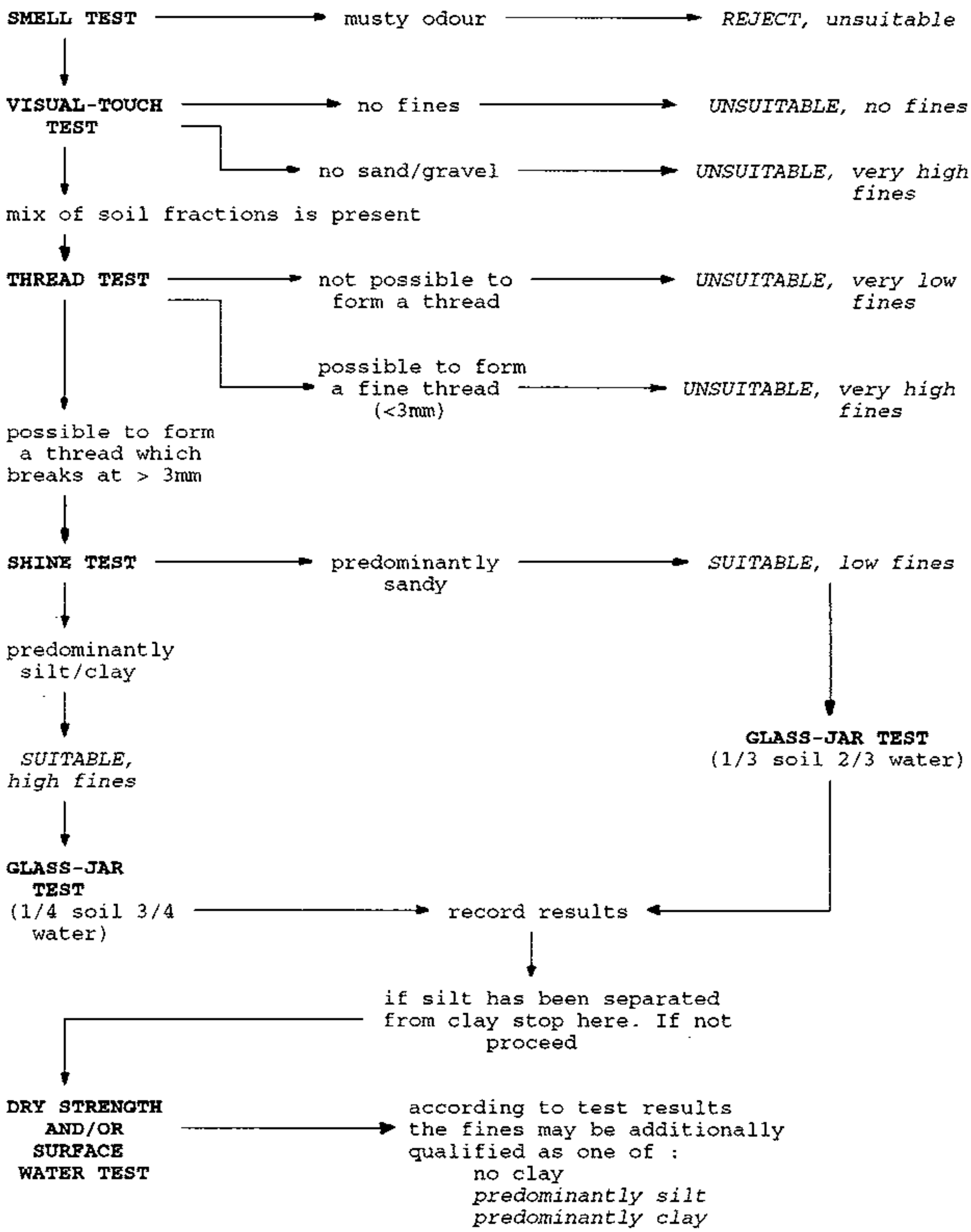
- *Organic..............Unsuitable.* REJECT.
- *Very low clay.........Unsuitable* unless clay added.
- *Very low/zero fines...Unsuitable* unless clay/silt added
- *Low fines............Suitable,* low cement content likely
- *High fines...........Suitable,* high cement content likely
- *Very high fines.......Unsuitable* unless sand added

Smell test. If musty smell is present record as organic and reject soil as unsuitable. If no smell proceed.

Visual-touch test.	To determine relative coarse/fine fraction present. If no fines are present record as unsuitable, no fines. If no sand/gravel present record as *unsuitable, very high fines* (proceed to shine and bite test to determine if fines are predominantly silt or clay for future reference). If a mixture of coarse and fine present proceed.
Thread test	To identify high plastic clay content and non-plastic soils. If a high plastic clay content present record as *unsuitable, very high fines.* If it is not possible to form a thread then non-plastic, record as *unsuitable, very low fines.* If neither proceed. (The ribbon test may be used as an alternative or for verification)
Shine test	To tentatively determine whether a combination soil is high or low fines. Predominantly sandy record as *suitable, low fines.* Predominantly silty or clayey record as *suitable, high fines.* Proceed with sedimentation test (use one third of a jar of soil if predominantly sandy and one quarter to one sixth if predominantly fines).
Sedimentation test	To give a rough analysis of relative sand/silt/clay composition. (Here the fines content may be further described by the recording the separate percentages of silt and clay not previously included in the above categorisation plan.) Less than fifty percent sand/gravel record as *unsuitable, very high fines.* Fifty to seventy percent sand record as *suitable, high fines.* Seventy to eighty percent sand record as *suitable, low fines.* Greater than eighty percent sand record as *unsuitable very low fines* (these are arbitrary boundaries and intended as a guide only). If no clay is present record as *unsuitable, very low clay.*
Dry strength and wet-shaking	These are additional mechanical tests on the fine soil fraction (< 0.425 mm.) and provide information on the clay content of the fines. These tests should be carried out if the glass-jar sedimentation test fails to discriminate silt from clay. If these tests show that no clay is present in the fines then the soil should be reported as *unsuitable, very low clay.*

Soils which are considered suitable from the on-site testing plan may then be more closely examined with the simple shrinkage box test and/or the following simple laboratory tests (dependant on the equipment and funds available). Such further testing will determine which soil is likely to produce the most acceptable blocks, remembering the points mentioned above in the consideration of suitable soils, section 3.4.

5.2 FIELD TESTING TREE TO ILLUSTRATE A COHERENT TEST PLAN



This field testing tree diagram illustrates one sequence in which the field tests may be carried out. This diagram does not include every possible field test but should illustrate that basic soil selection is possible if the tests are used coherently in a logical order.

5.3 LABORATORY TESTING PLANS

Laboratory tests will provide more precise detailed information on the soil's grading and plasticity. This information should be used to select the soil most likely to produce acceptable blocks based on the selection criteria given in sections 3.4 and 3.5. Laboratory test analysis for soils considered suitable on the basis of the above preliminary tests may be conducted using one of the following plans. Which plan is used depends on the resources available, Plan 1 requires accurate weighing equipment as the soil samples used for sedimentation and dry sieving are small. Plan 2 requires a moderately large supply of water for effective wet sieving. Plan 3 relies on representative soil samples being used. Other plans are of course possible.

If no single soil seems suitable or only barely suitable then a combination of two (or if justified more) soils may frequently produce a more successful material. For example a soil without fines may be improved (modified) by adding a suitable quantity of a clayey soil containing a high fines content. The grading information gained from the laboratory tests will enable the relative amounts of each soil type required to be provisionally calculated. Although the modified soil should be re-tested using the laboratory tests the modification process will be greatly simplified.

PLAN 1.

Sedimentation test (syphon)	Used to measure the clay fraction of the soil. The settled material may be subsequently dried and used in the dry sieve test.
Dry sieve test.	The settled material from above may be sieved dry to determine the gravel, sand and silt fractions.
Atterburg tests	Should be conducted using the original soil, suitably sieved, to determine the liquid/plastic limits and plasticity index.

PLAN 2.

Wet sieve test (fines retained)	Used to determine the gravel and sand fraction of the soil and to separate the silt/clay fraction for sedimentation.
Sedimentation test (syphon).	The material passing the 0.063 mm wet sieve may be separated into silt and clay fractions.
Atterburg tests	As above.

PLAN 3.

Wet sieve test (fines discarded)	Used to determine the gravel and sand fractions of the soil.
Sedimentation test (syphon)	A separate portion of the above sample is sedimented to determine the clay fraction. The silt fraction is found by adding the total measured soil percentages and taking this figure away from 100 %
Atterburg tests.	As above.

APPENDIX A : FIELD TESTS

Smell test

USE :	For determining the presence of organic material.
ACCURACY :	Medium to high.
TIME :	Fast.
LIMITATIONS :	This test does not determine the quantity of organic matter present.

EQUIPMENT: Minimal; small cooking stove or fire and a suitable pan.

METHOD: Take a representative sample of moist soil and smell it. If the soil smells musty then a significant quantity of organic matter is present (soil containing organic matter is unsuitable for building and should not be used). If a musty odour is not present, heat the soil in a pan and smell again. If there is now a musty odour then the soil again contains too much organic matter and should be discarded. If the soil does not smell musty at all then the soil is probably inorganic.

NOTE: Usually the top layer of soil will be organic but subsequent lower layers may be inorganic.

Visual-Touch test

USE:	For initial on-site examination of soil to determine the presence of gravel,sand, silt and clay.
ACCURACY:	Dependant on skill of tester.
TIME:	Fast.
LIMITATIONS:	Very difficult to tell silt from clay by a visual examination.

EQUIPMENT: None.

METHOD: Visual..Take a representative dry sample of soil. Breakdown any lumps or clods by rubbing between the fingers and examine to gain an idea of the proportion of different size particles. Particles larger than 2 mm are defined as gravel (BS1377) while those smaller than this but still visible to the naked eye form a continuum of coarse through medium to fine sand. The smallest grains visible to the naked eye are fine sand, approximately 0.06 mm. The conventional size boundaries are listed below. The dust which cannot be distinguished as single grains is a combination of silt and clay, normally called the fines.

Touch..The feel of the soil can also be an indicator of its basic components if rubbed both wet and dry. Sands are coarse particles which have a rough feel when rubbed between the fingers. They lack cohesion when wet, they do not stick together well. Dry silt has a similar but less pronounced feel to dry sand and shows limited signs of cohesion when wet. Dry clay usually forms hard but smooth clods. If these are broken down when dry the resulting powder has a smooth slippery feel. When wet, clay has a greasy or sticky feel and is very cohesive.

NOTE: This test is useful if it is used to gain a first broad idea of the soil constituents however unless the soil is a pure sand, silt or clay or the operator has considerable experience it is very difficult to assess the percentage composition. A no fines soil should be reported as *unsuitable, no fines*. Similarly a soil with little or no sand/gravel should be reported as *unsuitable, very high fines*. Further testing should be carried out if a mixture of sizes is observed.

British Standard and MIT definition of soil particle sizes:

Coarse gravel................	60 - 20 mm
Medium gravel................	20 - 6 mm
Fine gravel..................	6 - 2 mm
Coarse sand..................	2 - 0.6 mm
Medium sand..................	0.6 - 0.2 mm
Fine sand....................	0.2 - 0.06 mm
Silt.........................	0.06 - 0.002 mm
Clay.........................	> 0.002 mm

Thread test

USE: To test for the presence of a large quantity of plastic clay or pronounced lack of fines.
ACCURACY: Low.
TIME: Fast.
LIMITATIONS: Only gives a vague estimate as different clay types have different plasticity. Requires prior operator experience for successful interpretation.

EQUIPMENT: A smooth surface, a sheet of glass or similar.

METHOD: A small representative sample of moist, easily mouldable, soil should be formed into a cylinder about the same size as a thumb. This cylinder should then be lightly rolled with uniform pressure on a smooth flat surface by the outstretched fingers of one hand, forming a thread of soil (if it is not possible to form a thread report as *unsuitable, very low fines*). The thread will reduce in size until it breaks, either by snapping into shorter pieces or shearing along the length of the sample. The size at which the thread breaks gives an indication of the clay content. If the sample will easily form a 3 mm or lower diameter thread then there is probably a high plastic clay content. If the thread breaks at a larger diameter than 3 mm then there is either a moderate sand and silt fraction present or the clay is only slightly plastic. If the sample appears to have a high plastic clay content then it should be reported as unsuitable, very high fines.

NOTE: This is a simplified version of the Atterburg plastic limit. For the full test see the laboratory tests section.

Ribbon test

USE:	To test for the presence of a large quantity of plastic clay or pronounced lack of fines.
ACCURACY:	Low.
TIME:	Fast.
LIMITATIONS:	Only gives a vague estimate as different clay types have different plasticity. Requires prior operator experience for successful interpretation.

EQUIPMENT: none

METHOD: Take a representative sample of soil sufficient to form a roll about the diameter of the thumb but three times longer (A comfortable size to fit in the palm of the hand with the fingers rolled in to make a hollow fist). Wet this soil so that the sample is damp but not overly sticky. Hold the sample in the palm of the hand with the fingers rolled over and push the sample out from between the thumb and first finger, flattening it to form a ribbon 4 - 6 mm in thickness. Let the ribbon hang down from the hand, without supporting it, and see how long it gets before it breaks. Compare the length at which it breaks with the lengths given below.

.. 0 cm, no ribbon at all. This indicates that the soil contains very little or no clay and should be reported as *unsuitable, no clay*.

.. 4 - 10 cm, short ribbon. This indicates a soil containing a low to moderate quantity of clay and should be reported as provisionally suitable. The longer the ribbon then the larger the quantity of stabiliser which will be required for adequate stabilisation.

.. 25 cm and longer, long ribbon. This indicates a soil containing a high quantity of clay and should be reported as *unsuitable very high fines*. Such a soil would require an uneconomically high quantity of stabiliser for adequate durability.

NOTE: The lengths given above are not a set of rigid rules but should be treated as a set of guidelines. With experience of testing the local soils these lengths should be revised to improve the selection accuracy.

Shine test

USE: For determining the major soil components and identifying a silt or clay dominated soil.
ACCURACY: Low.
TIME: Very fast.
LIMITATIONS: This test determines which is the major soil component (sand, silt or clay). It does not determine the quantities present.

EQUIPMENT: Sharp knife (optional).

METHOD: Take a representative sample of soil. Moisten it and form into a ball. Cut the ball with sharp knife or polish a section of it with a fingernail. If the resulting surface is shiny the soil is predominantly clay. If the surface is dull and feels abrasive or harsh then the soil is predominantly sand or silt. Sand and silt may be distinguished by closely examining the surface. If the surface appears grainy then the soil is a sand. If grains cannot be seen the soil is silty.

Bite test

USE: For differentiating between silt and clay on-site.
ACCURACY: Dependant on skill of tester.
TIME: Very fast.
LIMITATIONS: Only useful for distinguishing on a presence/absence basis.

EQUIPMENT: None.

METHOD: Take a pinch of soil and lightly grind it between the front teeth. Any sand present will feel harsh or gritty and unpleasant. Silt will also feel gritty but much less unpleasant. Clay will feel smooth or flour-like.

Sedimentation test (glass jar)

USE: A simple test to give a rough numerical value to the percentage fraction of soil components.
ACCURACY: Medium to low.
TIME: Slow (up to 24 hours).
LIMITATIONS: The results from this test give an idea of the soil's component parts to a low accuracy. The low accuracy is due to the difficulty in discriminating the layer boundaries and the slow settling movement of these boundaries over time.

EQUIPMENT:
- 1 Wide transparent glass jar (> 65 mm diameter), straight sided and flat bottomed with a capacity greater than half a litre.
- 1 Bung for the glass jar (optional).
- 1 Stopwatch or clock.
- 1 Ruler long enough to measure the height of settled material.
- A supply of clean drinking water.

METHOD: A representative sample of soil is loosely placed in the glass jar up to one quarter of its depth for sandy soils or one quarter to one sixth of its depth for silty or clayey soils. Clear drinking water is placed into the jar to fill it almost to the top. The bung is then placed in the mouth of the jar and the jar left undisturbed until the soil is completely soaked with water. The jar is then shaken vigorously for one or two minutes and placed on a flat level surface to stand undisturbed for one hour. The jar is then reshaken for a further minute, replaced on the flat surface and the stopwatch started. The jar must now be left UNDISTURBED. After forty five minutes it should be possible to see a layer of sand settled at the bottom of the jar and a further layer of silt settled above. The cloudy suspension above the silt layer is the soil's clay content (If a pronounced clear layer is seen the soil has flocculated and should be treated with one of the chemical agents listed below). The clay settles out much more slowly than the sand or silt, settling at approximately 12 mm per hour. After a further twelve to twenty four hours the clay should also have settled. The different components can now be measured by measuring the height of the three layers. If the silt/clay boundary cannot be seen and the suspension has not flocculated then the experiment may be repeated using the timing system put forward by Norton (1986), the height of settled material is recorded after 1 minute, 30 minutes and 12 to 24 hours (depending on fineness of clay). The total depth of the sediment (not including the water remaining above) is taken as 100% of the soil. The height of each layer is then recorded as a percentage of the total depth. The three values then taken to be the sand, silt and clay content of the soil.

NOTE: This test has been in wide use for a long time but with some significantly different methods of application. The test is based upon Stoke's law of sedimentation which predicts the rate of settling for a spherical particle in free fall. This is only strictly valid for low concentrations of spherical particles. The jar test then has two sources of primary error in that the particle concentration is not low and the particles are not spherical, particularly so when considering the clay fraction, most clay particles being a plate-like shape. Results from this test should always be treated with caution.

This test frequently contains instructions to add a pinch of salt as a deflocculent. This is incorrect: salt should not be added except under special circumstances (for details see above, section 4.3.3 Sedimentation tests). Suitable deflocculents / dispersants are listed overleaf.

Deflocculents after Head (Ref 8, Head 1980):

sodium bicarbonate	starch
sodium carbonate	sodium silicate
sodium hexametaphosphate	tannic acid
sodium tetraphosphate	sodium hydroxide
sodium oxalate	trisodium phosphate | for laterites.
sodium tripolyphosphate	tetrasodium phosphate | for laterites.
sodium polyphosphate	

also: gum arabic (Ref 6, United Nations 1964)

Dry strength test

USE:	Additional test to estimate whether silt or clay predominate in the fines of a combination soil.
ACCURACY:	Low, dependant on operator judgement.
TIME:	Slow if sedimentation is used to prepare the sample, faster if dry sieved.
LIMITATIONS:	Only low accuracy without prior operator experience.

EQUIPMENT:

EITHER	1	Wide transparent glass jar (> 65 mm diameter), straight sided and capacity greater than half a litre.
	1	Syphon tube of suitable length approx 5 mm diameter.
	1	Separation disk with stem. A flat disk just smaller than the diameter of the glass jar attached to a suitable stem so that the disk may be lowered into the jar.
	1	Stopwatch to time thirty seconds.
	1	Wide dish to collect the syphoned liquid. Approx 150 mm diameter.
		A source of clean water (as clear as possible).
OR	1	0.06 mm. sieve and collector

SYPHONING METHOD: Place loose soil into the jar up to one quarter of its depth. Add water to nearly fill the jar, cover the mouth and shake vigorously. Leave to stand for one hour to allow the soil to soak.

Shake the jar vigorously for approx two minutes and stand on a solid flat surface. Time for thirty seconds from placing the jar on the flat surface.

Lower the separation disk quite quickly into the jar so that it covers (without disturbing) the sand settled after thirty seconds. Syphon off the liquid containing the remaining suspended matter into the wide dish. The easiest way of doing this is to tie one end of the syphon tube to the base of the separation disk's stem. This anchors the tube, preventing it from floating.

The particles will slowly settle out of the water in the wide dish leaving clear water. This water should then be decanted off, either by carefully pouring, without disturbing the settled material, or preferably by syphoning. Not all of the water should be removed like this as inevitably some material would be lost. The remaining water should be evaporated off.

DRY SIEVING METHOD: If a 0.06 mm sieve is available the silt and clay portion of the soil may be removed from the soil mass by dry sieving. A representative sample of soil should be dried and completely sieved through the 0.06 mm sieve. The material passing through the sieve should be collected and used for the test below. See section 4.3.2 for a discussion of sieving techniques.

The resulting material should then be well mixed with a little water to evenly distribute the particles and a representative sample should be formed into a 2 cm diameter ball. This ball should be soft but not sticky, (a dough-like consistency).

The ball should then be dried out either by gently heating or by leaving in the sun.

When dry the ball should be crushed between the first finger and thumb. The resistance of the ball to crushing gives an estimate of the type of fine predominating. If the ball falls apart when picked up then the soil either has a very low fines content or no clay and should be reported as *unsuitable, very low fines*. If the ball crushes easily the fines are very fine sand, inorganic silt or a combination of very fine sand, silt and a small quantity of clay. This reaction should be reported as *suitable, low fines*. If it crushes with moderate difficulty the fines are an organic clay, a silty clay or a sandy clay and should be reported as probably *suitable, high fines*. If the ball cannot be crushed or only with considerable difficulty the fines are an inorganic highly plastic clay and should be reported as probably *unsuitable, very high fines*.

Surface water test

USE:	Additional test to estimate whether silt or clay predominate in the fines of a combination soil.
ACCURACY:	Low to medium.
TIME:	Slow if sedimentation is used to prepare the sample, faster if dry sieved.
LIMITATIONS:	Requires careful observation.

EQUIPMENT: As for the dry strength test above.

METHOD: Follow the instructions for the dry strength test (above) to produce a soft 2 cm ball. The ball is then held in the palm of one hand and repeatedly jarred horizontally by striking against the other hand.

As the ball is jarred either a film of water may appear on the surface, characterised by a shiny appearance, or no change will occur. After noting the preceding reaction, squeeze the ball with the fingers of the other hand. Either the water will disappear from the surface, the mass hardening and eventually crumbling or the appearance will not change, the ball being deformed into a soft plastic mass.

Repeat the above shaking and squeezing steps several times to be sure of the reaction.

If water appears and disappears quickly, the ball hardening when squeezed then the fines are a very fine sand or an inorganic silt. Reported as *unsuitable, very low clay*.

If water appears and disappears slowly then the fines are a slightly plastic silt or a silt containing a small amount of clay. Reported as *suitable, low clay*.

If no water appears on shaking and the ball is deformed into a soft plastic mass on squeezing then the fines are predominantly clay. Reported as provisionally *unsuitable, very high clay*.

NOTE. If the sample is a silt containing some clay, water will appear on shaking but may only partially disappear on squeezing, the ball feeling slightly plastic. Reported as *suitable, high clay*.

APPENDIX B : LABORATORY TESTS

Dry Sieve Test

USE:	To separate grades of sand on a size basis and give a value for the total fines content (silt and clay) for low-cohesion soils.
ACCURACY:	High (providing the soil is sufficiently broken down).
TIME:	Medium/slow.
LIMITATIONS:	The results from this test usually give an accurate breakdown of the sand fractions but silt and clay are too fine to be easily separated by sieving. If the soil is not easily broken down into individual particles the wet sieve test should be used.

EQUIPMENT: Nesting sieves

6 mm	Coarse and medium gravel
2 mm	Fine gravel
0.6 mm	Coarse sand
0.2 mm	Medium sand
0.063 mm	Fine sand

Suitable sized collector to catch the combined silt and clay fraction passing the 0.063 mm sieve.
Mass measurement balance.

METHOD: The dry sieve test is very simple to conduct if suitable sized sieves are available or can be made. A large (2 Kg) representative sample of soil is taken and thoroughly dried either in a pan over a stove or by spreading the sample out and leaving it in strong direct sun. From the dry sample two accurately weighed sub samples of about 1 Kg are taken, (500 g is adequate if the soil is fine). The following procedure is then carried out on each and the results averaged. The sieves are stacked in order of decreasing size, the 6 mm sieve at the top of the stack and the collector at the bottom. The weighed soil sample is then broken down into individual particles either by hand or by light grinding in a pestle and mortar. Re-weigh the sample if any material is lost in grinding. Place the weighed sample in to the top sieve. The set of sieves is then shaken until no more material passes from one sieve to the next, this may take some time to complete, (15 minutes or more), as particles slightly larger than the sieve aperture size tend to jam in the holes and blind the sieve. If this occurs gentle brushing of the sieve with a soft brush will unblock these holes, but care must be taken not to force material through the holes as this would give a false value.

Once material transfer has stopped the soil particles lying on top of each sieve are carefully removed and weighed, remembering to brush the material from any blinded holes. The mass of the material on each sieve is converted to a percentage of the total mass hence giving a simple particle size analysis, but without distinguishing silt and clay. Soil loss during the experiment can be checked by comparing the initial mass with the sum of the mass of the separated fractions.

NOTE: This test may be carried out with the sediment from the syphon test (described below), the material in the collector should then be silt as the clay will have been removed but the smaller sample size requires more accurate mass measurement. If accurate mass measurement is available this will give a more reliable result than dry sieving as the clay fraction, which tends to adhere to larger particles when dry, will have been washed off.

Sedimentation test (syphon)

USE:	A more accurate version of the glass jar test enabling direct measurement of the clay fraction weight.
ACCURACY:	Medium to high.
TIME:	Slow.
LIMITATIONS:	The accuracy of the results depend on successfully separating the clay fraction (see comments above on flocculation).

EQUIPMENT:

- 1 Flat bottomed glass jar, approximately 65 mm internal diameter and 1 litre capacity (a rubber bung to close the end of the cylinder is useful but not essential).
- 1 Flat circular disk on a stem such that it may be lowered into the cylinder. The disk should be slightly smaller than the internal diameter of the cylinder with the stem 10 cm longer than the height of the cylinder.
- 1 Flexible rubber syphon tube to remove suspended material from the cylinder.
- 1 Stopwatch or clock.
- 1 Weighing balance accurate to at least 0.1 g preferably 0.01g.
- 1 Heat proof container to receive the syphoned suspension.
- A clean supply of water.

METHOD: Weigh out a representative 100 g sample of dry soil and place it in the cylinder. Add clean water to 200 mm, measuring the height from the internal cylinder base. Close the cylinder with the palm of one hand or a suitable sized rubber bung and shake it vigorously end over end to produce a uniform suspension of soil. This may take some time depending on the type of soil. If the soil does not appear to form a uniform suspension then leave it to soak for thirty minutes and reshake. Once a uniform suspension has been formed place the cylinder on a flat steady level surface and begin to time 20 minutes.

At the end of 20 minutes slowly lower the disk to cover the settled material, taking care not to disturb it. If the soil contains a high proportion of fines, it may not be possible to see the upper edge of the settled layer. If this is the case then repeat the experiment using a smaller soil sample. (The top layer of material is silt, if the disk is allowed to rest on the surface then some silt will be forced up around the edge of the disk. Any silt forced back into suspension will give a misleadingly high value for the clay fraction.) The remaining suspended material may now be syphoned off with the rubber syphon tube. The syphoning operation is more simple to perform if the tube is tied to the stem just above the upper face of the disk. This stops the tube from floating or curling.

The material syphoned off is then dried, weighed and recorded as the clay fraction. The purity of the dried clay fraction may be tested with the bite test above if silt contamination is suspected. The settled material should then be combined sand and silt, these should now be separated by sieving. The sieving may be done wet or dry. In this case, the soil having had the cohesive clay component removed, dry sieving is the more appropriate. The settled material should be dried and placed into the top of the set of sieves as described above for the dry sieve test. In this case the material passing the 0.063 mm sieve is the silt fraction.

NOTE: This test is also based on Stoke's law of sedimentation and hence open to the problems mentioned for the glass jar sedimentation test. In particular salt is not a suitable deflocculent, one of the reagents mentioned above (Glass-jar sedimentation test) should be used if required. Flocculation should always be avoided if possible as it results in significant "wipe down" of the clay fraction (see below) and frequently results in a semi-settled layer of combined silt and clay above the settled material, causing difficulty in determining the level for the disk.

The syphon test uses a less concentrated sample of soil than the glass jar test and hence is more accurate but it is still prone to "wipe down" whereby the larger soil particles carry the smaller particles down with them. These effects can be reduced by carrying out a second syphon test on the settled remains of the first test, subsequently combining the two clay fraction values to give a more accurate reading. However this does increase the time required for the test.

Wet sieve test

USE:	To separate sand from the fines, particularly for lateritic soils which are difficult to breakdown when dry and may contain clay trapped in particle fissures.
ACCURACY:	Medium to high.
TIME:	Slow, dependant on the drying time after wet washing.
LIMITATIONS:	Flushing the soil particles down through the set of sieves requires quite large quantities of water which must be subsequently dried off before weighing the sample. Care must be taken when handling this water to prevent loss.

EQUIPMENT: Nesting sieves

6 mm	Coarse and medium gravel
2 mm	Fine gravel
0.6 mm	Coarse sand
0.2 mm	Medium sand
0.063 mm	Fine sand

Two or more large sized collectors to catch the wash water carrying the combined silt and clay fraction passing the 0.063 mm sieve.
Mass measurement balance.
A clean supply of drinking water.

METHOD: A representative dry sample of soil is weighed out accurately, about 1 kg for fine soils and 2 kg for coarse soils. This sample is mixed in a suitable clean bowl with an excess of water and left to soak for 1 hour. If available a dispersant should be added to aid the particle separation. Suitable dispersants are listed above under the glass jar sedimentation test. After one hour the soil is remixed and poured into the nesting sieves making sure to rinse any soil residue into the sieves with more water. The soil is then washed through the sieves with more water until no further particle transfer occurs between sieves. This may be checked by judicious inspection. This will require a large quantity of water and hence the collector should be regularly checked and replaced when nearly full.

When washing has been completed the soil fractions on each sieve should be dried, weighed and recorded as a percentage of the initial mass.

The wash water should be left to stand undisturbed until clear. This clear water can then be removed by syphoning or carefully pouring off without allowing any material to be lost. The residue is then either dried, weighed and recorded as a percentage as above or further separated into silt and clay fractions by the Sedimentation test (syphon).

SIMPLIFIED METHOD: If the wash water is allowed to run to waste then the total fines content may be found by subtracting the combined collected masses from the initial mass. The clay fraction may then be found from a separate sedimentation test and the silt fraction would be assumed to be the difference between the combined sand and clay percentage and 100%.

Atterburg Limit tests

USE:	To provide an indication of the properties of the soil fraction finer than 0.425 mm.
ACCURACY:	Medium to high.
TIME:	Medium/slow.
LIMITATIONS:	Considerable difficulty may be experienced finding the plastic limit when the soil contains a low plasticity clay. Tests on the same sample may give different results if performed by different operators.

EQUIPMENT:

1	Curved dish approx 93 mm diameter, 27 mm deep at centre.
1	Grooving tool to form a 2 mm wide, minimum 8 mm deep groove with sides 60 degrees off horizontal, or a knife to cut the groove.
2	Water proof, air-tight containers one large enough to hold approx 250 g of soil the other large enough to hold approx 100 g of soil.
1	0.425 mm. sieve.
1	Flexible blade to mix the soil.
1	Smooth surface. eg. plate glass 200 x 200 mm.
1	3 mm diameter rod (optional).
1	Mass measurement balance.
	A supply of clean drinking water.

METHOD: Dry a representative sample of soil, grind it in a pestle and mortar to break up any agglomerations of particles and sieve it through the 0.425 mm sieve to give a sample of about 200 g. Place this sample into the larger air-tight container and seal it. The following two tests should be performed on this sieved sample.

Liquid Limit. Mix about 70 g of the soil sample with the drinking water to form a thick homogenous soil paste. The mixing operations should continue for about 10 minutes but if the soil contains a moderate to high quantity of clay then the mixing stages should be very thorough taking up to 30 or 40 minutes each. With the flexible blade, smooth this paste into the curved dish, taking care not to trap any air. The soil should be 8 mm deep at the centre of the dish and full height at the edge. Using the grooving tool (or a sharp knife), divide the paste in two across a diameter leaving a clean groove 2 mm wide with sides 60 degrees from the horizontal.

Solidly hold the dish level in one hand with the groove pointing away from the body. Gently tap the dish horizontally against the heel of the other hand by moving it 30-40 mm (keep the empty hand still). After 10 taps the groove should close so that the two portions of soil come into contact along the bottom of the groove over a continuous distance of 13 mm. If the groove closes before 10 taps then the soil is too wet. It should be removed from the dish and more dry soil mixed with it and the test repeated. If the groove does not close after 10 taps then the soil is too dry. It should be removed, mixed with more water and the test repeated.

When the groove just closes over 13 mm the soil is at its liquid limit, put the sample into a pre-weighed container, seal it and reweigh it. Then dry the sample and weigh it again The plastic limit is now found by calculating the mass of water in the sample as a percentage of the soils dry mass.

Plastic Limit. Take about 10 g of the sieved soil sample and mix it with water to form a thick paste which should be malleable but not sticky. Roll the soil into a ball with the hands until it begins to dry and crack slightly. Divide the ball into four roughly equal parts and follow the following procedure for each part.

Mould the soil into a cylinder about 6 mm diameter. Place it on the flat surface and roll it under the fingertips with an even light pressure to reduce its diameter to 3 mm (check with the 3 mm rod) after between five and ten back-and-forth movements, slightly more for heavy clays. It is important to maintain a uniform rolling pressure throughout (do not reduce the rolling pressure as the thread approaches 3mm). If the sample breaks into pieces by shearing longitudinally or laterally at 3 mm diameter it is at the plastic limit. If it breaks before 3 mm, slightly wet the sample and retest. If it does not break at 3 mm it is too dry. Roll the sample between the palms of the hands and retest. If the soil always breaks before 3 mm then it should be recorded as non plastic[6].

When the soil breaks at 3 mm, quickly gather the pieces together, place then into a pre-weighed air-tight container, seal the container and repeat the test with the next soil sample. When all samples have been tested weigh and record the sealed container's mass then dry the sample and reweigh. Calculate the percentage of water as a fraction of the dry weight. This percentage is the plastic limit.

Plasticity Index. The plasticity index is the numerical difference between the liquid and plastic limit recorded as the nearest whole number.

---

[6]. A non plastic soil may still be suitable for soil cement production provided that some clay is present. The plastic limit test is a standard reference and failure to produce a result does not automatically mean that the soil should be rejected. For soil with a low clay content or containing a clay of low plasticity considerable difficulty may be experienced in attaining a plastic limit despite the soil exhibiting some plasticity.

Shrinkage test

USE: To provide an indication of the cement content required for a given soil compacted with a low pressure moulding machine such as the Cinva Ram.

ACCURACY: Medium.

TIME: Slow (at least seven days drying time).

LIMITATIONS: Requires a large soil sample and mould. It may take seven days for the shrinkage to be complete. This test has been calibrated for use with particular presses and as such is not directly relevant to machines operating at different compaction pressures.

EQUIPMENT:
- rectangular mould box of internal dimensions 40x40x600 mm.
- 6 mm sieve.
- mixing container and mixing implements.
- a supply of clean water (drinking water).
- a ruler or tape measure.
- a lubricant, either silicone grease, mould release oil, used engine oil or grease.

METHOD: The internal length of the mould cavity is accurately measured and recorded. All of the internal mould faces are smeared with the available lubricant to reduce the tendency of the soil to adhere to the mould.

A representative damp soil sample is taken and sieved through the 6mm sieve. This soil is then thoroughly mixed with water until it has a wet pudding or porridge-like consistency (this should occur near the liquid limit, see above). The mould is then filled with this soil mixture, in three roughly equal layers. After the addition of each layer the mould box is tapped to remove any air trapped in the soil. When the final layer has been tapped the excess soil is removed from the top of the mould leaving a smooth flat soil surface. It is important that the soil does not extend beyond the internal edge of the mould wall as this will increase the soil drag as the sample dries.

The mould containing the soil sample is then placed in a shaded area to dry. Once the soil appears to be shrinking away from the box sides it may be moved into direct sunlight to speed the drying process. The mould should be protected from rain throughout the drying time.

When the drying is complete the length of the dry soil sample should be accurately measured and recorded. If the sample has cracked across its width and separated into several pieces these pieces should be pushed together and the combined length recorded. If the soil has hogged up out of the mould forming a crescent-shaped length, the length of both upper and lower faces should be measured and their average recorded as the dry length. Cracking indicates a soil containing a high sand/silt fraction while hogging indicates a high clay content.

The linear shrinkage on drying may then be found by subtracting the dry soil length from the length of the mould box. This shrinkage length may then be referred to the table given below after VITA for the low-pressure (2MPa) Cinva Ram machine and after Webb for the high-pressure (10MPa) Brepack machine (Ref 5, Webb 1988).

Measured Shrinkage (mm. per 600 mm)	Recommended* Cement percentage (for Cinva Ram)	Recommended* Cement percentage (for Brepack)
under 5	too difficult to handle when block making	
5 - 15	5.56	perhaps insufficient clay (see sect'4.3.5)
15 - 30	6.25	5.0
30 - 45	7.14	6.7
45 - 60	8.33	8.3
over 60	not suitable for use unless more sand is added	

* The Cinva Ram blocks are to meet a wet strength criteria of around 1MPa, while the Brepack blocks meet a criteria of around 2.8 MPa. If the same strength criteria were to be used the high-pressure Brepack blocks would probably require about 40% less cement that the low-pressure Cinva blocks.

APPENDIX C: BIBLIOGRAPHY

Publications dealing directly with soil-cement and testing procedures

1. Handbook For Building Homes of Earth, Department of Housing and Urban Development, Office of International Affairs, Washington, D.C. 20410 (undated).

2. Stulz, R. (1983). Appropriate Building Materials, Intermediate Technology Publications Ltd. London, U.K / SKAT. St.Gallen, Switzerland.

3. Norton, J. (1986). Building With Earth: A Handbook, Intermediate Technology Publications Ltd. London, Great Britain.

4. International Labour Office. (1987) Small-scale Manufacture of Stabilised Soil Blocks, Technology Series, Technical Memorandum No.12, ILO Publications. Geneva, Switzerland.

5. Webb, D.J.T. (1988). Stabilised Soil Building Blocks, Ph.D. Thesis, University of Newcastle-upon-Tyne.

6. United Nations. (1964). Soil-Cement its Use in Building, United Nations Publication, Sales No.: 64.IV.6

7. Doat, P. et al. (1979). Construire en Terre, CRATerre, Paris.

Publications dealing with full laboratory soil testing methods:

8. Head, K.H. (1980). Manual of Soil Laboratory Testing, Pentech Press, London.

Publications dealing with the properties of clay:

9. Grimshaw, R.W. (1971). The Chemistry and Physics of Clays, Ernest Benn, London.

10. Worrall, W.E. (1968). Clays: Their Nature, Origin and General Properties, Maclaren, London.

General references:

11. Spence, R.J.S. & Cook,D.J. (1983). Building Materials in Developing Countries, Wiley, Chichester, England.

12. Ingles, O.G. & Metcalf, J.B. (1972). Soil Stabilization Principles and Practice, Butterworths, London.

13. Carter, M. & Bentley, S.P. (1991). Correlations of Soil Properties, Pentech Press, London.

14. Webb, D.J.T. (1991). The work of the Building Research Establishment, on lime stabilised soil blocks for Third World housing, Paper presented at the First International Seminar on Lime and Other Alternative Cements, 9-11 December 1991, Stoneleigh, UK, held by the Intermediate Technology Development Group, Rugby, UK.

DEVELOPMENT TECHNOLOGY UNIT

Working Paper No. 39

The Performance Testing of Treadle Pumps

October 1993

Development Technology Unit
Department of Engineering University of Warwick
Coventry CV4 7AL UK
Tel: +44 (0)203 523523 extn 2339 Fax: (0)203 418922 Telex: 311904 UNIVWK G

Working Paper No. 39

The Performance Testing of Treadle Pumps

by Dr. T.H. Thomas

October 1993

Abstract:
A good treadle pump satisfies many user criteria including some that can be tested under laboratory conditions. The paper describes a set of laboratory tests and suggests performance thresholds that should be reached in them. The tests relate eitther to efficiency or to ease of priming. Under the description of priming tests there is included an analysis of the effect of any 'dead volume' of trapped air upon the maximum suction head of a pump during priming and during steady operation.

List of Variables

A	cross-sectional area of cylinder or equivalent cross sectional area of diaphragm
f	pump cadence in cycles per second
g	gravitational constant = 9.81 m/s^2
H	total head experienced by pump = pressure rise thro' pump / ρg $= H_d + H_s$
H_f	friction head loss
H_d	delivery head = delivery port pressure / ρg
H_p	(unaided) priming suction head
H_s	suction head = inlet port negative pressure / ρg
H_{sm}	maximum pumping suction head
L_d	piston stroke
Q	flow through pump in m^3/s (1 m^3 = 1000 litres)
Q_i	ideal flow (if no leakage)
Q_l	leakage (including internal leakages) flow
W_f	friction power loss (mechanical)
W_{in}	input power at treadles
W_{out}	output power = $Q\ H\ \rho g$
η_{erg}	ergonomic efficiency
η_{flow}	output flow/(output flow + leakage flow) = $Q/(Q + Q_l)$
η_{head}	output head/(output head + head loss) = $H/(H + H_f)$
η_{hyd}	hydraulic efficiency = $\eta_{head} \times \eta_{flow}$
η_{mech}	mechanical efficiency = $(W_{in} - W_f)/W_{in}$
η_{pump}	pump efficiency = output power/input power = $W_{out}/W_{in} = \eta_{hyd} \times \eta_{mech}$
ρ	density of water = 1000 kg/m^3

1. Introduction

Any good treadle pump must satisfy many criteria. It must be 'effective' or efficient, durable, portable, adaptable over a range of users and delivery heads, easy to clean and repair, easy to install and start. For the purpose of this paper we will restrict our interest to performance in the narrow sense of effectiveness in use, i.e. performance then can be measured in laboratory tests.

Treadle pumps are human operated. The amount of water that can be pumped depends on the pump's efficiency, on the total head and on the effort and physique of the operator. It is not practical to replace human operators for testing purposes by calibrated motors or engines, so any test procedure must accommodate the variability of a person as a power source. Indeed the extent to which the pump matches the operator (allowing him or her to deliver power with the least fatigue) is an important performance parameter.

Human-powered pumps are not used continually or very steadily. They start and stop. For test purposes we usually distinguish between two phases of use:

a) Start-up or 'priming' when a pump is first applied to a new site.

b) Steady pumping.

There is a transition from the first phase to the second which may take as little as 5 seconds or as much as 10 minutes. During this settling-down period leather pistons may swell to achieve better sealing, residual air may be washed out of pistons, lubricating oil may spread to reach rubbing parts. However the transition has ill-defined boundaries and it is normally adequate to simplify behaviour into just the two phases already mentioned.

Priming takes a certain time t_p which terminates when the pump output has reached (for a given effort) say 95% of its final or steady state value.

2. Performance Measures

There are many possible performance measures, some of a global nature and some describing the effectiveness of particular pump components. Common measures are:

Output flowrate - for a defined (typical?) operator at various total heads and various suction heads. A convenient representation is to plot flowrate Q versus total head H for each of several suction heads (H_s) spanning from 0 to (say) 8 meters. Clearly H must not be less than H_s. The weakness of this measure is the variability of operators. However by using an operator of defined physique (eg. health, age, sex and weight) in a defined duty (eg. a cycle of 10 minutes treadling followed by 5 minutes rest maintained for 1 hour), this variability can be limited. Output flowrate is an easy measure to interpret and especially useful when comparing different pumps tested by the same operator.

Suction head - this is the maximum depth from which the pump can suck water. There is a absolute limit of about 10 meters on this depth at 20°C at sea level (assuming atmospheric pressure is 1000 mbar) which diminishes by about 10% for each kilometer above sea level. In practice suction heads are usually much less than 10 meters. We need to distinguish between two solutions. H_p = *unaided priming suction head*, is the maximum depth from which some water can be raised in say 1 minute of treadling without the addition of any water from the top. H_{sm} = *maximum pumping suction head* is the maximum depth to which the intake of a steadily operating pump can be lowered before delivery flow ceases. Alternatively a *pumping suction head* may be defined as the depth at which output falls to say 50% of its no-suction-head value. *Pumping suction head* may be observed from the *output flowrate* characteristic; *priming suction head* cannot. A further measure connected with suction is t_{lop} = *loss of prime time*, the time that a pump can be left idle after use before the achievable suction falls from H_{sm} to H_p or more realistically falls from H_{sm} to ½ $(H_{sm} + H_p)$.

Friction loss - can be divided into fluid and mechanical components. H_f = *friction headloss* is a function of flow rate Q and varies approximately with Q^2.

W_F = *mechanical friction power loss* varies with pump cadence (treadling rate) and would be expressed in watts. It measures friction heat generated in bearings, inside ropes, in the rubbing of pistons against cylinder walls and in the take-up of slack in mechanisms. Both H_F and W_F affect efficiency as discussed below.

Efficiency in any device is the ratio of the performance actually obtainable to that 'ideally' obtainable. When energy is being passed along a chain of processes - for example from human muscle to lifted water - the efficiency of each process can be expressed as the useful energy out of it divided by the energy put in. There are several efficiencies of particular interest. The hardest to measure is

$$\eta_{erg} = \textit{ergonomic efficiency} = \frac{\text{energy given by the treadler}}{\text{energy the treadler ideally could have given}}$$

Obviously we must compare "like with like", so the two human outputs should be measured over the same time and correspond to the same level of human effort. A machine which is awkward to use, i.e. which doesn't make effective use of the treadler's muscles, will have a low ergonomic efficiency. An ergonomically ideal machine will have $\eta_{erg} = 1$. The basic justification for employing a treadle pump rather than a cheaper handpump is precisely because of its higher ergonomic efficiency, its use of leg muscles rather than arm muscles.

More straightforward for measurement is

$$\eta_{hyd} = \textit{hydraulic efficiency} = \underbrace{\frac{\text{output head}}{\text{output head and head loss}}}_{=\eta_{head}} \times \underbrace{\frac{\text{output flow}}{\text{output flow and flow loss}}}_{=\eta_{flow}}$$

Both = η_{head} and = η_{flow} vary with pump throughput. At low heads and high flows, η_{head} is sometimes too low. At high heads and low flows, the seriousness of any leakage increases and η_{flow} is sometimes too low.

$$\eta_{head} = \frac{H}{H + H_f} \qquad \eta_{flow} = \frac{Q}{Q + Q_l}$$

where H_f rises with Q^2 and leakage flow Q_l rises with H. The second component η_{flow} is sometimes called *volumetric efficiency* and is affected both by leakage *from* the pump and, more important, back leakage *within* the pump due to imperfect non-return valves.

Mechanical losses determine:

$$\eta_{mech} = \textit{mechanical efficiency} = \frac{\text{energy that reaches the water}}{\text{energy put into treadles}}$$

$$= \frac{W_{in} - W_f}{W_{in}} \text{ or } = \frac{W_{wat}}{W_{wat} + W_f}$$

3. Prediction, Measurement and Interpretation of Output and Efficiencies

We can visualise flowing through a pump as undergoing a series of transformations each with an efficiency less than than 1.0 (100%).

Potential human power	$\xrightarrow[\eta_{erg}]{}$	Power applied to treadles	$\xrightarrow[\eta_{mech}]{}$	Power applied to water	$\xrightarrow[\eta_{hyd}]{}$	Output (pumping) power

$$(\eta_{pump} = \eta_{mech} \times \eta_{head} \times \eta_{flow})$$

However our interest in "efficiency" is only secondary. Efficiencies tell how effective the individual transformations are and how much room there is for improvement. Users are only interested in output flow under particular circumstances of operator, head and so on, and how the relative output flows of rival pumps compare with their rival costs. In any human-operated device the 'energy' cost of operation is high (for equal energy outputs, in most countries human labour costs up to 50 times that of engine fuel), so efficiency is important. Unfortunately the situations that favour use of human energy are also those that require the capital cost of equipment to be very low. To some extent high efficiency and low cost are incompatible.

In the flow chart above the first efficiency shown is ergonomic efficiency. Although there is a large literature on this topic and specialised measurement techniques (eg. human oxygen consumption), for pump testing it is not usually practical to separately measure ergonomic efficiency. The design of most treadle pumps allow the operator to adjust her/his position on the treadles to get the most comfortable combination of stroke, cadence (cycle rate) and foot force. The scope for adjustment has limits, so that heavy operators in situations of very low lift or light operators in situations of high lift may not be able to find an optimum position.

In the absence of direct tests of ergonomic efficiency, some indirect tests of treadler 'comfort' can be made. The pump can be operated at a mid-range head (eg. 3 m or 4 m) with a large strong operator, a medium operator and a small weak one. The cadence and stroke they choose for sustained pumping should be noted. The medium operator should not be at either end of the range of possible foot positions along the treadle, nor should she/he be persistently hitting the treadle end stops. With a good pump these conditions should also not arise with the heavy or light operators either (at this medium head). No operator should find the handle position awkward or the treadle angles too steep for ankle comfort.

Repeating the trials with a minimum head (1.5 to 3 meters depending on pump type) and a maximum head (5 to 10 meters), the operator choosing new best positions, will show whether the 'medium' operator can maintain a similar cadence and stroke as before. A change in either of more than 50% indicates likely low ergonomic efficiency at one or other end of the head range. It is unlikely that the heaviest operator will be comfortable and efficient at the minimum head, or a child will be efficient at the maximum head.

In the absence of complex equipment, a simple guide to human effort is pulse rate. If, on two different occasions of sustained pumping trials, the same person has the same pulse rate, then his two efforts are likely to be similar. Therefore if one occasion (i.e. choice of pump and head) gives a higher output power than the other, one may assume the former has the higher overall efficiency (ergonomic and pump efficiencies combined).

The three component parts of pump efficiency are the mechanical, head and flow (or 'volumetric') efficiencies. These can be approximately measured in isolation as follows.

$$\eta_{mech},\ mechanical\ efficiency = \frac{W_{in} - W_f}{W_{in}}$$

and power W is force × speed. Since speed is the same for both useful forces and for friction forces.

$$\eta_{mech} = \frac{F_{in} - F_f}{F_{in}}$$

provided both input force and friction force are measured at the same place.

Input force F_{in} can be measured at any defined position along a treadle when that treadle is moved slowly, the pump being connected to a defined typical head. Friction force F_f is approximately the force at the same point to slowly move the treadle when the pump head is reduced to zero (inlet and outlet at the same height). η_{mech} measured in this way should exceed 0.95 in a new well-adjusted pump and still exceed 0.9 in a worn pump.

$\eta_{head} = H/(H + H_f)$ requires measurement of head, which is simple, and friction head loss which is not. Friction head loss for a given flow Q can be obtained by first measuring the reverse head H_r (i.e. the height the pump outlet pipe is held *lower* than its input) necessary to drive that flow through the pump.

$H_f = k_f H_r$ where k_f is at least 4 and typically between 5 and 7.

For a two-cylinder treadle pump, whether with piston or diaphragm, any delivery flow has passed through two valves in series in either one cylinder or the other. During the reverse-head test the total flow divides about equally between the two cylinders (it may be necessary to clamp the pistons during the test to keep them in their cylinders), whereas in normal use the flow alternates between cylinders. The reverse head test is therefore subjecting each valve to only half the peak flow it would normally experience - hence the correction factor of $k_f = 4$ ($=2^2$). The reverse head also uses steady flow, whereas treadle pumps produce a slightly pulsating flow. This difference raises the factor k_f from 4 to about 6. As H_f is proportional to Q^2 we should measure H_r at maximum normal flow, which occurs at full power and minimum normal head H_{min}. Using the resultant value $H_{r,\,max}$ and H_{min} will give a worst case head efficiency of

$$\eta_{head,\,min} = \frac{H_{min}}{H_{min} + k_f H_{r,\,max}}$$

which should not be lower than 0.5 (50%).

$$\eta_{flow} = Q/(Q + Q_l) = Q/Q_i$$

so we need to measure either Q_l (difficult) or to calculate the ideal flowrate $Q_i = Q + Q_l$. With a piston arrangement, Q_i = piston stroke × cylinder area × cadence × 2 = $2\ L_p\ A\ f$ in consistent units. This flow efficiency should not be below 0.9 (90%).

With a diaphragm it is necessary to calibrate the volume displacement for each stroke length, a calibration which may be significantly in error if the diaphragm distorts strongly with variation in water pressure. In fact, measurement of volumetric/flow efficiency is probably not worth undertaking with diaphragm pumps.

The measurement of overall pump efficiency η_{pump} ia impractical unless calibrated gymnastic equipment is available to measure the power output of the operator. If such equipment is available and its action is similar (ie. treadling) to that of the pump, it may be possible to relate operator pulse rate to operator power. In such cases using the pump at exactly the same operator power W_{in} as on the calibrated equipment will permit the calculation of

$$\eta_{pump} = \frac{\rho g\, Q\, H}{\text{operator power}}$$

Pump efficiency will generally be lowest at low heads (and hence high flows). A worst case efficiency of 50% might be just acceptable. Alternatively $\eta_{pump} = 0.5$ at full power can be used to define the minimum rated head H_{min}.

4. Start up, Suction and Priming

The measurement of (unaided) priming suction H_p, and maximum pumping suction head H_{sm} is fairly straight forward.

A dry pump can be operated with an open delivery port and a vertical suction pipe (of normal diameter) attached to its inlet port. Starting with 10 meters of suction (if available) the suction depth should be reduced until the pump delivers water. The suction should be reduced not faster than 1 meter every 2 minutes: this can be done either continuously or in half meter steps. When the suction head H_p at which delivery begins has been found, it should be checked that the pump will prime within say 1 minute from this depth even after the suction pipe is completely drained of water and left to dry for 30 minutes. Once the pump is running and kept running for at least 5 minutes at a suction of H_p or less, the suction point should be gradually lowered while treadling slowly until (at depth H_{sm}) the delivery flow stops. This test can be repeated several time, and the values of H_{sm} averaged, to give a more reliable measure.

Suggested values for H_p and H_{sm} are 2 meters and 6 meters. H_p is quite variable as it depends on treadling rate and can also be badly affected by leaks round pistons or through valves. A diaphragm pump should reach a better priming suction H_p than a piston pump.

An alternative approach to measuring priming and suction performance is to estimate them from indirect measurements. Failure to prime or failure to maintain prime occurs when the pumping flow rate is less than the leakage flowrate. This may be due to high leakage or low flowrate or both.

The suction side of a pump and the suction pipe are normally surrounded by air that will leak in through any fine cracks. During priming the piston-cylinder seal and the valves are also in air (not water as during normal pumping) and will therefore leak faster than when wetted. The simplest form of priming assistance is to somehow wet these components, for example by pouring water on top of the piston.

The flow efficiency of a pump is lowered by the presence of any air in the cylinders or diaphragm chambers, due to air being thousands of times more compressible than water.

Consider a cylinder that contains a 'dead' volume V_d of air at atmospheric pressure when the piston is in its lowest position. As the piston is raised the air volume expands and its pressure drops approximately according to the relation p. $V^{1.2}$ = constant. Only when the piston has moved a considerable distance will the pressure have dropped enough to be below the inlet pressure and hence open the inlet valve. Thereafter further piston movement will perform useful pumping, drawing air (and below it water) up the suction pipe. We can therefore calculate a flow efficiency which assumes perfect seals (no leaks) but expresses the effects of this residual air. This ideal flow efficiency depends upon the suction head (expressed as a fraction of the atmospheric head of 10 meters) and the ratio of volume swept by the piston V_s. So let us define two ratios, a normalised suction S and a cylinder ratio R.

R = dead volume / swept volume = V_d/V_s

Table Ideal volumetric efficiency of cylinder (as %)

	S = 0 (H_2 = 0	0.1 1	0.2 2	0.3 3	0.4 4	0.5 5	0.6 6	0.7 7	0.8 8	0.9 9 m)
R =0.0	100	100	100	100	100	100	100	100	100	100
0.2	100	98	94	93	89	84	77	65	44	0
0.4	100	96	92	86	79	69	54	31	0	0
0.6	100	94	88	79	68	53	31	0	0	0
0.8	100	93	84	72	58	37	08	0	0	0
1.0	100	91	80	65	47	22	0	0	0	0
1.5	100	86	69	48	20	0	0	0	0	0
2.0	100	82	59	31	0	0	0	0	0	0

During priming when the pump is dry, the ratio of dead air volume to swept volume can be quite high, over 2 for some diaphragm pumps and over 1 for piston pumps. These will therefore not self-prime for suctions over 3.5 meters suction and 5.5 meters suction respectively even with perfect seals. For good self priming R must be kept well below 1.

During steady use however, much of the dead volume at the bottom of cylinders/chambers will be occupied by water. The dead air volume V_D will be much reduced and R ratios of below 0.1 may be attainable.

Using the table above, and measuring the pump to ascertain the values of R when the pump is dry and again when it is working (but air can still be trapped above the delivery valve), it is possible to predict maximum values for priming suction H_p and pumping suction H_{sm}. Actual values will be less due to leakage, especially during start up.

Regardless of any leakage losses, an operational *volumetric efficiency* due to these compression effects of less than say 60% would be unacceptable. Such a pump would be "springy" and difficult to treadle.

5. Conclusion

Because it uses a human energy source of variable and unknown power, it is not normally practical to test the efficiency of a treadle pump. However a number of tests and measurements are possible to check that the ergonomic, mechanical and hydraulic efficiencies (defined in the paper) are tolerably high.

If it is the relative performance of two rival pumps, rather than the absolute performance of a single pump, that is required, then suitable tests are available. For example one such test in Zimbabwe in 1991, comparing treadle pumping with bucket-carrying, established that the former was three times as productive as the latter in a typical irrigation task.

The ease of priming of a treadle pump, and the limits of its ability to draw water from a source some meters below itself, can be readily tested and also roughly predicted from measurements of pump geometry. It is shown that a pump whose 'dead' air volume exceeds its piston swept volume can not achieve high suction.

Treadle pumps have a lower head limit below which their inefficiency becomes acceptably high. This limit is typically 2 meters. Although treadle pumps can be designed for high heads, ergonomic and size constraints make them inefficient to operate over a head range of more than about 5:1, and economics discourage their use for heads exceeding 10 meters.

Good performance depends on generously sized valves and passages (to reduce head losses), good seals and fast closing valves (to reduce internal and external leakage), correct gearing (to allow users to choose ergonomically efficient cadences and strokes) and smooth, well-lubricated linkages (to raise mechanical efficiency).

DEVELOPMENT
TECHNOLOGY
UNIT

D
T
U

Working Paper No. 40

Quasi-Static Compression Forming of Stabilised Soil-Cement Buildig Blocks

1993

D.E.M.Gooding

Development Technology Unit
Department of Engineering, University of Warwick
Coventry CV4 7AL UK
Tel: +44 (0) 1203 523122 Fax: (0) 1203 418922
email: dtu@eng.warwick.ac.uk

DTU. WORKING PAPER No. 40 November 1993

Quasi-static Compression Forming of Stabilised Soil-cement Building Blocks

by Dr.D.E.Gooding

ABSTRACT

This paper examines the quasi-static compression (slowly applied pressure) method of compacting stabilised soil-cement building blocks. It describes a self-contained piece of research which was conducted to enable the comparison of quasi-static compression with the alternative dynamic methods of soil compaction. It gives an initial over view of the process of soil stabilisation and outlines the roles which soil structure and block curing play in stabilisation. The alternative methods of block compaction are briefly described, followed by a discussion of the material factors which affect the compaction of stabilised soil. A number of simple theoretical models to describe the internal compaction mechanisms of quasi-static compaction are then given.

The results of an experimental investigation to asses the effect of double-sided compaction, mould wall roughness, mould wall taper and pressure cycling relative to the datum process of single-sided, single-cycle compaction are then discussed. This is followed by an experimental investigation to determine the relation between compaction pressure, cement content and seven day wet compressive strength. A formula relating cement content and compaction pressure to wet compressive strength is put forward as the best fit to the experimental data gathered. This formula is then used as the basis for a simple economic analysis of high and low pressure compaction machines.

CONTENTS

INTRODUCTION

The following paper has been produced as the result of an on-going research program to investigate the compaction process used in the production of soil-cement building blocks. It is concerned with the standard quasi-static compaction process. This is the form of compaction normally used in the field by machines such as the Cinva Ram and the Brepack.

The paper is organised into six main sections followed by four appendices containing a bibliography and experimental details. The first section gives an initial overview of the stabilised block technology and briefly describes the roles which various factors play in the final strength of the block. Section two briefly describes the alternative compaction mechanisms and puts this research in context as a reference base for future work to investigate impact and vibration compaction. The third section deals with the material factors which affect the compaction process. The fourth section gives a number of simple theoretical models to describe the compaction process. Section five examines moulding factors which affect the quasi-static compaction of soil-cement blocks. Mould taper, mould roughness, compaction pressure cycling and double sided compaction are examined relative to standard single-sided compaction (the datum process) by recording the pressure transmitted from the compacting soil to the mould walls with an LVDT-based pressure transducer. Section six examines the relationship between compaction pressure, cement content and seven day wet compressive strength. This relationship is then used in a simple economic analysis to asses the cost effectiveness of high and low pressure compaction machines.

1. STABILISED SOIL

Some form of soil covers virtually the whole land surface of the Earth. This soil is usually readily processed with simple hand tools into an easily mouldable material which possesses good compressive strength when dry. Given soil's widespread availability, it is not surprising that it was traditionally widely used as a building material.

The major drawback to building with soil in its natural condition is its susceptibility to water. A soil wall may be considered as a load bearing skeleton of silt and sand glued together by clay. This glue-like behaviour when dry is caused by micro-droplets of water which exist at clay particle interfaces. Clay particles are usually electrostatically charged as a result of surface ion substitution. The charge tightly bonds a thin adsorbed layer of water to the particle's surface. The bonding is sufficiently strong for some adsorbed water to remain even at oven drying temperatures (105-110°C). At the point of contact between two adjacent particles, a micro-droplet of water can exist where the two adsorbed water layers come into contact. These micro-droplets generate both surface and capillary tension forces which hold the clay particles together. However, when any significant quantity of water is absorbed into empty soil pores, the droplets increase in size and the capillary and surface tension forces reduce, causing the soil to quickly soften and subsequently swell. On repeated wetting and drying the outer surfaces of a soil wall expand and contract more quickly than the main body. In a comparatively short time this leads to cracking and spalling of the outer surfaces and low durability for the wall. Moreover, if the wall becomes saturated with water the compressive strength may fall sufficiently to allow complete collapse.

When a soil has been treated to reduce the effect of strength loss on water saturation to a low level, then it may be considered a permanent, durable building material and may be called stabilised soil.

1.1 THE PRINCIPLES AND PRACTICE OF SOIL STABILISATION

If a soil is to be used in any but the driest of climates, it should be stabilised against the weakening effect of water ingression if it is to be durable. There are three primary methods of stabilising a soil: by adding a chemical waterproofer to reduce the tendency of the soil to adsorb water, by adding a chemical binder to give a strength mechanism which persists even when the block is saturated, or by compressing the soil to increase its density, hence increasing its load bearing capacity and at the same time reducing its water permeability.

Under normal conditions the soil's density is increased by compaction to reduce the soil-pore void volume and hence its permeability. By reducing the permeability of the soil, the length of time for which a soil wall may be exposed to water without adverse effect will be extended. However, increased density will not stop the ultimate failure of a soil wall if it is allowed to become completely saturated. To maintain strength when completely saturated, the soil must have an additional strength mechanism.

The advantages of increased density complement those derived from the addition of chemical waterproofers and binders, such that it is normal practise to compact the soil block whenever an additional stabilising agent is to be employed. The reduced permeability reduces the speed of water ingress, while the reduction in soil-pore void volume results in a lower chemical requirement. Whether binder or waterproofer, the chemical additive extends throughout the soil-pore void structure. If the volume of the void structure is reduced, then the amount of stabiliser required to fill or bridge the voids is reduced and hence an increase in soil density will usually allow a reduction in stabiliser content for the same final block strength (see section six for more information on the pressure-cement trade off).

When adequately stabilised, a soil block should not collapse when saturated with water, even after several cycles of complete wetting and drying. A waterproofing agent such as bitumen cutback has two effects. It reduces the ability of water to wet individual soil grains and hence reduces the ingression of water. It also provides a secondary method of particle adhesion by weakly sticking particles together. A dry soil block which has been waterproofed with bitumen may be weaker than an unstabilised block as the adhesion power of bitumen is frequently less than that provided by the silt and clay fraction which it is supplementing. However, when subjected to water the unstabilised soil block will rapidly loose almost all of its strength while the waterproofed block will largely retain its lower strength.

A binding agent such as cement will also reduce the effect of water saturation. Again any compaction of the soil during production will reduce the permeability of the soil and reduce the soil pore-void volume. The addition of a chemical binding agent does not stop the ingression of water much beyond that provided by the increase in density. Instead the binder provides a secondary method of bonding the soil particles together which is independent of the soil's water content. In the case of cementitious binders, the stabiliser forms hard insoluble fibres throughout the soil pore-voids. These fibres then effectively form a rigid skeleton which continues to hold the soil particles together even when wet. The addition of cementitious binders to an appropriate soil will improve both the wet and dry strength of the final cured block. The following paper is centred on binder-type stabilisation and in particular the production of stabilised soil-cement blocks where the stabilising agent is ordinary portland cement. Other binders, such as lime, have been used to make successfully stabilised blocks; however cement is the most commonly used stabiliser and as such has been the subject of this research.

Regardless of the type of stabiliser used, the production process for stabilised-soil blocks is similar. Firstly the local area is surveyed to establish the location of a suitable soil type.[1] Having found a suitable soil or modified a less than suitable one, the soil is dug, sieved to remove any large particles (greater than 6mm) and break up any large lumps. It is then laid out to dry. A suitable quantity of stabiliser is mixed with the dry soil and a uniform mixture is produced. A suitable quantity of water is then mixed with the soil such that the resulting mix is at the optimum moisture content (see section 3.1) for compaction. This mixture is transferred to the block mould for compaction. After compaction the "green" block is ejected from the mould and transferred to a curing area, which should be flat, level and protected from rain. The block is left to cure before incorporation in a wall. The curing regime differs according to the type of stabilisation being used. The

---

[1]. The process of soil selection and the discussion of soil suitability is covered in an earlier paper, DTU WORKING PAPER No. 38 "Soil Testing for Soil-Cement Block Preparation."

curing requirements for cement stabilised blocks will be covered in section 1.3.

1.2 SOIL STRUCTURE

The soil structure plays a very important role in the stabilisation process. In general use the high cost of soil will dictate the stabilisation of natural local soil. Natural soil exists in layers of differing composition. The top most layer of soil is generally organic and hence unsuitable for stabilisation. The subsequent lower layers will normally be inorganic and contain differing fractions of gravel, sand, silt and clay, and may or may not be suitable for direct stabilisation. If the first inorganic layer is predominantly clayey it will be poor for stabilisation. However, if the second layer is more sandy, a soil more suitable for stabilisation may be produced by blending the two.

Any soil may be described by a particle grading curve and its plasticity or Atterburg limits. The particle grading curve details the proportions of gravel, sand, silt and clay present in the soil while the plasticity limits give information on the properties of the fine (silt and clay-size) components. The different size fractions play different roles within the stabilised block.

Particles of sand size and larger act as the main inert body of the block. These sand and gravel fractions should be present in correct proportions to allow the most dense packing arrangement. The theoretical optimum distribution of particle sizes is that given by the Fuller curve (Fig 1.2a).

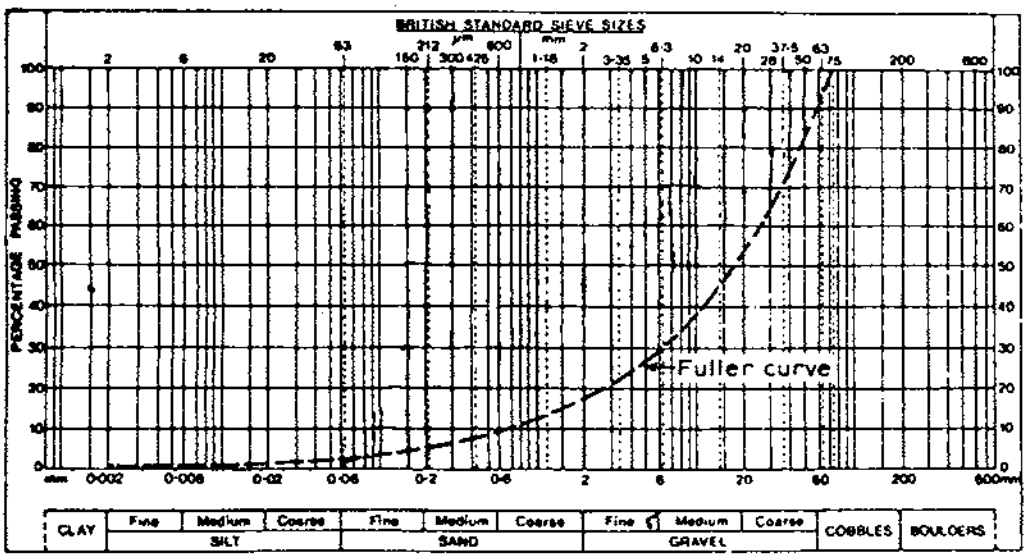


Figure 1.2a The Fuller Curve

The Fuller curve is based upon the assumption that all of the particles are spherical and that the largest particles just touch each other, while there are enough intermediate particles to fill the voids between the largest, but without holding them apart. The intermediate size particles are also similarly arranged with progressively finer particles filling the voids between larger ones. The Fuller distribution is an ideal model and never occurs naturally. However, a natural soil which has an even distribution of particle sizes, termed well-graded, is a good approximation and will ensure adequate packing.

The fine fraction of the soil may be considered as the active fraction. The coarse material is essentially inert, it does not change volume on wetting and does not play a significant role in particle bonding. The fine fraction is responsible for the dry cohesion of a soil. If this fraction is wetted it will expand. Clay consists of a large number of very small usually plate-like shaped particles. In a dry condition these particles pack closely together, held tightly in place by micro-droplets of water. On exposure to more water, capillary suction tends to draw water into the inter-particle fissures and separate the particles. This particle separation occurs simultaneously in three directions such that the clay expands.

The expansion of dry clay on wetting is the source of dimensional variations which can lead to cracking and spalling of a soil-cement block. For block durability, the extent to which the block expands and contracts on exposure to water should be minimised. A stabilised soil-cement block exhibits less dimensional variation than an equivalent unstabilised block. The force exerted by the expanding clay fraction must be resisted if the block is to remain intact over any significant length of time. In the case of soil-cement blocks this expansive force is largely resisted by the confining action of the insoluble cementitious matrix. Hence a large clay content soil will require a large quantity of cement to stabilise it. In order to minimise this expansion for a given water exposure level, and hence the amount of cement required, it is necessary to reduce the clay content of the block to a minimum.

However, the cohesive strength of damp clay is largely responsible for the green strength of the freshly formed block. Indeed a pure sand will exhibit negligible cohesion when dry and only slightly more when damp. It is most important to the economics of soil-cement block production that the fresh block can be ejected from the mould immediately after compaction rather than the cure-in-mould approach seen in conventional concrete work. To enable immediate ejection after compaction, the fresh block must have enough strength to allow at least careful handling without damage.

Hence it can be seen that the clay fraction is both a help and a hindrance to block production and a compromise must be reached. The extent of the compromise and the "optimum" clay content will be different for each particle grading and type of clay, montmorillonites for example exhibiting a much higher expansion on wetting than other types of clay. Moreover, the compaction pressure used to form the block also plays a role. At high compaction pressures the soil particles are forced into more intimate contact which increases the green strength of the block for a given clay content.

The sensitivity of the soil's Optimum Water Content (discussed in more detail below) also depends on the soil's structure. The enormous range in particle size from gravel (6mm) to clay (less than 0.002mm) results in a large difference in the specific surface area (SSA) of soils containing different proportions of different sized particles. When a soil is made up of predominantly fine material then the SSA of that soil is very large, when the soil contains larger fractions of more coarse material then the SSA is reduced. For a high SSA soil, a given change in water content will have a reduced effect on the compaction process as the area over which the water acts will be relatively large and result in only minor physical change. Conversely for a low SSA soil, the area over which the water acts is reduced and hence the physical effect on the soil is greater.

This effect is illustrated in figure 1.2b (Fig 1.2b). Dry density after compaction is plotted against moisture content at the time of compaction. Curve A shows the pattern for a well graded soil containing a range of particles from gravel to clay size, Curve B shows the pattern for a more narrowly graded soil containing particles from only fine sand to clay size. It can be easily seen that curve A has a more pronounced peak than curve B. It may also be noted that the density peak for curve B is shifted towards higher moisture contents and that the peak dry density is reduced. The increased optimum water content would be expected from the above discussion of SSA while the reduction in dry density would be expected from the discussion on particle grading.

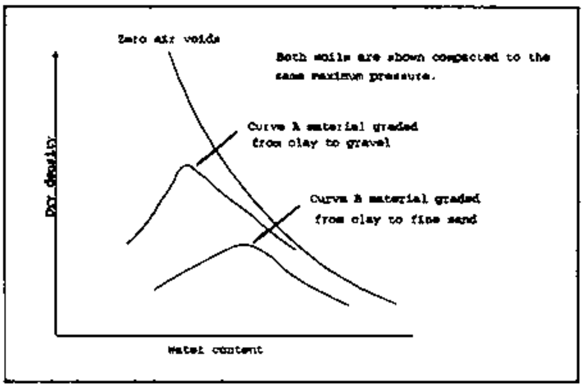


Figure 1.2b Dry density plotted against water content.

The factors affected by the structure of the soil may be summarised as follows:

GRADING The soil grading should be as close to the optimum Fuller curve as practical to enable the most dense packing of the soil particles. The more dense the packing of the soil particles then the lower the permeability of the soil and the higher its compressive strength. The larger the greatest size of particle the more care must be taken in controlling the moisture content.

PLASTICITY The soil should contain sufficient clay to allow adequate green strength on ejection from the mould. Too little clay and the breakage rate on ejection of the compacted block will be unacceptably high. Too much clay and the long term durability of the block will be adversely affected.

1.3 THE CURING PROCESS

The curing process described below refers to that required for blocks which have been stabilised with ordinary portland cement. The curing regime will be broadly similar for lime stabilised blocks but the length of time for curing should be doubled. For bitumen stabilised blocks, "curing" is the process of evaporation of the bitumen solvent and of the moisture content required for compaction. The bitumen curing process is primarily a drying operation while that for cement and lime is a period of time for the bulk of a chemical hydration reaction to occur.

In the case of bitumen stabilisation drying out of the block is desirable, provided that the drying is not sufficiently rapid to lead to warping of the block. For cement and lime stabilisation, drying out of the block will stop the cementitious hydration reaction and hence not allow the blocks to gain their full strength.

After ejection from the compaction mould the block should be carefully transported. The green blocks are weak until the chemical hydration reaction has occurred and any significant breakage rate will have an adverse effect on the economics of the project. The safest way to transport green blocks is to place them on individual boards and subsequently carrying the board to the curing site. The blocks may be placed onto and removed from the board by placing the palms of the hands flat against the largest sides of the block and squeezing the hands together just enough to grip the block to lift it.

The blocks should be taken to a flat level area which should be protected from direct sunlight and rain. Direct sunlight would cause the blocks to dry out too quickly, while rain would easily erode the fresh blocks at least until the cement has had time to hydrate. During the first four days the block should be kept damp to allow the chemical cement hydration reaction to occur. It is this hydration reaction which gives the soil-cement block its superior wet strength.

The exact mechanism by which a small content of cement may stabilise a large mass of soil is not fully understood. Ordinary Portland Cement is made up of 45% tricalcium silicate (C_3S)[2] and 27% dicalcium silicate (C_2S). In the presence of damp soil these

---

[2]. C and S represent Calcium and Silicon respectively, not carbon and sulphur. This is in keeping with most of the published concrete literature and is acceptable, allowing these simple equations to be given as illustrations instead of the more complicated fully balanced chemical equations.

components hydrate to form mono and di-calcium silicate hydrate gels (CSH and C_2SH, see equation below). These gels then slowly crystallise into an insoluble interlocking matrix throughout the soil voids binding the soil particles together. As the matrix is insoluble it gives a strength mechanism which works to restrain the softening and swelling of the unaffected soil, thereby dramatically reducing the weakening effect of water. The interlocking calcium silicate fibres may be seen when a cured soil cement sample is examined under an electron microscope. The hydration of the calcium silicate also results in the release of free lime (CH) according to the reaction:

$$C_3S + 2H = C_2SH + CH$$
$$C_2S + 2H = CSH + CH$$

The free lime then reacts further with the clay fraction (pozzolanic reaction) by the removal of silica from the clay minerals and subsequently forms more calcium silicate gel which also gradually crystallises.

2. BLOCK COMPACTION PROCESSES, THE ALTERNATIVES.

This paper examines the effects of various process and material factors on the production of soil-cement blocks by quasi-static pressure application. This work has been conducted as a self-contained piece of research but will be used as a reference datum for the future examination of alternative compaction mechanisms, the alternatives being dynamic force application by impact and vibration.

QUASI-STATIC COMPRESSION: Quasi-static or slowly applied compaction is the process used by the majority of soil-block making machines. The loose soil is compressed by slowly applying a large force. The magnitude of the pressure which is applied varies from machine to machine but is generally within the range of 1-8 MPa. The Cinva Ram is a well known low-pressure machine which uses force applied manually through a lever mechanism to produce a compaction pressure of about 2 MPa. The Brepack is an example of a high-pressure machine which applies between 8 and 10 MPa compaction. The Brepack uses a lever mechanism for the initial compaction and finishes with a manually operated hydraulic ram.

If a standard block's dimensions are assumed to be 290x140x100 mm then compaction pressures of 2 and 10 MPa equate to static loads of 8.3 and 41.4 metric tonnes. This is an appreciable loading for the structure of any machine to withstand.

DYNAMIC COMPACTION: Dynamic compaction is compaction by a mass which has a non-trivial velocity on impact, much like a hammer blow driving a nail into a piece of wood. Dynamic compaction requires far less massive machinery to generate large forces than quasi-static compaction does (to *push* a nail into a piece of wood requires a much greater static mass than that of the hammer). Dynamic compaction may also require less effort from the manual-machine operator.

A preliminary investigation into the efficiency of road compacting machines by the Transport and Road Research Laboratory (Williams and Maclean, 1950, Ref No.21) showed that slowly applied static loading may not be the most effective method of compaction for soil. It appears that dynamic compaction is more effective in terms of the depth of soil compacted.

Single impact and vibration are an extension of the same theme. Vibration may be thought of as a repeated light blow which, although requiring more complicated machinery to implement, does not involve the dropping of large and potentially dangerous weights.

3. MATERIAL FACTORS AFFECTING THE COMPACTION OF STABILISED-SOIL BLOCKS

3.1 MOISTURE CONTENT

Any soil placed in the mould for compaction should contain a known quantity of water. Earlier literature on the subject has suggested that there exists an optimum water content (OMC) at which the maximum density of the soil may be reached for a given compaction pressure. This is correct. By definition the resulting cured dry density will be highest for a block which has been compacted at its OMC. It should be forcefully noted that the OMC is not a parameter solely dependant on the soil, it varies considerably with the compaction pressure used to form the block. In general, as the compaction pressure increases so the OMC decreases. The content may be conveniently thought of as a lubricant which has adverse effects in excessive quantities. At higher compaction pressure the applied force is greater, the soil particles will move more readily and hence less lubricant (water) will be required.

The sensitivity of any particular soil to moisture depends on the soil's particular composition and has been discussed above with reference to specific surface area. Well-graded soils including a substantial fraction of large-size particles will be more sensitive to moisture content variation than soils with smaller fractions of large size material. The degree of sensitivity to change in moisture content is an important parameter to consider when producing blocks. If the soil material being used does contain a significant quantity of large size material then the control of the moisture content is more critical.

At the outset of this work a number of tests were conducted to establish the optimum moisture content for the soil used in the subsequent work (artificial *soil-A*). To this end a number of small cylinders were produced at varying water contents and at both 2 and 10 MPa. The soil used for the testing was well graded and the largest size fraction was coarse sand. This type of soil was found to have a low sensitivity to water (it contained a negligible proportion of gravel-size material), resulting in a flat moisture/density curve. However as a result of the low aspect ratio of the cylindrical mould used, the soil-strength required for successful demoulding was quite high. This emphasised the importance of green strength from an early stage in the experimental proceedings. The OMC at 10 MPa was found to be 8% and the cylinders became too weak to allow demoulding without the most careful and elaborate system at 10%! At lower moisture contents the green samples became progressively stronger. This follows readily from the strength mechanism described above for unstabilised soil.

Although the ultimate compressive strength of the block is important, so too is the green strength of the block. By lowering the moisture content at compaction, the green block produced will be stronger although the final cured strength will be lower. The extent to which the reduction in moisture content will reduce the final strength of the block depends on the nature of the soil. In general a more sensitive soil will show a larger drop in final strength as a result of the larger drop in compacted density. It is usually the case that compaction above[3] the optimum water content will result in a weak green compact. If this green compact is not sufficiently strong to allow handling on ejection then it is likely that the breakage rate both on ejection and subsequent transportation will be high.

Low green strength is one of the major problems for many block production systems. Indeed Dr. Lawson (1992, Ref No.22) found breakage rates to be as high as 50% for blocks produced on a Cinva Ram type machine in Nigeria. If an excessive green breakage rate is found then the water content should be reduced to increase the green strength. It should be remembered that any block which has been broken on ejection or during handling operations is lost. It may not be broken up and recompacted, as the cement hydration reaction will have progressed to such an extent that the amount of remaining unreacted cement would be too low for adequate stabilisation.

[3] With low clay content soils even compaction at the OMC may produce blocks with inadequate green strength.

The moisture content at the time of compaction also has an effect on the durability in terms of the blocks' permeability. It has been reported by CRAterre that compaction at moisture contents less than the optimum will result in a more permeable structure than compaction above optimum. A verification of this work will be carried out at The University of Warwick shortly. CRAterre use the concept of flocculated and dispersed microstructure to explain this phenomenon (Doat et al, 1979, Ref No.8). At low water contents the plate-shaped clay particles mutually attract each other. The outer edge of one plate electrostatically attracts the centre section of the neighbouring plates, leading to a flocculated clay structure (fig 3.1a). At high water contents, the surface charge of the clay plates is largely neutralised by the surrounding water dipoles and creates a pattern of mutually repelling particles or a dispersed structure (fig 3.1b). However at present the magnitude of this permeability change is not clear. In wet climates where water penetration is likely to be of importance then compaction slightly wet of the OMC may be appropriate to increase the blocks' resistance to water.

In summary, compaction at the OMC will produce the most dense blocks (by definition). Compaction wet of the OMC will produce blocks with a lower green strength on demoulding but possibly a lower permeability when cured. Compaction dry of the OMC will produce blocks with a higher green strength on demould and possibly a higher permeability.

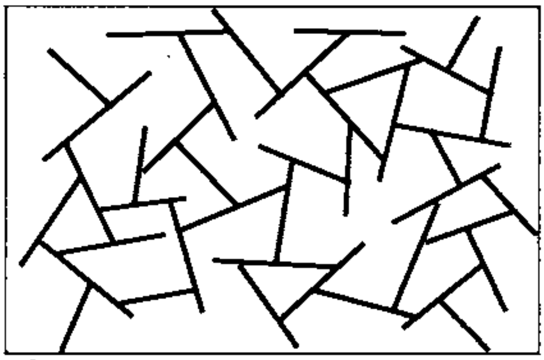


Figure 3.1a Flocculated

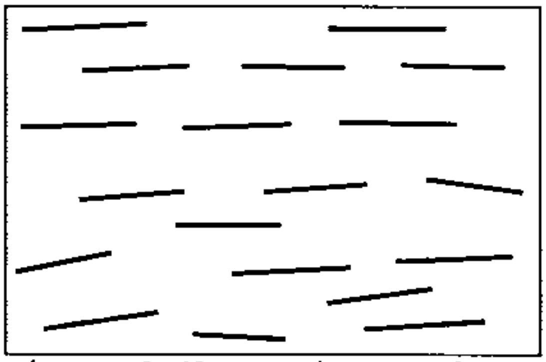


Figure 3.1b Dispersed

Compaction at the OMC is the best compromise. In dry climates and where low green strength has been seen to be a problem, the moisture content may be reduced. In wet climates were low green strength has not appeared as a problem then compaction at slightly increased moisture content may be considered. It should be remembered that the final cured strength of the block may be increased and the permeability reduced by increasing the cement content (although this may be costly) whereas the green strength may only be increased by improving the structure of the soil, compacting to higher pressure or reducing the compaction moisture content.

3.2 COMPACTION DELAY

The strength of the final cured block depends heavily on the adequate hydration of the cement. Cement is the most expensive ingredient in the production of stabilised soil blocks. Ordinary Portland Cement starts to react immediately on coming into contact with water. The reaction progresses quickly to begin with and progressively slows down over several weeks.

The hydration of the cement produces the insoluble calcium silicate hydrate skeleton which extends throughout the soil voids. This skeleton provides the restraining mechanism which gives soil cement its superior wet strength. The precipitation of these fibres begins as soon as the cement comes into contact with water. The fibres' effect is significantly reduced if they are deposited as discrete entities rather than as a continuous skeleton. Most of the fibres which form prior to the compaction of the soil cement mix will be broken during the pressing process and are then not as effective. The most effective precipitation occurs after the compaction process when the soil particles will remain undisturbed. As a result, the final strength of the stabilised block depends to a degree on the length of time for which the cement is exposed to water prior to compaction. Ingles and Metcalf (Ingles & Metcalf, 1972, Ref No.10) have reproduced a graph from G.West which indicates that over 50% of the final strength of cement stabilised soil may be lost by a delay of 2 hours (Fig 3.2a). Even after half an hour West indicates that between 30 and 40% of the strength is lost. A set of trials conducted at The University of Warwick on cylindrical test compacts has failed to convincingly reproduce such dramatic results. This may be because the short curing period used (seven days) was insufficient, not allowing the samples to cure for long enough to develop sufficient strength for adequate testing in the compression-test equipment available. However, a general trend was observed confirming some loss in (seven day) strength with increased delay between water addition and compaction. A 2 hour delay in compaction produced a strength loss of about 20%.

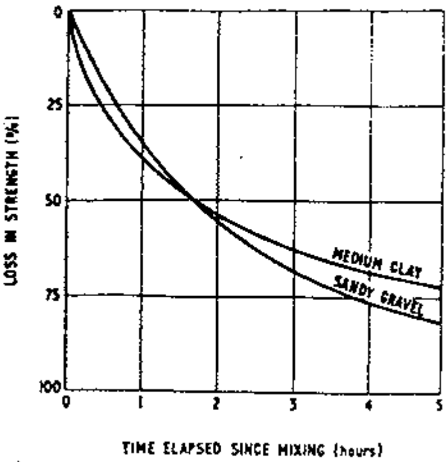


Figure 3.2a Cured strength loss due to compaction delay

4. MODELS TO DESCRIBE THE INTERNAL COMPACTION MECHANISM FOR COMPRESSED BLOCKS PRODUCED BY QUASI-STATIC COMPRESSION.

A number of simple models were initially postulated to describe the internal compaction mechanism; a simple hydrostatic fluid model, a pipe flow model, a solid model (based on Poisson's Ratio) and an elasto-plastic band compaction model.

4.1 Simple Hydrostatic Fluid Model.

The simplest model which might be used to describe the compaction process is the hydrostatic fluid model. This model assumes that the soil behaves like a contained fluid, namely that the pressure within the soil is the same in all lateral directions and increases in a downward vertical direction only as a result of the overburden pressure (weight of soil above the layer in question). This overburden pressure is insignificant compared to the external pressures applied to a block during moulding.

This model predicts that if a compaction pressure of 10MPa is applied to the top surface of the mould, both the mould walls and the base of the mould should also experience a transmitted pressure of 10 MPa. It further predicts that there will be no shear force between the soil and the mould walls.

4.2 Pipe Flow Model.

This model assumes that the soil is behaving like a viscous fluid flowing through a pipe. A viscous fluid may support a shear force while it is flowing. The more viscous the fluid, the longer it will take to flow and the larger the shear force it can maintain. This model predicts that if a pressure of 10 MPa is applied to the top surface of the mould the soil will "flow" downwards towards the bottom of the mould away from the source of high pressure. As some of the applied force is resisted by the mould wall shear force, the force on the bottom of the mould is reduced. As the distance from the compacting piston increases so the area for shear with the mould wall increases. As a result, progressively more force is transferred to the mould walls as a shear force or friction. This effectively reduces the driving force or pressure. The reduced driving pressure then results in a lower lateral pressure than that predicted by the simple hydrostatic model.

The above model is based on fluid pipe flow; flow through an open ended pipe. In the case of soil compaction one end of the mould is fixed and soil is forced towards it by the movement of the opposite end. According to fluid theory, when the velocity of flow reduces to zero, hydrostatic conditions should again prevail as a stationary fluid cannot support a shear force. As the flow of soil may be expected to stop when the compacting piston ceases to move this zero soil flow should result in equalised hydrostatic conditions, the mould wall pressure should equalise to the simple hydrostatic state mentioned above. If the mould wall pressure does not equalise then either the soil has become so viscous that its rate of flow is too slow to show this equalisation in the time available (typically 30 sec for block production) or that the soil may not be described as a fluid by the end of compaction. If the mould wall pressure does not equalise then the soil must be resisting internal shear forces, behaving as a solid.

4.3 Solid Model (Poisson's Ratio)

By applying a pressure to the top surface of a solid object, vertical strain is induced in the medium this results in a lateral stress which in turn causes lateral strain. The ratio of vertical compressive strain to lateral tensile strain is defined as the Poisson's Ratio of the medium and is constant within the elastic deformation regime. Typically, for most metals, Poisson's Ratio is around 0.3 (for a fluid it would be 0.5).

Poisson's Ratio is normally used to describe the deformation of unconfined solids when subjected to compressive or tensile stress. A simple unconfined example would be the compression of a steel cube. In such a simple case the Poisson's Ratio is relatively simple to define, the three axial strains being simple to measure and uncomplicated by constraint interference. In the case of soil block compaction this is not the case. Lateral strain is less than it would be when unconfined, as the movement is restrained by the mould walls. The applied stress results in a vertical strain, this producing a lateral stress resulting in a lateral strain. This lateral strain acts on the mould walls, as a result the mould walls deflect. The resistance of the mould walls to the soil deflection decreases the amount of lateral soil strain which can take place.

The mould walls restraining influence further complicates the model as the wall is not completely rigid. The wall elastically deflects as a result of the stress within the soil. The amount of deflection depends both on the stiffness of the wall and the soil stress acting on it. As the wall deflects it provides more resistance to movement as its effective rigidity increases, also as the soil expands so the soil stress is reduced. There should then exist a balanced condition where the restraining force exerted by the wall is equal to the residual soil stress. What effect this residual soil stress will have is unclear. The walls' restraining force may be considered as producing a lateral compressive force which in turn would produce a vertical stress opposing the initial compaction force.

The compaction may be described by this solid model provided that the soil's Poisson Ratio is assumed to change as the compaction cycle progresses. During the initial compaction stage (say up to 10% of the final compaction pressure) a large block height reduction takes place, typically 80-90% of the total height reduction, with almost no measurable lateral strain. This indicates largely plastic flow; on the removal of the compacting force only limited recovery takes place (typically 1-3 mm, 1/80th of the total deflection). As the block height reduces further the amount of relaxation expansion increases.

4.4 Elasto-plastic Band Compaction Model.

This model assumes that the compaction proceeds over a set of surfaces arranged roughly parallel with the moving compaction piston. As the compaction force is applied, so the top layer of soil plastically and, to a lesser extent, elastically compresses. As the soil plastically compresses it becomes stiffer and passes more of the applied force on to lower layers which are thus compressing at a slower rate. On decompression the plastic deformation is not recovered but the elastic compression reverses resulting in a degree of material expansion.

5. MOULDING FACTORS AFFECTING THE COMPACTION OF SOIL-CEMENT BLOCKS

Blocks are commonly moulded in a single cycle by moving a piston down into a parallel sided mould containing soil. The mould walls start with some roughness due to machining during manufacture, and this may increase due to soil abrasion during use. This process has been taken as a datum ("standard compaction"). Research was undertaken to better understand the standard compaction process and to investigate a number of variants to it. The pressure transmitted through the soil to the bottom of the mould was taken as a key variable that should correlate strongly with block density and subsequent cured block strength.

The following section examines the effect of the following moulding parameters:

Double Sided Compaction (instead of single sided)
Mould Wall Friction (high verses low)
Mould Wall Taper (instead of parallel-sided mould)
Pressure Cycling (instead of simple compression)

The effects of the changes in moulding parameters were observed by measuring the pressure transmitted to the mould wall through the compacting soil. The transmitted pressure was recorded by placing an LVDT-based pressure transducer (designed in-house) in seven separate locations in the mould wall and mould piston. The compacting pressure was applied in discrete increments up 2MPa or 10MPa. At each increase in load the block height, mould wall deflection and transmitted pressure were recorded. Details of the experimental method and the experimental instrumentation are included in appendices C and D.

The initial testing was conducted on blocks made without the use of cement such that the material could be reused and the testing procedure would be less time dependant. Moreover, without the addition of cement the soil mix has a higher internal friction[4] and therefore amplifies the effect of the changes in pressing parameters. It has been assumed that although the magnitude of the variation in the observed parameters will be different for cement mixes, the pattern of change will be similar. Any variation apparent in a non-cement mix is likely to be present in a mix containing cement, but to a lesser degree. Following the initial set of tests two further set of blocks were produced with the addition of cement, one set by the datum process and one by double-sided compaction. These soil-cement blocks were allowed to cure for seven days and then tested for wet compressive strength to asses the actual improvement in strength resulting from double-sided compaction.

All of the blocks produced were made from an artificial soil (*Soil-A*). This soil has been carefully tested both to BS1377 and by the methods given in the Development Technology Unit Working Paper (DTU) No.38 "Soil Testing for Soil-Cement Block Preparation". The soil is composed of Kaolin grade E powder and a poor quality building sand (high fines content). It has a low plasticity index and is well graded with particles from clay to coarse sand size. This soil has been used to allow repeatable experimentation for the duration of this work and if required the soil may be reconstructed at a later date. By using an artificial soil the large variation in properties of natural soil can be minimised. The details of this soil are included in appendix B.

5.1 SINGLE-SIDED COMPACTION - THE DATUM PROCESS

The first case which was examined was that for single-sided compaction. This is used as the datum against which to compare the other cases and as such it is examined in detail. Figure 5.1a shows the pressure variation as recorded by the LVDT transducers[5]. The pressure applied to the top face of the block, via the piston, was increased in small equal steps so as to achieve a uniform increase over time. It was then reduced at the same rate back to zero. After each step pressure and displacement readings were made, the applied pressure being held constant for long enough for these readings to stabilise (quasi-

---

4 The fine cement particles appear to act like a dry lubricant during compaction. Without cement the soil mix displays higher internal friction.

5 The detailed description of the performance, calibration and location of the LVDT transducers is given in appendix D

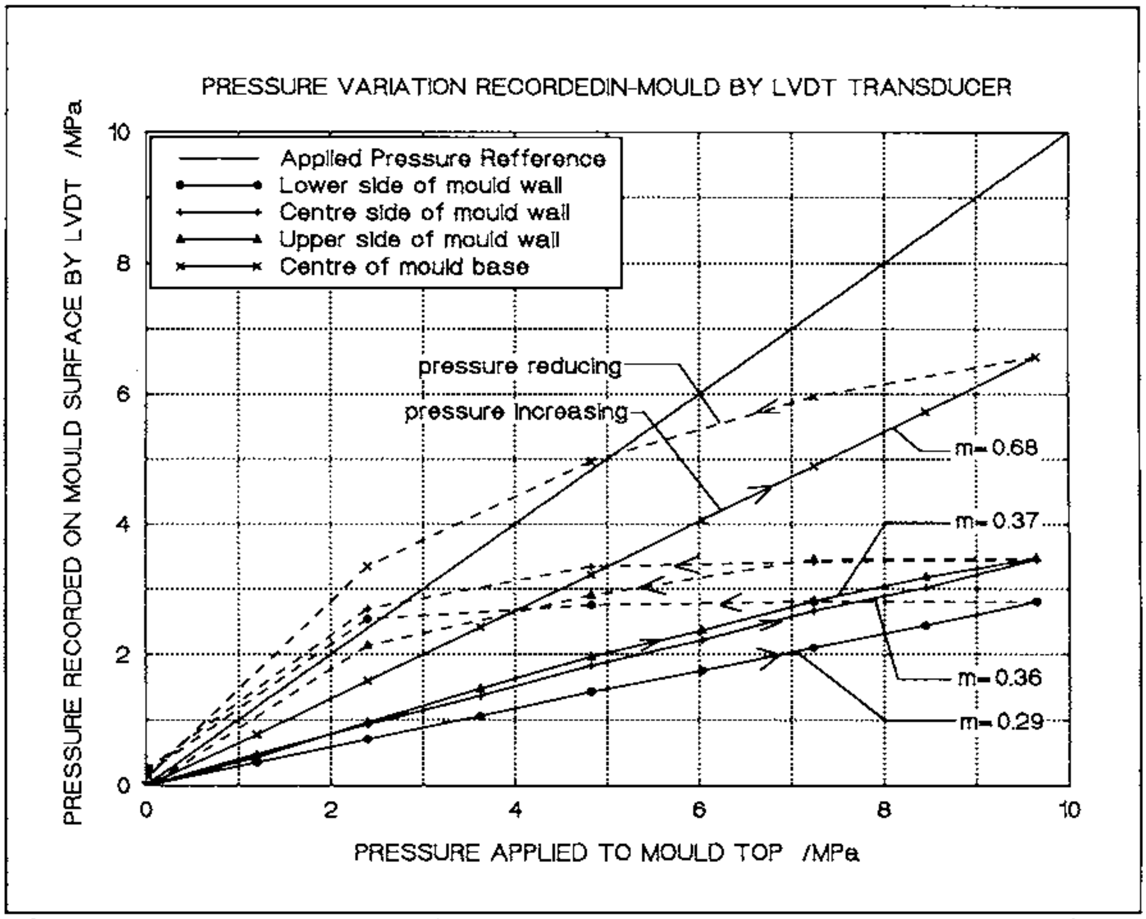


Figure 5.1a Compaction pressure plotted against transmitted pressure

static process). Pressure recorded on the block wall is plotted against pressure on the mould top (applied pressure calculated from the applied force and the known area of the mould surface). Only one transducer was initially used and hence the traces in figure 5.1a are actually the amalgamation of six separate block pressings, each trace being the average of two presses.

The maximum pressure readings agreed surprisingly well between pairs of blocks, averaging an agreement within ± 0.25 MPa (± 4% for base pressure). The applied pressure plot has been included in the figures for ease of comparison and plots as a straight line with a maximum of 9.66MPa (40 tonnes applied to the largest block face). In general the internal pressure throughout the mould increases linearly with the applied pressure but with differing rates of gain around the block (Table 5.1).

The transducer on the mould base recorded the highest pressure gain of 68%, rising to a maximum of 6.7 MPa compared to the applied 9.7 MPa, a loss of 3 MPa.

The transducers located at the upper and central regions of the mould-side wall both gave recorded maximum pressures of 3.5MPa (3.5 for the upper region and 3.48MPa for the central region i.e gain = 37% and 36% respectively). The pressure distribution along the length of the block at mid-height was also recorded. The pressure recorded at one third the length along the mould side wall was also 3.5 MPa (gain = 37%), while the pressure recorded at the end of the mould wall was 3.9 MPa (gain = 40%), slightly higher than at the centre.

The lower centre side wall transducer recorded a maximum pressure of 2.8MPa.

Table 5.1 Average Transmitted Pressure Recorded by LVDTs

Location of LVDT	Average Max Pressure /MPa	95% Confidence bounds	Pressure Gain %
Base	6.67	6.40..6.94	68
Upper Side	3.50	N/A	37
Centre Side	3.48	3.14..3.83	36
Lower Side	2.80	2.75..2.86	29

If the increase in pressure in the upper region of the mould wall (Fig 5.1a) is examined, it can be seen that this rise decreases slightly with increasing applied pressure and may be tending towards a maximum. This trend is not apparent for the remaining traces.

This pattern does not fit any obvious simple model and the only firm conclusion which may be drawn is that the soil condition is far from a simple hydrostatic fluid model.

The pattern of quasi-static pressure reduction was also recorded. The mould base pressure begins to drop off as soon as the applied pressure is reduced and continues to reduce at an increasing rate. At approximately 5MPa the base pressure is the same as the applied pressure. Below 5MPa the base pressure continues to decrease but remains higher than the reducing applied pressure.

The mould wall pressures also fall back as log curves. There is a lag period between when the applied pressure is reduced and when the mould wall pressures begin to drop significantly. This lag is least for the upper regions of the mould and increases for the central and lower regions respectively.

The material inside the mould under full (9.7MPa) compression will have undergone both plastic and elastic deformation. Large plastic deformation is evident from the significant volume reduction, typically 1.5:1. Elastic deformation is also apparent, although less pronounced, by the increase in block height as the applied load is removed (see Fig 5.1b, block height during compaction).

The pattern of mould pressure fall may indicate a pattern of vertical and horizontal stress reduction. As the applied pressure is reduced so those regions of the block nearest to the moving piston begin to decompress. In the upper regions of the block this decompression takes place initially in the vertical direction, the direction of movement. The resulting elastic expansion of the material is largely vertical as the mould maintains a lateral constraint. The vertical expansion reduces the lateral stress which is being applied to the mould-side walls, according to a poisson-type relation. As the upper lateral stress reduces so does the frictional shear stress with the mould wall. The reduction in mould wall shear stress effectively reduces the vertical constraining force, allowing the lower regions of the block to decompress. As the applied pressure is reduced further so the region of elastic expansion moves downward.

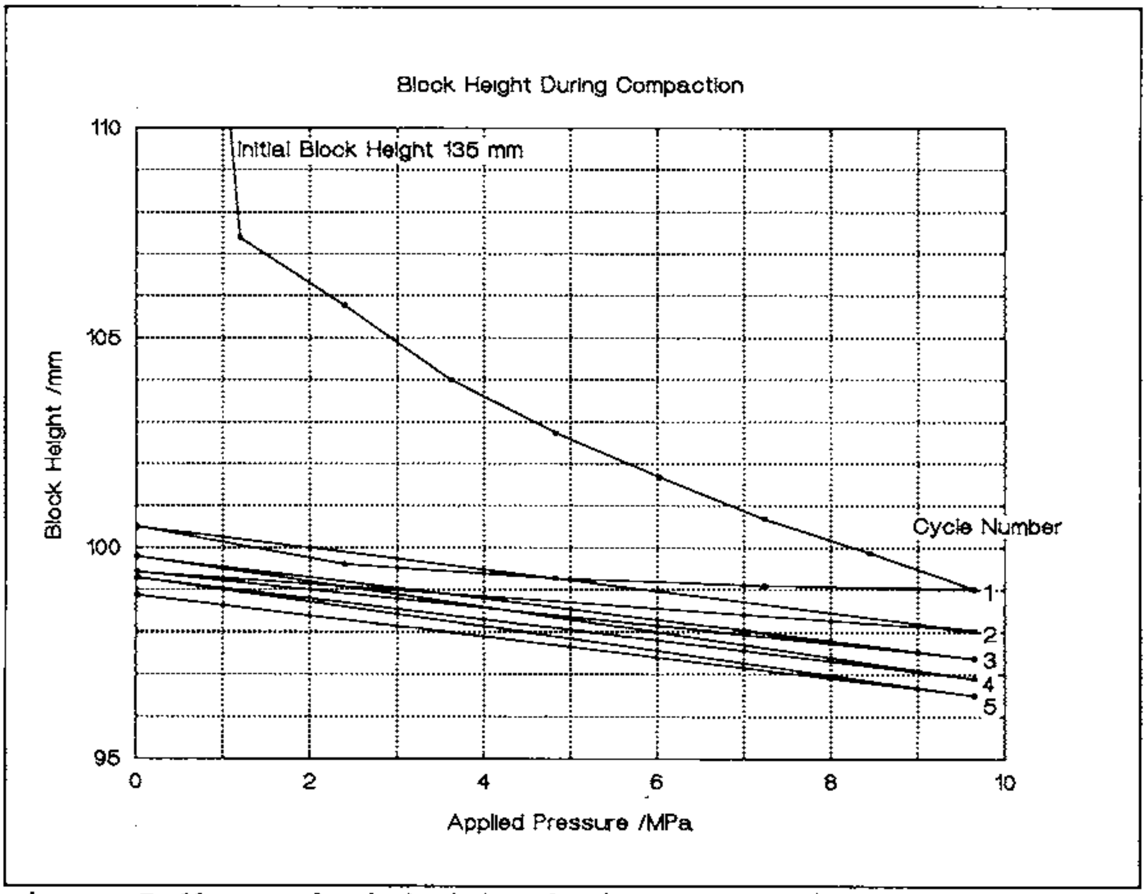


Figure 5.1b Block height during compaction.

The upper region of the block, being the nearest to the moving piston during compaction, would be expected to be in the most stressed/compacted state (closest to a hydrostatic condition). Therefore it could be assumed that this region would exhibit the largest elastic recovery and consequently the most rapid rate of reduction in side wall pressure. The central region of the block would be in a similar condition but could not begin to significantly decompress until the upper layer's vertical restraining influence had diminished sufficiently. This would then result in an effective lag in lateral decompression which would increase with distance from the moving piston i.e. towards the lower regions. Fig 5.1a (above) illustrates this progressive decompression. If the lateral and base pressures are examined at an applied pressure of 2.4MPa, it can be seen that the base pressure is the largest, greater than that applied but reducing the most rapidly. The upper region transducer shows that the lateral pressure in this band is the lowest and reducing quickly but not as fast as the base pressure. The central region has just begun to decompress significantly but remains higher than either the lower or upper regions and the lowest region is yet to begin dropping significantly.

In summary, as the applied pressure is raised to a maximum of 9.7MPa, the base pressure as recorded by the LVDT rises to 68% of this value and the mould side pressures to only 37%, 36% and 29% (upper, central and lower regions of the block respectively). On compression the bottom and side pressures rise linearly with applied (top) pressure. On decompression however, they fall back more slowly than the applied pressure, showing a strong hysteresis pattern.

5.2 DOUBLE-SIDED COMPACTION

There are various ways of applying "double-sided" pressure. The one used here was to fix the base plate, move the top plate (piston) and to let the sidewalls of the mould "float". An exact equality of top and bottom pressures was not achieved, as it might have been with mechanically linked top and bottom pistons. Perfect double-sided compression might have raised the sidewall pressure a little higher, probably to about 50% of the applied piston pressure.

The plot of applied pressure against recorded pressure is shown as Fig 5.2. The arrangement of the double sided compaction rig was such that only the central side wall and base pressures were recorded. Both of the recorded pressures were seen to be significantly higher than for single sided, 7.9MPa and 4.4.Mpa for the base and centre-side respectively. This represents a 12% increase in mould base pressure and a 9% increase in mould sidewall pressure (to 81% and 45% respectively).

This would appear to clearly indicate that double sided compaction was more effective in compressing the block. However, although significant in terms of pressure transmission for a high internal friction mix (no cement) when 5% cement was added to the mix, the pressure difference between single sided and double sided reduced to 10% and 5% for base and centre respectively.

Furthermore, when these blocks were tested for wet compressive strength after seven days of damp curing, the single

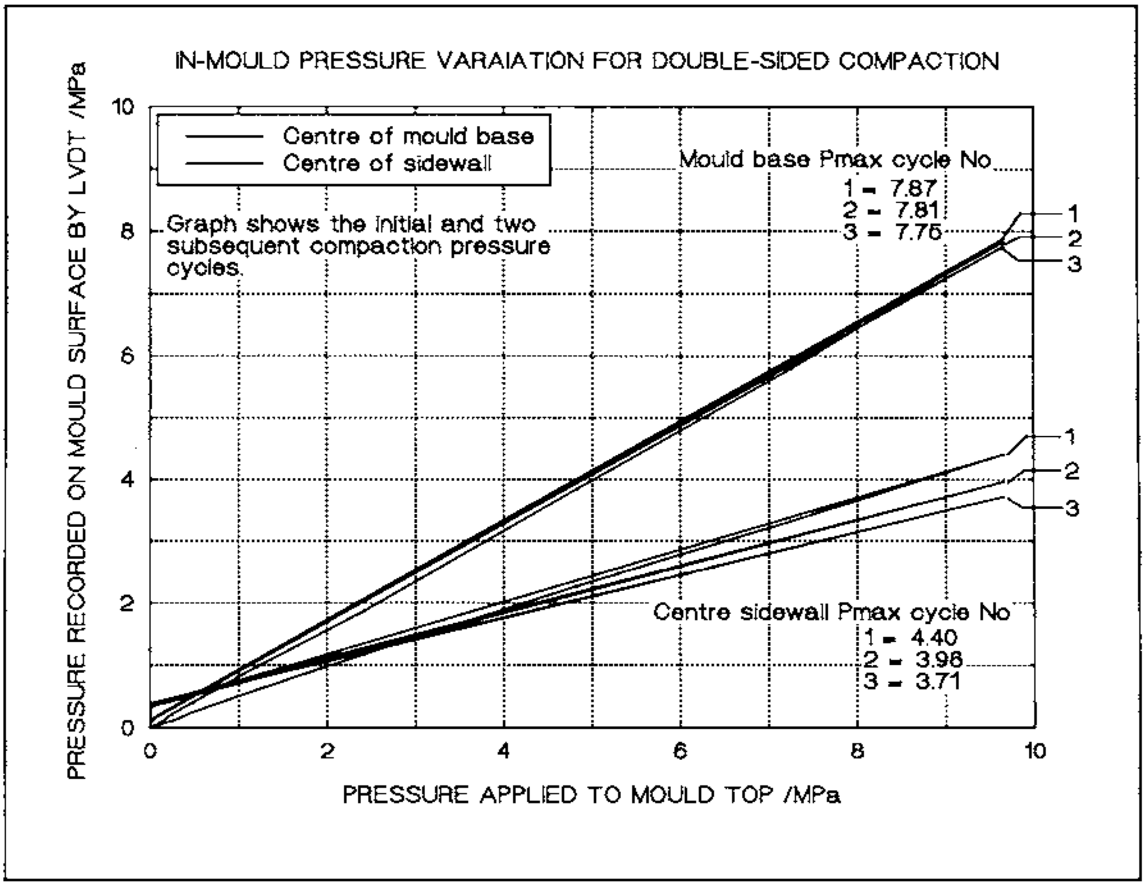


Figure 5.2a Pressure transmission for double-sided compaction

sided ones gave an average of 2.84 MPa (std 0.076) while the double sided ones gave an average of 3.03 MPa (std 0.087). This represents only a 7% increase in wet compressive strength, although this difference is likely to increase marginally with additional curing time.

Having initially stated that by using a soil mix without cement the internal friction would be higher, the difference between the above cases with and without cement should be expected. The apparently large increase in transmitted pressure for the no-cement blocks translates into only a small increase for the cement blocks.

If it is assumed that density is proportional to strength as has been suggested by other authors (Ingles & Metcalf, 1972, Ref No.10 & Webb, 1991, Ref No.20) then it might be expected that double sided compaction would produce a more uniform internal density distribution. If the compacted sintered bearing is taken as an example; single sided compaction produces a compact which is demonstrably more dense in the region nearest to the compacting piston. When compaction is double-sided the compact density is improved throughout but with a reduced improvement in the central region. If this is related to stabilised block strength then this would suggest that double-sided compaction should produce blocks which have more uniform internal density distribution and hence a more uniform strength distribution through their height.

Double sided compaction is more effective than single sided compaction which is clearly shown by the increase in both base and mould wall pressures. However, this increase only results in a 7% increase in seven day wet strength. As such the additional mechanical complexity and associated cost required to produce a commercial double sided block press would appear unwarranted.

5.3 REDUCTION IN MOULD WALL FRICTION

The effect of reducing mould wall friction was examined by lining the mould with a twin thickness of plastic sheeting, separated by a lubricating oil film. Figure 5.3 shows the plot of applied pressure against the recorded pressures. Lining the mould with plastic was used as an experimental technique to assess the effect of mould wall friction and would not be recommended for field use. During compaction the inner layer of plastic was dragged down with the compacting soil and forced to ruck into the body of the block thus producing flawed blocks. However, this should not invalidate the pressure transmission data gathered.

Both the base pressure and all of the recorded mould wall pressure were seen to rise significantly compared with the much rougher datum model. The base pressure rose to 7.6MPa whilst the upper and lower mould wall regions rose to 4.1 and 3.6 MPa respectively. This represents a 13% increase in base pressure and a 17% and 29% increase in upper and lower pressures. Again it would be expected that this would translate into a reduced effect in the final wet strength of the blocks but it does show that mould wall friction plays an important role in determining the effectiveness of the applied pressure in block compaction. It would be recommended that the mould should be as smooth as possible and that any machining marks etc should be orientated in the direction of the soil material movement during compaction i.e. perpendicular to the compaction piston.

5.4 MOULD WALL TAPER

Mould wall taper was investigated by angling the mould side walls to 1° and 5° from the vertical. This was done by separating the mould walls and bolting them in place with tapering sets of shim steel to produce the desired angles. The mould was arranged such that compaction was from the larger side of the taper. Figure 5.4 shows the plot of applied pressure against recorded pressure. The base pressure was seen to rise slightly to 6.9MPa but this was believed to be a function of the experimental method. The same dry mass of soil was used to produce each block throughout the set of tests and as a result

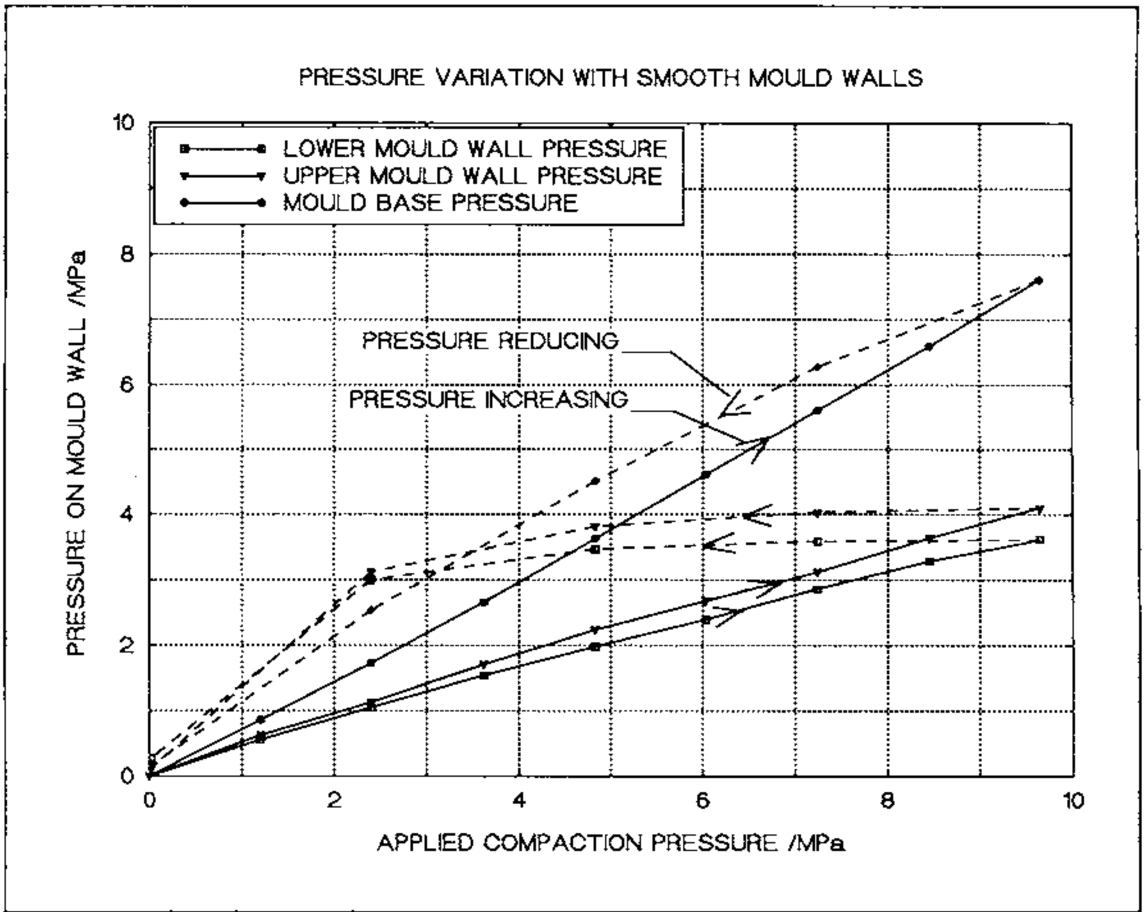


Figure 5.3 Pressure transmission with smooth mould walls

of the manner in which the mould was tapered, the final block height was reduced. Therefore some increase in base pressure would be expected as the height through which the applied pressure was acting was reduced. The mould side wall pressures were also recorded. The upper region pressure was significantly lower than that for the standard mould configuration but this was again a result of the manner in which the mould was tapered, the reduced block height and increased separation between the compacting piston and the mould wall effectively placed the upper transducer above the top of the block. The central and lower region pressure were slightly increased at 4.2 and 2.9MPa but not above what might be expected as a result of the increase in projected area seen by the compacting material. The 5° taper mould produced similar results; the base pressure increased to 7.5MPa while the central pressure increased to 4.4MPa.

In conclusion, mould taper does not have any apparent beneficial effect on pressure transmission. The block ejection cycle however required much lower forces which had to be sustained for a much shorter period of time. Taper may be used to ease ejection but the somewhat awkward shape of the blocks does not seem to justify the improved ease of ejection. Taper would not be recommended for incorporation into block production machines.

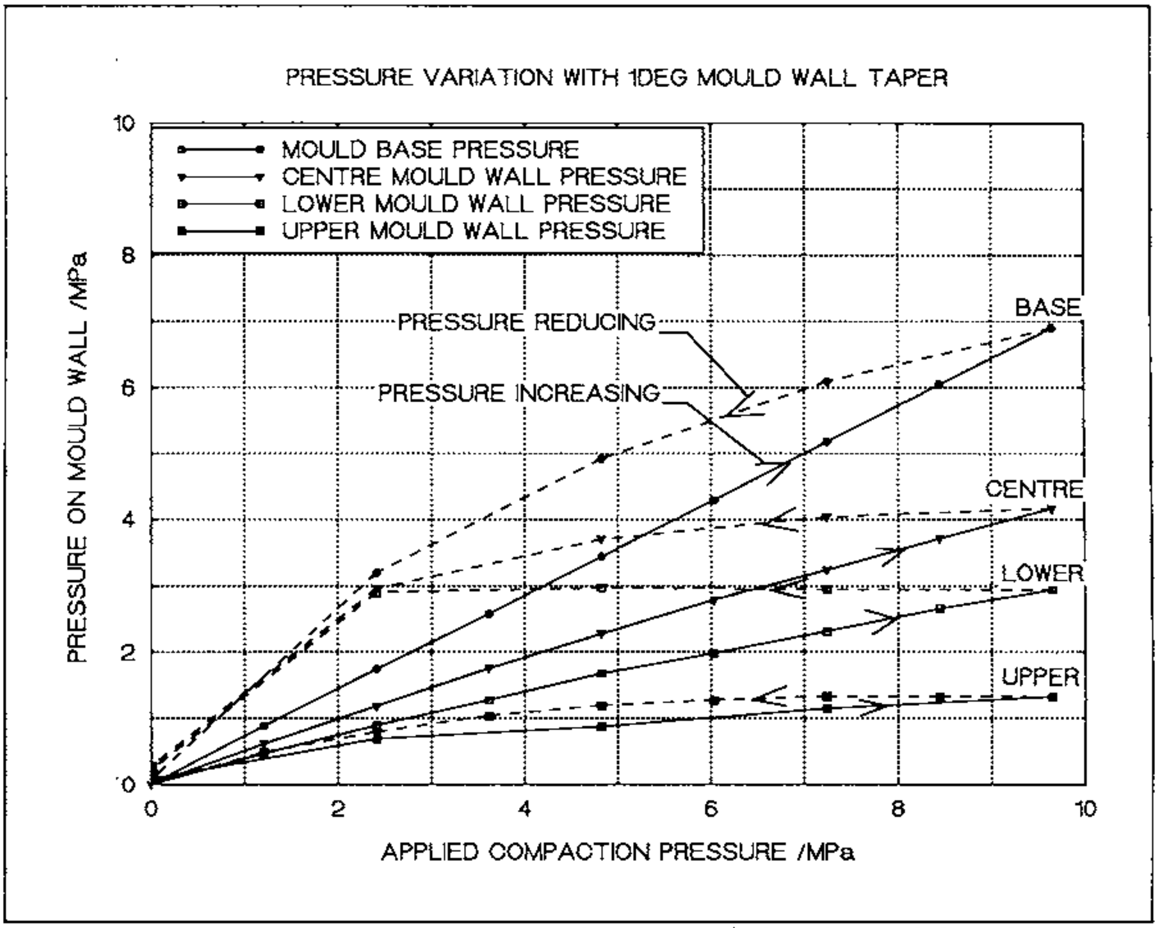


Figure 5.4 Pressure transmission with 1° mould wall taper

5.5 PRESSURE CYCLING

Here the term pressure cycling is used to imply the decompression of a fully compacted block to various levels of pressure and the subsequent recompression back to the original fully compacted applied pressure.

The manner in which a soil block would respond to applied pressure cycling was unclear. Three possible effects were expected prior to experimentation. The cycling would have no effect on the transmitted pressure, the transducers recording the same value of pressure as that recorded for the initial cycle. The cycling would progressively increase the transmitted pressure to a limiting maximum, independent of the magnitude of the cycle. The cycling would increase the transmitted pressure to a limiting maximum value depending on the magnitude of the cycle.

Table 5.5a Pressure transmission on full-pressure cycling

Cycle No	Maximum applied pressure /MPa	Maximum mould base pressure /MPa	Maximum upper mould wall pressure /MPa	Maximum centre mould wall pressure /MPa	Maximum lower mould wall pressure /MPa
1	9.65	6.72	3.48	3.55	2.84
2	9.65	6.85	2.55	3.38	2.26
3	9.65	6.93	2.14	3.20	2.04
4	9.65	6.92	1.87	3.18	1.89
5	9.65	6.92	1.69	3.13	1.81

Table 5.5a shows the pattern of pressure change in the mould side wall for cycling from 9.7 to 0.1 MPa for standard single sided compaction. It can be seen that the base pressure remains almost constant, in general rising slightly with successive cycles but not significantly (the LVDT hysteresis will account for an increase of 0.1MPa per cycle when cycling from 0 to 7 MPa). The mould side wall pressure appears to drop significantly with each cycle, dropping less with successive cycles.

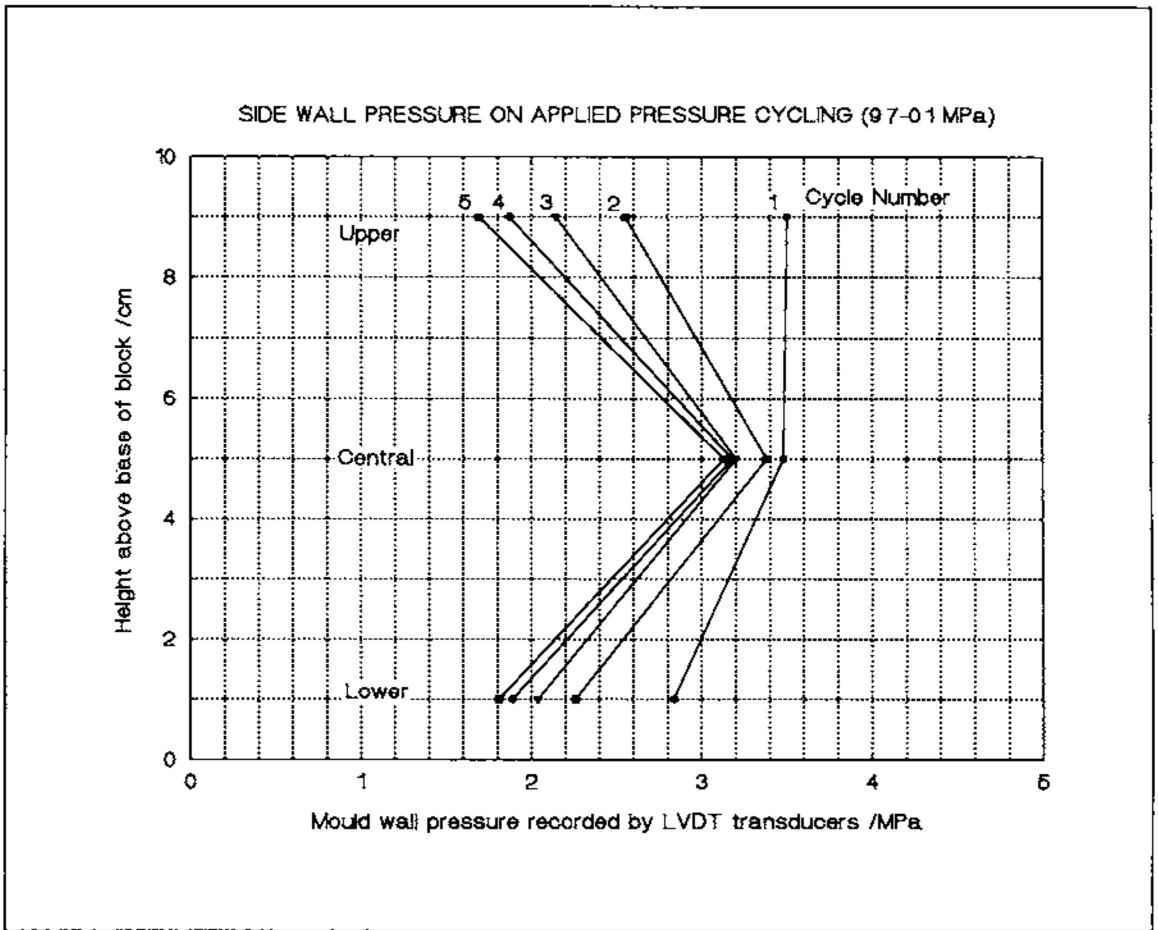


Figure 5.5a Recorded pressure on mould walls (upper, central and lower regions) with successive pressure cycles

Figure 5.5a shows the pattern of pressure reduction with successive cycles plotted with height of the transducer from the base of the block. This appears to indicate that the cycling action reduces the upper and lower side wall pressure at a greater rate than that for the central region. It might be argued that the reduction in upper side wall pressure is a result of the reduction in block height (see below). However, the reduction in the central and lower regions could not be accounted for on this basis.

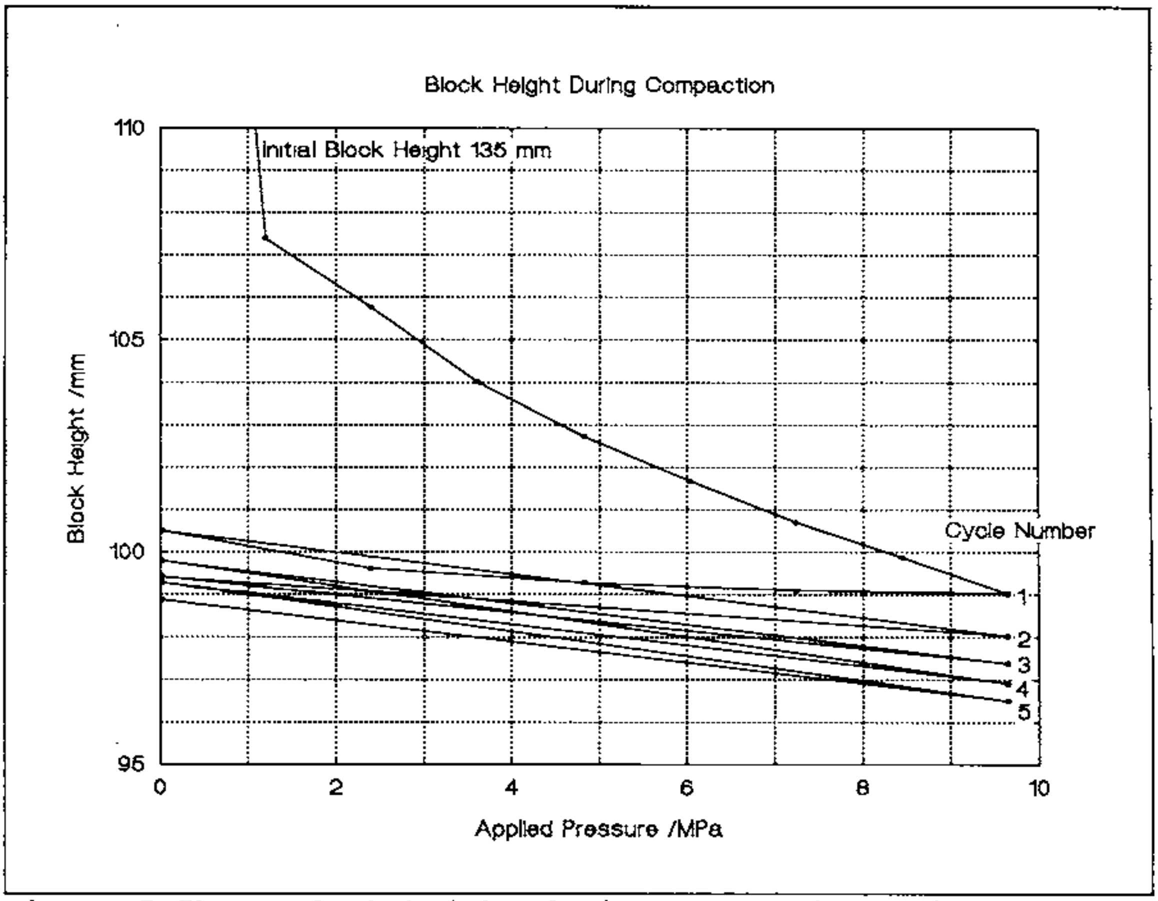


Figure 5.5b Block height during compaction and decompression.

If Figure 5.5b is examined then it can be seen that the block height reduces to 96.5 mm under full pressure after 5 cycles. The upper transducer's centre line is 90 mm above the base of the block and the active face is 5mm in radius. The transducer is therefore recording the upper side wall pressure from 3.9-13.9 mm below the upper surface for the first cycle and from 1.5-11.5 mm below the surface for the fifth cycle. The pressure reduction might then be accounted for if it is assumed that the edge region of the compaction piston is at atmospheric pressure (0 MPa on the LVDT scale) and that a region of transition from 0 to 3.5 MPa (max side wall pressure on the first cycle) exists. As the upper face of the block moves downward approaching the upper LVDT then it might be expected that the recorded pressure would drop. However without including a pore pressure type concept, it appears more difficult to explain the fall in the lower side wall pressure. No satisfactory explanation has yet been formulated.

Partial pressure cycling (cycling from full compaction pressure to a lesser pressure greater than zero) is shown as Figure 5.5c. Table 5.5b shows the numerical values for the maximum and minimum cycled pressure values for each cycle. This plot of applied pressure against recorded pressure is from one of the blocks which were used to determine the effect

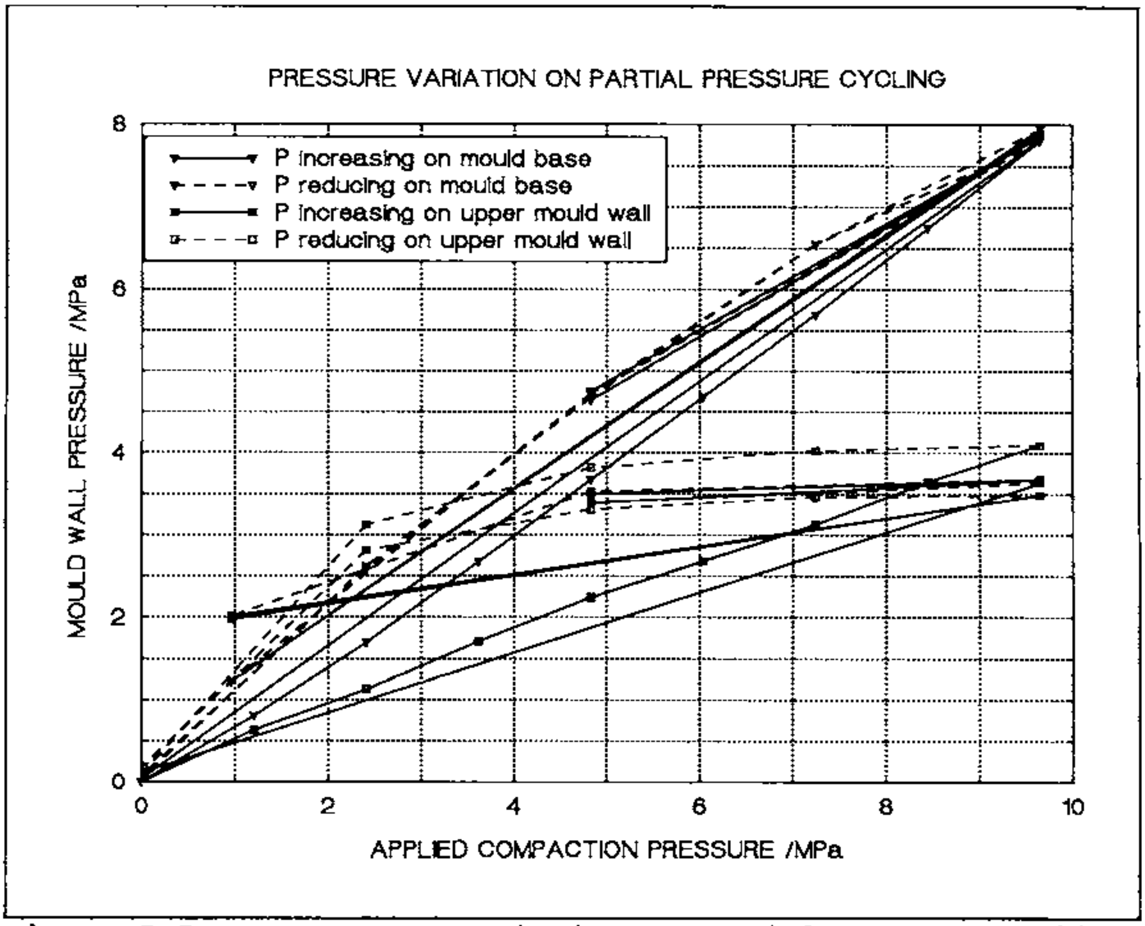


Figure 5.5c Pressure variation on partial pressure cycling

Table 5.5b Pressure values relating to figure 5.5c

Cycle No	Applied pressure /MPa	Mould base pressure /MPa	upper mould wall pressure /MPa
1	0.00 → 9.66	0.00 → 7.80	0.00 → 4.10
2	0.20 → 9.66	0.05 → 7.82	0.12 → 3.63
3	4.83 → 9.66	4.64 → 7.86	3.39 → 3.67
4	4.83 → 9.66	4.75 → 7.87	3.49 → 3.68
5	0.97 → 9.66	1.22 → 7.91	2.01 → 3.48
6	0.97 → 9.66	1.22 → 7.95	1.97 → 3.48
End	0.02	0.10	0.19

of reduced friction but this trace shows the effect of residual wall shear forces well. The block was initially compacted to 10 MPa and then cycled down to 0.1MPa. On repressurising the mould wall side pressure dropped significantly. The block was then cycled from full pressure to half pressure twice and showed no

This appears to suggest that pressure cycling has little or no effect on a region unless the pressure is dropped back to significantly less than the lag period pressure mentioned above (section 5.1).

5.6 Summary

From the work outlined above it may be concluded that the internal compaction state of the soil material within the mould is not hydrostatic. It would appear unlikely that the side mould walls, under datum conditions, will ever exceed 50% of the applied pressure and normally be less than 40%. This being the case, significant material savings in mould wall thickness would appear possible. A significant reduction in the pressure applied to the compacting piston is apparent in the base plate which may be assumed to be indicative of internal shear and inter-particle, particle-mould-wall friction. The mould wall shear is significant and may be reduced by smoothing/lubricating the mould walls. Taper has little or no beneficial effect on the compaction proses but does reduce the ejection load and the time for which the load must be applied. Double sided compaction does have a significant beneficial effect on compaction but this benefit would be small compared to the cost of the extra mechanical complexity entailed.

6. THE PRESSURE-CEMENT-COMPRESSIVE STRENGTH RELATION

6.1 INTRODUCTION

The compaction pressure and the cement content used in the manufacture of stabilised soil blocks are, for a given soil, the prime determinants of the block's final cured strength. Compaction pressure and cement content may be traded against each other for a given final cured strength. The compaction pressure used in field production will depend on the type of compaction machinery selected at the project outset and so may be considered a capital cost for the process while the cement content used may be considered a running cost. As high pressure compaction machines are more expensive to purchase than low pressure machines it is useful to examine the relative effect of both compaction pressure and cement content on cured strength.

M.G.Lunt of the UK Building Research Establishment (Building Research Establishment, 1980, Ref No.4) conducted a series of tests on two Ghanaian soils (both with high fines content, 49.0 and 56.0%) to assess the effect of increased compaction pressure on the blocks' performance when stabilised with 6% lime by dry weight. He concluded that "Improved performance can be achieved by increasing the compaction pressure although the degree of improvement diminishes as this pressure is increased. It is suggested that presses operating in the range 8 to 16 MPa could give satisfactory and economical results." This work thus suggests that there may be some economic advantage in using a high pressure compaction machine. However, increasing the compaction pressure is only one way to improve the final compressive strength of a block. Although it is generally accepted that the performance of a block will be improved both by raising the compaction pressure and by increasing the stabiliser content, the relative effect of these two changes appears to be uncharted. For example, does a doubling of compaction pressure give the same improvement in strength as a doubling of cement content? The following section examines the effect of both compaction pressure and stabiliser (cement) content on the final (seven day) wet compressive strength of the *soil-A* used for the above tests.

6.2 EXPERIMENTAL INVESTIGATION

A number of soil-cement cylinders were produced for a range of compaction pressures from 1 to 10 MPa and a range of cement contents from 3% to 11%. For each combination of pressure and cement three cylinders were produced, the average value of the wet compressive (seven day) strength was used in subsequent comparisons. The *soil-A* used is described in appendix B. This soil was selected as one which should be suitable for stabilisation based on previous authors' reports (United Nations, 1964, Ref No.17). Although the numerical values given below are unlikely to be correct for other soil types, it is expected that the trends exhibited will be, provided that the other soils fall within the range of suitable soils as defined in DTU Working Paper No.38.

The mould used was that specified in BS1924 (British Standards Institution, 1975, Ref No.3). A constant water content of 8% was used throughout the set of experiments[1]. The mould was filled with a constant mass (± 0.2%) of stabilised mixture regardless of the cement content. The cylinders produced were between 110mm and 125mm high, depending on the compaction pressure used, each having a nominal diameter of 50mm. The compacted green cylinders were sealed inside plastic bags in a damp atmosphere in batches of three and left to cure for seven days before complete immersion in water prior to wet compressive strength testing.

The results of the testing showed that both an increase in cement and an increase in compaction pressure increases the seven day wet strength, however the relative influence of each is different. Figure 6.3a and 6.4a respectively show the rate of gain in strength when either cement or compaction pressure is held constant. On each graph the data points connected by dashed lines are the average of three experimental results. The error bars associated with the data points represent a statistical confidence level of 90%, the length of the bars giving an indication of the scatter in results.

In order to relate cement percentage and compaction pressure, the raw experimental data (compaction pressure, cement percent and cured strength) was used as the input for a PC-based modelling package SPSS. A number of models were tried of which a natural log against natural log type plot was found to be the best. The solid lines on the figures given below represent the best fit to the data generated by SPSS;

$$\ln(str) = (0.315 \times \ln(pr)) + (1.216 \times \ln(cem)) - 2.178)$$

Where; str = compressive strength in MPa
pr = compaction pressure in MPa
cem = cement content percentage.

This model gave an adjusted R square measure of fit of 98.2% (Multiple R 99.1%)

---

[1] The optimum water content at the time of compaction should strictly have been found for each compaction pressure and cement content. However the *soil-A* used has a low sensitivity to moisture content and as such the effect on the experimental data should be minimal.

6.3 THE EFFECT OF COMPACTION PRESSURE AND ENERGY ON SEVEN DAY WET STRENGTH

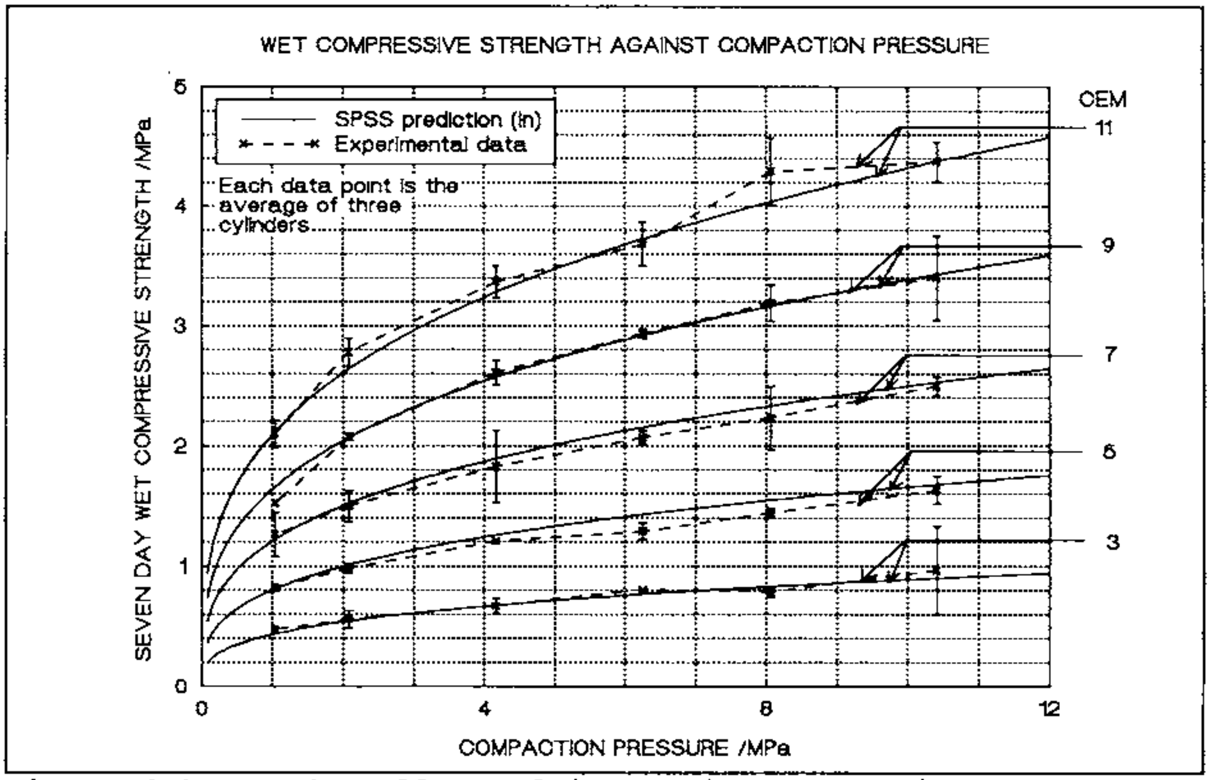


Figure 6.3a The effect of increasing compaction pressure on seven day wet compressive strength

For a given cement content (Fig 6.3a) strength increases with increased compaction pressure. Below 2 MPa this increase is rapid while above this the rate of increase reduces, tending towards a maximum strength as previously reported by Lunt (in Building Research Establishment, 1980, Ref No.4). Table 6.5 (below) shows the effect of doubling compaction pressure on the wet compressive strength. The figure given is the fractional increase in absolute strength resulting from doubling the respective variable. It can be seen that although the absolute rate of gain in strength is higher for high cement contents, the fractional increase in strength is fairly constant. A doubling of compaction pressure results in roughly 23% increase in wet compressive strength throughout the range.

Figure 6.3a shows that a given change in the compaction pressure of low pressure machines will have a large effect on the cured strength. A poorly operated or poorly maintained Cinva Ram may only operate at 1MPa, instead of the 2MPa usually quoted. This would result in a cured block strength 25% lower than that for a well operated/maintained machine.

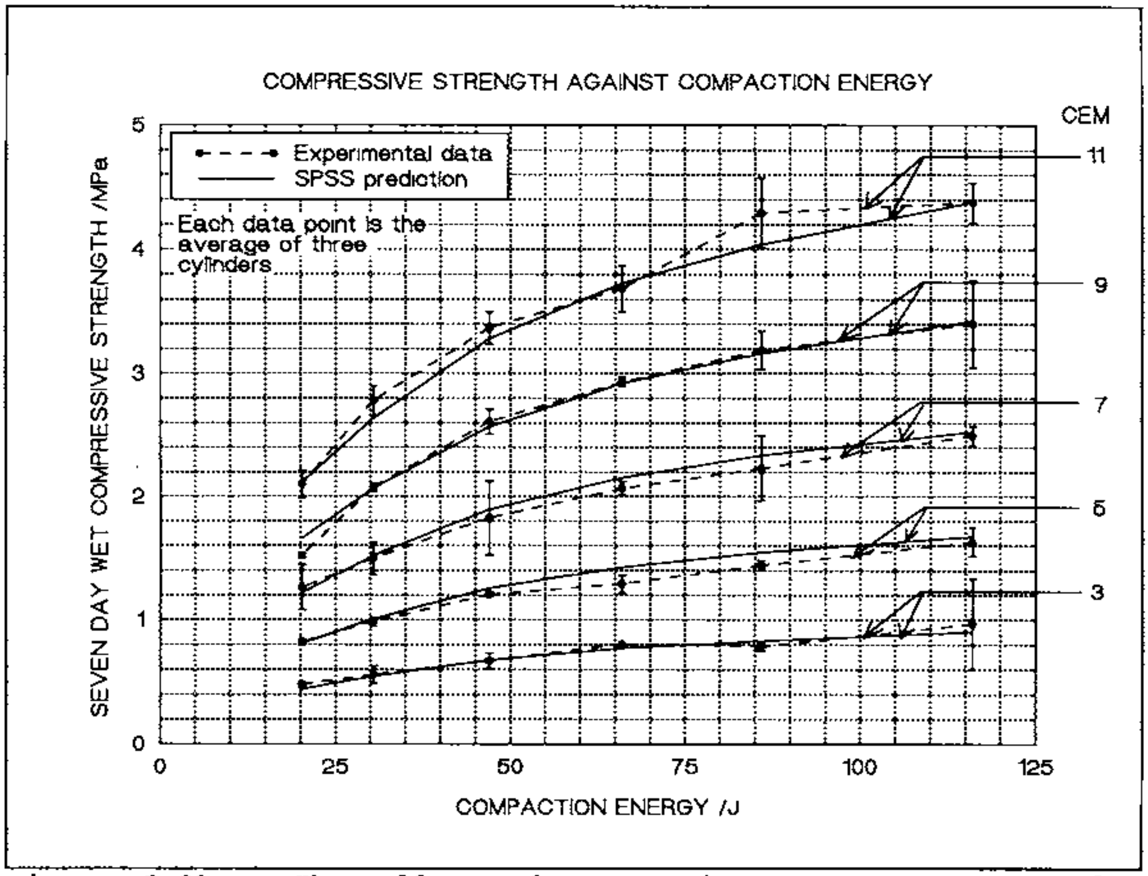


Figure 6.3b The effect of compaction energy on seven day wet compressive strength

Figure 6.3b is a plot of compressive strength against compaction energy. This graph is similar to 6.3a but shows the compaction process in terms of the energy expended in compacting the samples[2]. If the 3% cement trace is examined then it can be seen that by doubling the compaction energy from 25 to 50J the compressive strength increases by 37%, but doubling the energy from 50J to 100J only increased it by 22.9%. This is in keeping with the results shown in figure 6.3a as the energy required to increase the compaction pressure one unit is greater at higher pressures than at lower ones and hence the diminishing return.

---

2. The energy values are based on those found when quasi-statically compacting full-size blocks and were not directly recorded for the cylinders.

6.4 THE EFFECT OF CEMENT CONTENT ON SEVEN DAY WET STRENGTH

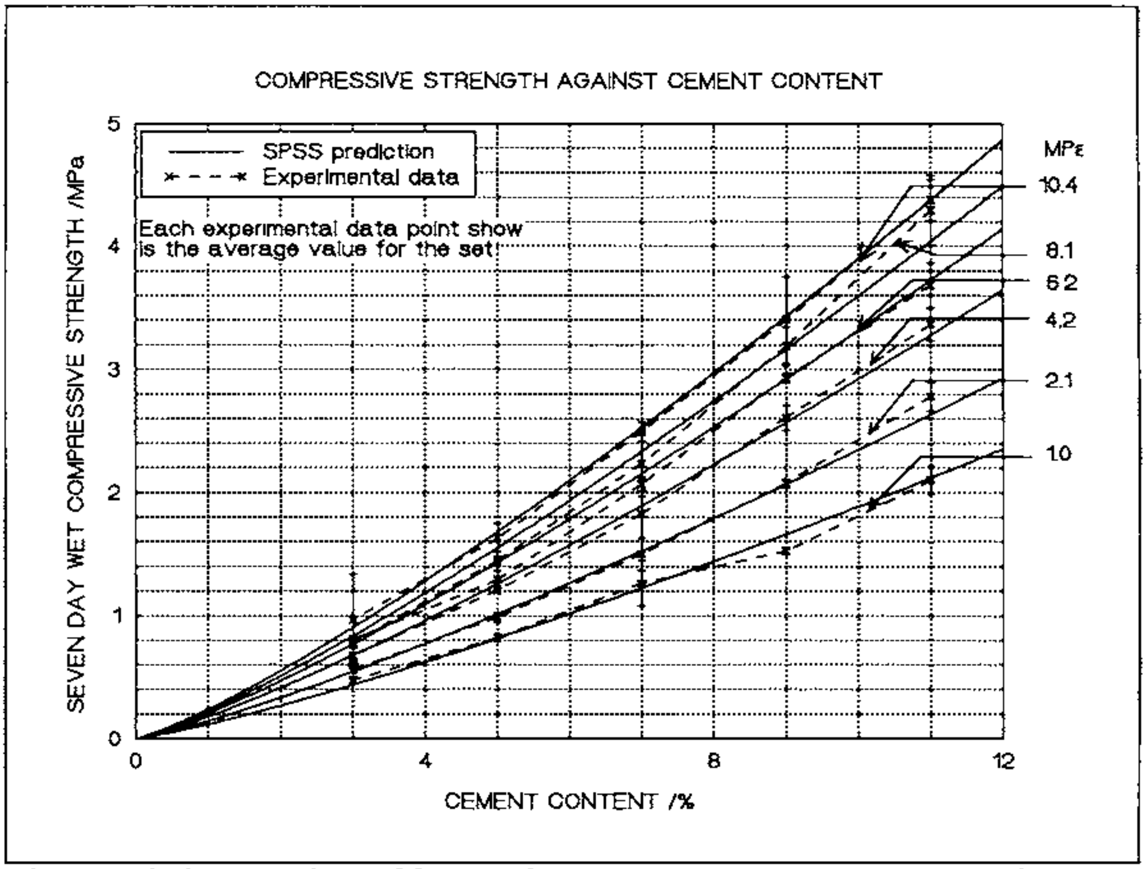


Figure 6.4a The effect of cement content on seven day wet compressive strength

If figure 6.4a is examined it can be seen that for a given pressure the rate of increase in absolute strength increases with increasing cement content. However, if table 6.5 (below) is examined it can be seen that the fractional increase in strength remains approximately the same, reducing slightly at higher cement contents. A doubling of cement content from 3 to 6% at a compaction pressure of 1.0 MPa produces a strength increase of 140% while a doubling of cement from 6 to 12[3]% produces an increase of 133%.

[3] The values for 12% cement used here are based on the SPSS model and represent an extrapolation above the maximum experimental value of 11%.

6.5 THE PRESSURE-CEMENT-STRENGTH RELATIONSHIP

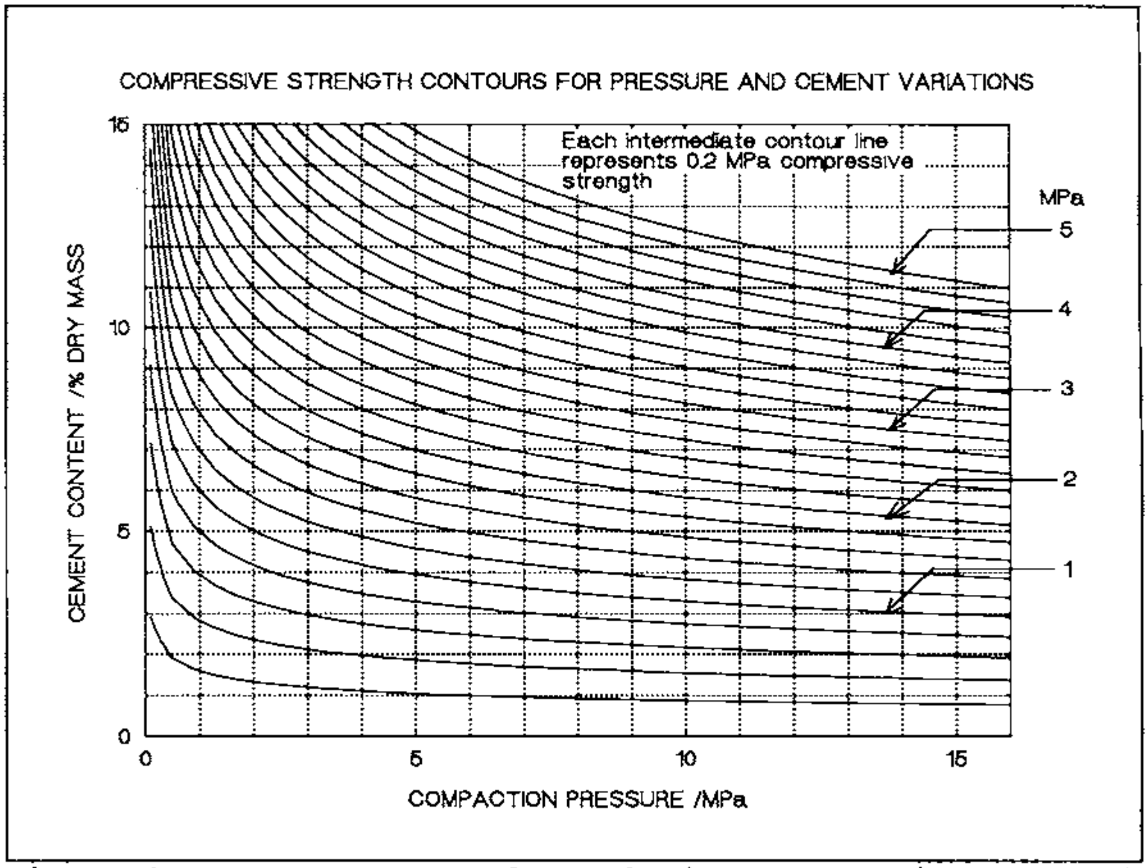


Figure 6.5a A contour plot relating compressive strength to cement content and compaction pressure

Figure 6.5a and 6.5b show the combined picture. Figure 6.5a is a contour plot showing lines of constant wet strength in relation to cement content and compaction pressure. Figure 6.5b is a three dimensional representation of the strength equation produced by SPSS from the experimental data. It can be seen from these figures that increasing the cement content of a stabilised block will in general provide a more effective method of increasing strength than increasing compaction pressure. Two standard wet strength values are normally quoted, either 1.4 MPa (Fitzmaurice 1958) or 2.8MPa (Webb 1988)[4]. The relative effect of cement and compaction pressure may be examined by regarding the reduction in cement content required when changing production from a 2MPa compaction machine to a 10 MPa machine at each of these two strength standards. Figure 6.5a shows that for a wet strength of 1.4 MPa and a compaction pressure of 2 MPa a cement

[4] Strength standards for soil-cement blocks are now becoming more widespread and vary from country to country to reflect the differing climates.

content of 6.6% would be required, while for a compaction pressure of 10MPa a cement content of 4.3% would be needed. In effect, for this soil, increasing the compaction pressure five times produces only a 35% reduction in cement demand. If the 2.8 MPa strength standard is considered, the same five fold increase in compaction pressure again results in a cement saving of only 35%.

Table 6.5 The relative effects of pressure and cement

Doubled Parameter	Strength Increase Compaction 2 → 4 MPa Energy 25 → 50 J Cement 3 → 6 %	Strength Increase Compaction 4 → 8 MPa Energy 50 → 100 J Cement 6 → 12 %
Compaction Pressure Doubled (11%cem)	2.59 → 3.23 MPa + 24.7%	3.23 → 4.03 MPa + 24.7%
(3%cem)	0.54 → 0.67 MPa + 24.1%	0.67 → 0.82 MPa + 22.4%
Compaction Energy Doubled (11%cem)	2.38 → 3.35 MPa + 40.7%	3.35 → 4.20 MPa + 25.0%
(3%cem)	0.49 → 0.70 MPa + 37.0%	0.70 → 0.86 MPa + 22.9%
Cement Content Doubled (10.4MPa)	0.90 → 2.10 MPa + 133.3%	2.10 → 4.89 MPa + 132.9%
(1.0MPa)	0.42 → 1.01 MPa + 140.5%	1.01 → 2.35 MPa + 132.7%

The trend which emerges from this study is that the final wet strength achieved by the block is much more sensitive to changes in the cement content than in the compaction pressure. By increasing the compaction pressure 400% the cement saving resulting is 35%. In order to interpret these figures, a simple economic model was constructed to compare the cost effectiveness of a 2MPa Cinva Ram and a 10MPa Brepack when producing blocks of 1.4 and 2.8 MPa wet compressive strength. This model is presented in the following section, 6.6.

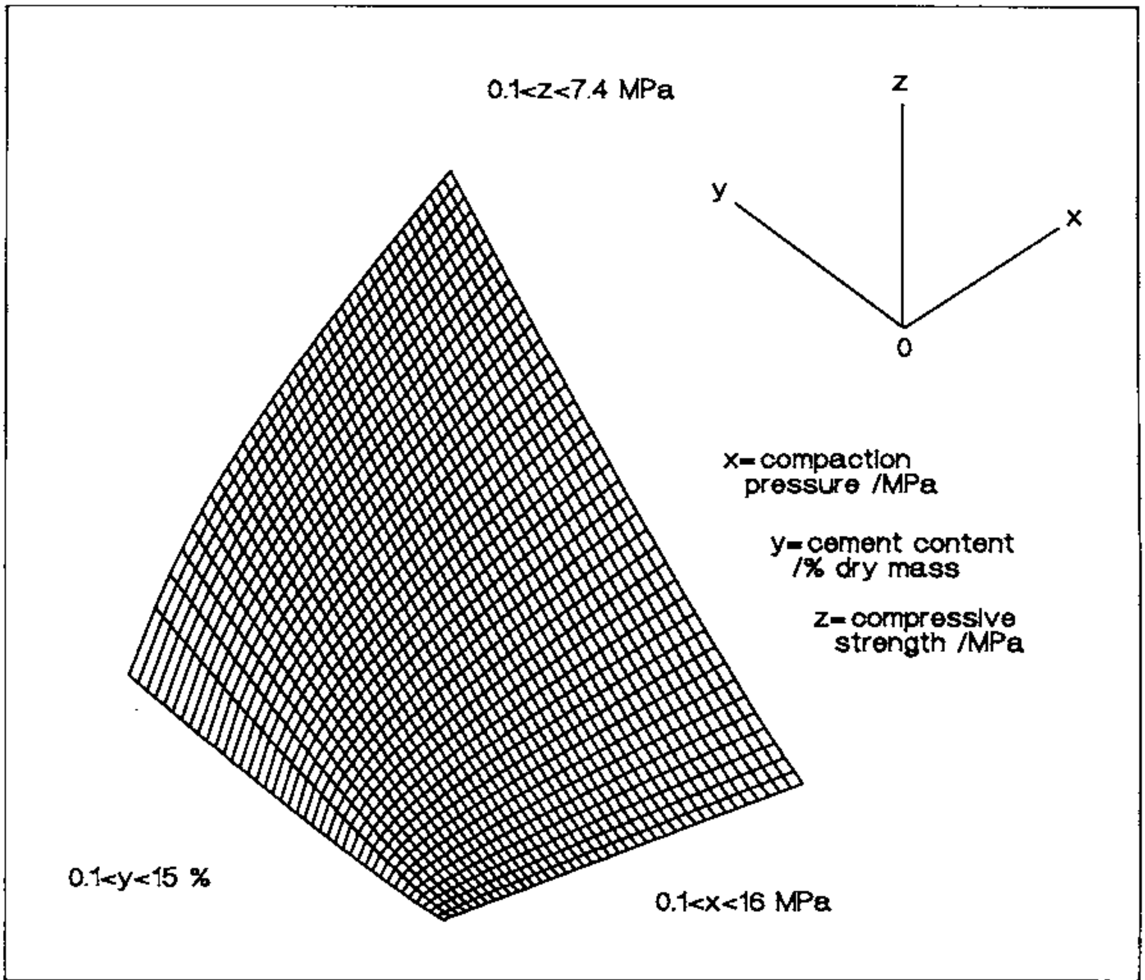


Figure 6.5b A three dimensional representation of the relationship between compaction pressure, cement content and compressive strength

6.6 SIMPLE ECONOMIC COMPARISON BETWEEN MACHINES GIVING RESPECTIVELY 2 AND 10MPa COMPACTION

The following model uses the 2MPa compaction pressure Cinva Ram and 10MPa Brepack machines for comparison. Two comparisons are made, one for blocks of 1.4 MPa wet compressive strength and one for blocks having 2.8MPa wet compressive strength.

It is assumed that both machines are operating in the same country with the same cost for cement and labour; £3.00 per 50 kg bag and £3.50 per man day (the costs quoted are those for Sri Lanka in 1993). Both machines use the same *soil-A*, as used in the above experimentation, compacted at 8% water content, both produce blocks of 290x140x100mm.

Both of the machines are manually operated toggle lever mechanisms. The Brepack generates higher pressure by the incorporation of a hydraulic ram which is operated after initial compaction by toggle lever has occurred. As large variations in the actual and quoted production outputs of each of these machines is common (production output depends heavily on the experience and dedication of the operators), it has been assumed that the maximum Cinva Ram output, quoted by the machine manufacturers, of 300 blocks per 8 hour day will be achieved and will be taken as the datum from which to extrapolate a comparable production figure for the Brepack. The maximum Cinva Ram production rate equates to the production of one block every 96 seconds. It will be assumed that the additional time taken to operate the Brepack hydraulic system is 20 seconds giving the Brepack a production rate of one block every 116 seconds or 248 blocks per 8 hour day.

The labour requirement for the Brepack may also be based on that for the Cinva Ram. The machines produce different numbers of blocks of different density; 300 blocks per day (see above) of 1980 kg/m^3 for the Cinva and 248 blocks per day of 2130 kg/m^3 for the Brepack. Hence the labour required, per day or per block, for soil winning and block compaction is different. If the labour distribution shown below is assumed for the Cinva Ram when producing blocks of 1.4 MPa wet compressive strength (requiring 2,084.4 kg of soil per 300 blocks) then soil winning/processing labour costs may be calculated per kg of soil required. Similarly the labour cost of compaction and stacking may be calculated per block, 16 man hours are required to compact 300 Cinva Ram blocks (a labour cost of £0.023 per block) while 16 man hours are required to produce 248 Brepack blocks (£0.028 per block).

It is assumed that the soil used is free and the only cost is then the cost of winning which is included in the labour cost.

It is assumed that the working life of a Cinva Ram is 3 years at full production resulting in 270,000 blocks when working for 6 days per week and 50 weeks per year. The life of the Brepack is also assumed to be 270,000 blocks.

It is assumed that the initial capital will be recouped with 30% interest within the working life of the machine assuming a 60% utilisation i.e. in five years for the Cinva Ram and six years for the Brepack. This results in a discount factor of 2.436 for the Cinva Ram and 2.643 for the Brepack.

LABOUR COSTS

Labour per day for a Cinva Ram producing 300 blocks (1.4 MPa wet compressive strength) per 8 hour day

soil winning	2 men to dig the soil, spread it out for drying and crush/sieve the dried material.
soil winning	1 man to mix the material and prepare batches for compaction.

block pressing	2 men to operate the machine and stack the green blocks for curing.

Assuming the above labour distribution for a Cinva Ram production unit then the cost of labour per kg of soil used may be found.

Soil required per block (see data)	6.948 kg
No of blocks produced per day	300
Total mass of soil required	2,084.4 kg
Labour cost per man per day	£3.50
Labour required to win/process soil	3 man days
Total soil winning labour cost	£10.50
Soil winning labour cost per kg	£0.0050

Labour cost for block pressing (Cinva Ram)	£0.0233 per block
Labour cost for block pressing (Brepack)	£0.0282 per block

CEMENT COST

Cement cost per 50kg bag	£3.000
Cement cost per kg	£0.060

CINVA RAM DATA (prices in Pounds Sterling 1993)

Purchase cost of machine £382.88 (1988 cost reported by Webb (1988) inflated by 5% pa)
Total freshly demoulded weight of one block 8.0kg (demould density 1980kg/m^3, volume 290x140x100mm)

1.4MPa wet compressive strength

Cement percentage required for 1.4MPa wet strength 6.6% (based on figure 6.5a)

Mass of soil per block	6.948 kg
Mass of cement per block	0.459 kg
Mass of water per block	0.593 kg

2.8MPa wet compressive strength

Cement percentage required for 2.8MPa wet strength 11.7% (based on figure 6.5a)

Mass of soil per block	6.631 kg
Mass of cement per block	0.776 kg
Mass of water per block	0.593 kg

CINVA RAM ANALYSIS FOR 1.4MPa WET COMPRESSIVE STRENGTH

Cost of cement per block 0.459 kg cement per block @ £0.060 per kg	£0.0275
Cost of soil winning labour per block 6.948 kg soil per block @ £0.0050 per kg	£0.0347
Cost of soil pressing labour per block (for Cinva Ram)	£0.0233
Cost of machine depreciation per block £382.88 ÷ 2.436 x 5 ÷ 270,000	£0.0029
Total cost per block	£0.0884

CINVA RAM ANALYSIS FOR 2.8MPa WET COMPRESSIVE STRENGTH

Cost of cement per block 0.776 kg cement per block @ £0.060 per kg	£0.0466
Cost of soil winning labour per block 6.631 kg soil per block @ £0.0050 per kg	£0.0332
Cost of soil pressing labour per block (for Cinva Ram)	£0.0233
Cost of machine depreciation per block £382.88 ÷ 2.436 x 5 ÷ 270,000	£0.0029
Total cost per block	£0.1060

BREPACK DATA (prices in Pounds Sterling 1993)

Purchase cost of machine £3828.80 (1988 cost reported by Webb (1988) inflated by 5% pa)
Total freshly demoulded weight of one block 8.65kg (demould density 2130 kg/m^3, volume 290x140x100mm)

1.4MPa wet compressive strength

Cement percentage required for 1.4MPa wet strength 4.3%

Mass of soil	7.679 kg
Mass of cement	0.330 kg
Mass of water	0.640 kg

2.8MPa wet compressive strength

Cement percentage required for 2.8MPa wet strength 7.6%

Mass of soil	7.444 kg
Mass of cement	0.566 kg
Mass of water	0.640 kg

BREPACK ANALYSIS FOR 1.4MPa WET COMPRESSIVE STRENGTH

Cost of cement per block 0.330 kg cement per block @ £0.060 per kg	£0.0198
Cost of soil winning labour per block 7.679 kg soil per block @ £0.0050 per kg	£0.0384
Cost of soil pressing labour per block (for Brepack)	£0.0282
Cost of machine depreciation per block £3828.80 ÷ 2.643 x 6 ÷ 270,000	£0.0322
Total cost per block	£0.1180

BREPACK ANALYSIS FOR 2.8MPa WET COMPRESSIVE STRENGTH

Cost of cement per block 0.566 kg cement per block @ £0.060 per kg	£0.0340
Cost of soil winning labour per block 7.444 kg soil per block @ £0.0050 per kg	£0.0372
Cost of soil pressing labour per block (for Brepack)	£0.0282
Cost of machine depreciation per block £3828.80 ÷ 2.643 x 6 ÷ 270,000	£0.0322
Total cost per block	£0.1316

The data given above is summarised in table 6.6 below. It can be seen from this simple analysis that for a final wet strength of 1.4 MPa and 2.8 MPa, high pressure compaction is 33.5% and 24.1% more expensive respectively.

The above model assumes a 30% interest rate and hence penalises the Brepack as a result of its higher capital cost. However this high capital cost is not the only penalty.

The Brepack compaction process must take longer than the Cinva Ram as an additional hydraulic circuit must be pressurised and hence the compaction cost in terms of operator labour is higher as the productivity is reduced.

Table 6.6a Block production cost comparison (Sri Lanka)

Cost per Block	wet strength 1.4 MPa	wet strength 2.8 MPa
Cinva Ram 2 MPa compaction	£0.088	£0.106
Brepack 10 MPa compaction	£0.118 +33.5% over 2 MPa	£0.132 +24.1% over 2 MPa

Compaction at higher pressure produces denser blocks which use less cement but more soil. Hence the costs associated with the soil are increased. In the above model it was assumed that the soil would be available free of charge except for the labour cost involved in winning it. If a secondary cost must be paid for the soil, land rental or a purchase price, then the high pressure compaction route is further disadvantaged.

Table 6.6b Percentage breakdown of block costs (Sri Lanka).

Cost Parameter	Cinva Ram 1.4 MPa strength	Brepack 1.4 MPa strength	Cinva Ram 2.8 MPa strength	Brepack 2.8 MPa strength
cement	31.1%	16.6%	44.0%	25.8%
soil winning labour	39.2%	32.4%	31.3%	28.3%
pressing labour	26.4%	23.8%	22.0%	21.4%
machine depreciation	3.3%	27.2%	2.7%	24.5%
total	100%	100%	100%	100%

Table 6.6b shows the percentage cost breakdown for the four blocks produced. It can be seen that although the high pressure compaction machine does reduce the cement demand, both the machine depreciation and the labour costs counteract this benefit.

For these machines using this soil type, increasing the cement content appears to be more economic than increasing the compaction pressure. Even if the life of the high pressure machine is doubled high pressure compaction remains the more costly. However what is not clear from this analysis is the quality of the final blocks. Although both machines should produce blocks with the same wet compressive strength, their densities will be different. Ultimate bearing strength when wet is not the only valid measure of performance but the most expedient to test and numerically quantify. The blocks' durability may be different as a result of their differing density. Work is currently under way at The University of Warwick to investigate this.

The above analysis is only valid for the cement and labour rates quoted for Sri Lanka. In other areas the relative cost of cement and labour may be completely different. For example in rural Zimbabwe the cost of cement is increased to £3.50 per 50kg bag while the wage rate is reduced to £0.80 per man per day. The effect of this shown in tables 6.6c and 6.6d

Table 6.6c Block production cost comparison (rural Zimbabwe)

Cost per Block	wet strength 1.4 MPa	wet strength 2.8 MPa
Cinva Ram 2 MPa compaction	£0.0479	£0.0698
Brepack 10 MPa compaction	£0.0701 +46.3% over 2 MPa	£0.0864 +23.8% over 2 MPa

It can be seen that even for a rural environment, where the daily wage rate is much lower than the cost of a bag of cement, high pressure compaction remains the more expensive option. For this case it is primarily the machine depreciation cost which dominates the analysis as the labour costs are greatly reduced.

Table 6.6d percentage breakdown of block costs (Zimbabwe)

Cost Parameter	Cinva Ram 1.4 MPa strength	Brepack 1.4 MPa strength	Cinva Ram 2.8 MPa strength	Brepack 2.8 MPa strength
cement	67.0%	32.9%	77.8%	45.9%
soil winning labour	15.8%	12.1%	10.4%	9.5%
pressing labour	11.1%	9.1%	7.6%	7.3%
machine depreciation	6.1%	45.9%	4.2%	37.3%
total	100%	100%	100%	100%

If the cement cost were increased to £6.00 per 50kg bag while the labour cost remained at the Sri Lankan value of £3.50 per man per day then the high pressure compaction route still remains the more expensive although the margin of difference is reduced (table 6.6e) to 18.8% and 8.5% for 1.4 MPa and 2.8MPa strength standards respectively.

Table 6.6e Block production cost comparison (if £6.00 per 50 kg of cement in Sri Lanka)

Cost per Block	wet strength 1.4 MPa	wet strength 2.8 MPa
Cinva Ram 2 MPa compaction	£0.1160	£0.1525
Brepack 10 MPa compaction	£0.1378 +18.8% over 2 MPa	£0.1655 + 8.5% over 2 MPa

The above analysis is not able to cover differences in the production efficiency and adaptability. Compaction to high pressure produces blocks which have a higher freshly demoulded (green) strength as a result of their higher density. This reduces the risk of block breakage during ejection and transportation to the curing area which has been reported by Lawson (Lawson, 1992, Ref No.22) to be as high as 50% in some extreme cases.

Moreover, because of the increased green block density the range of soil which can be used for production is larger for the high pressure machines. Green strength depends on the soil particle grading and the block density. If the green block density is reduced then for the same green strength or handleability the soil's clay content must be increased. i.e. high pressure compaction allows the use of soils with lower clay contents than those acceptable for low pressure compaction.

In conclusion, in most situations low pressure compaction will be more economic than high pressure compaction, provided that the block breakage rate is acceptably low, i.e. a moderate to high[5] clay content soil is used. If the cost of high pressure machines can be significantly reduced, while keeping the production rate similar to that of the low pressure machines, then high pressure compaction may prove to be more economic. Moreover if high pressure compaction is found to increase block durability then a small cost premium may be acceptable.

[5] Moderate and high clay contents within the acceptable clay content bounds of 10 - 30%.

APPENDICES

APPENDIX A: BIBLIOGRAPHY

APPENDIX B: *SOIL-A* PROFILE

APPENDIX C: EXPERIMENTAL METHOD

APPENDIX D: EXPERIMENTAL INSTRUMENTATION

APPENDIX A BIBLIOGRAPHY

1. Agarwal, A.: Mud, mud. The potential of earth-based materials for Third World housing. (London, International Institute for Environment and Development, 1981).

2. British Standards Institution: Methods of testing soils for civil engineering purposes, doc. BS1377 (London, BSI, 1991).

3. British Standard Institution: Methods of test for stabilised soil, doc. BS1924 (London, BSI, 1975).

4. Building Research Establishment.: Building in hot climates, A Selection of Overseas Building Notes. (London, H.M.S.O., 1980).

5. Carter, M. & Bentley, S.P.: Correlations of Soil Properties. (London, Pentech Press, 1991).

6. Craig, R.F.: Soil Mechanics, fourth ed. (Wokingham, Van Nostrand Rheinhold (UK) Co Ltd, 1987).

7. Das, B.M.: Fundamentals of Soil Dynamics. (New York, Elsevier Science Publishing Co, 1983).

8. Doat, P. et al. Construire en Terre. (Paris, CRATerre, 1979).

9. Handbook For Building Homes of Earth. Department of Housing and Urban Development, Office of International Affairs, Washington, D.C. 20410 (undated).

10. Ingles, O.G. & Metcalf, J.B.: Soil Stabilization Principles and Practice (London, Butterworths, 1972).

11. The Institution of Civil Engineers.: Ground Treatment by Deep Compaction. (London, ICE, 1976).

12. International Labour Office.: Small-scale Manufacture of Stabilised Soil Blocks. (Geneva, Switzerland, Technology Series, Technical Memorandum No.12, ILO Publications 1987).

13. Norton, J.: Building With Earth: A Handbook. (London, Intermediate Technology Publications Ltd, 1986).

14. O'Reilly, M.P. & Brown, S.F.: Cyclic Loading of Soils: from theory to design. (London, Blackie, 1991).

15. Spence, R.J.S. & Cook,D.J.: Building Materials in Developing Countries. (Chichester, Wiley, 1983).

16. Stulz, R.: Appropriate Building Materials, (London, U.K / SKAT. St.Gallen, Switzerland, Intermediate Technology Publications Ltd, 1983).

17. United Nations.: Soil-Cement its Use in Building, (United Nations Publication, Sales No.: 64.IV.6, 1964)

18. Volunteers in Technical Assistance: Making building blocks with the CINVA-RAM block press. (Mt. Rainier, Maryland, VITA, 1977).

19. Webb, D.J.T.: Stabilised Soil Building Blocks. (University of Newcastle-upon-Tyne, Ph.D. Thesis, 1988).

20. Webb, D.J.T.: The work of the Building Research Establishment, on lime stabilised soil blocks for Third World housing. (Paper presented at the First International Seminar on Lime and Other Alternative Cements, 9-11 December 1991, Stoneleigh, UK, held by the Intermediate Technology Development Group, Rugby, UK, 1991).

21. Williams, F.H.P. & Maclean, D.J.: The Compaction of Soil. A Study of the Performance of Plant, Road Research Technical Paper No. 17. (London, H.M.S.O., 1950).

22. Lawson, W. Personal communication. (1992).

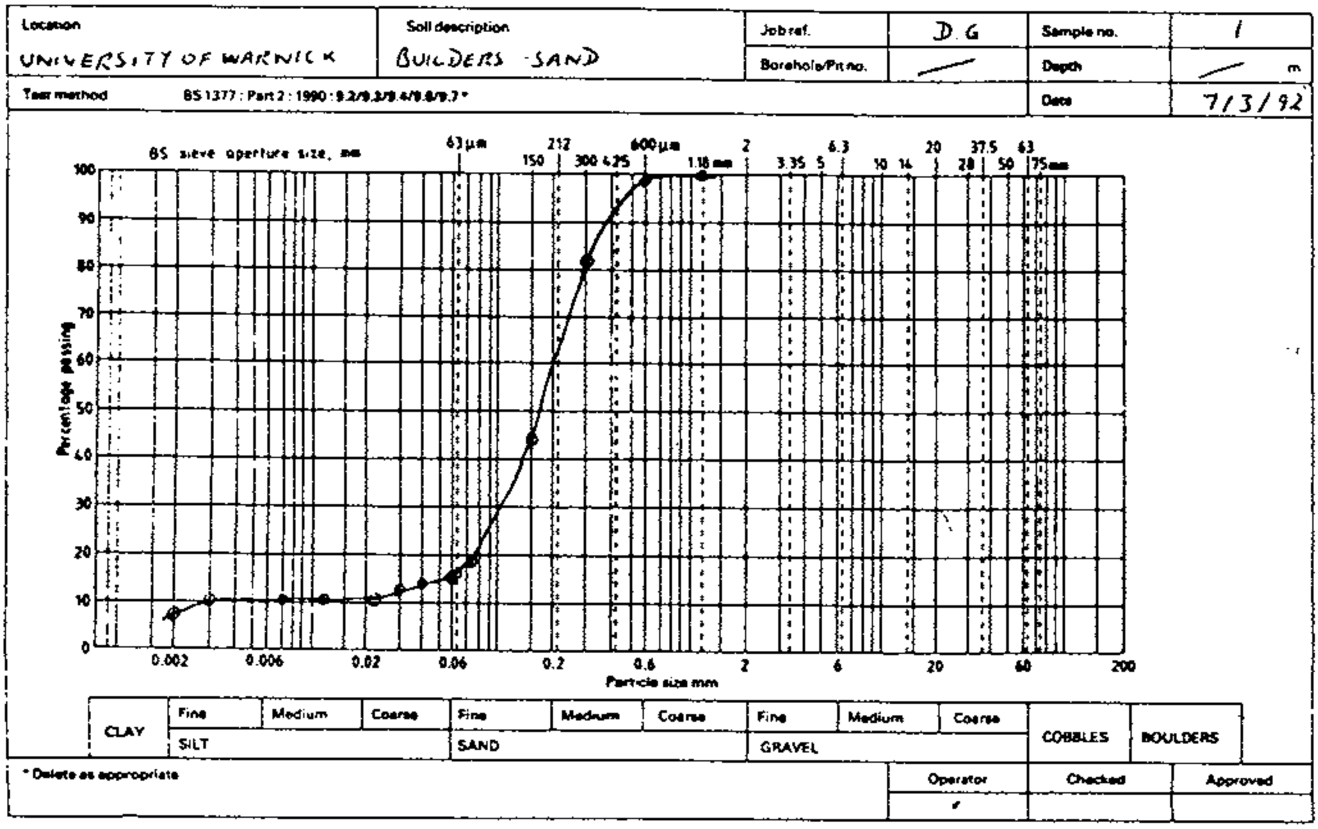
APPENDIX B SOIL-A DETAILS

Soil-A is an artificial soil produced at the University of Warwick by blending building sand with grade E kaoline powder. This soil was used for all experimentation to aid repeatability and allow a consistent soil composition throughout the course of the current research work. The soil was blended such that it fell within the ideal specification for soil-cement given by United Nations (1964, Ref No.17). This states that the optimum soil composition is; 75% sand, 25% silt and clay, of which more than 10% is clay.

Building Sand:

Grading:	Sand	84.2%
	Silt	8.8%
	Clay	7.0%

Grading curve:



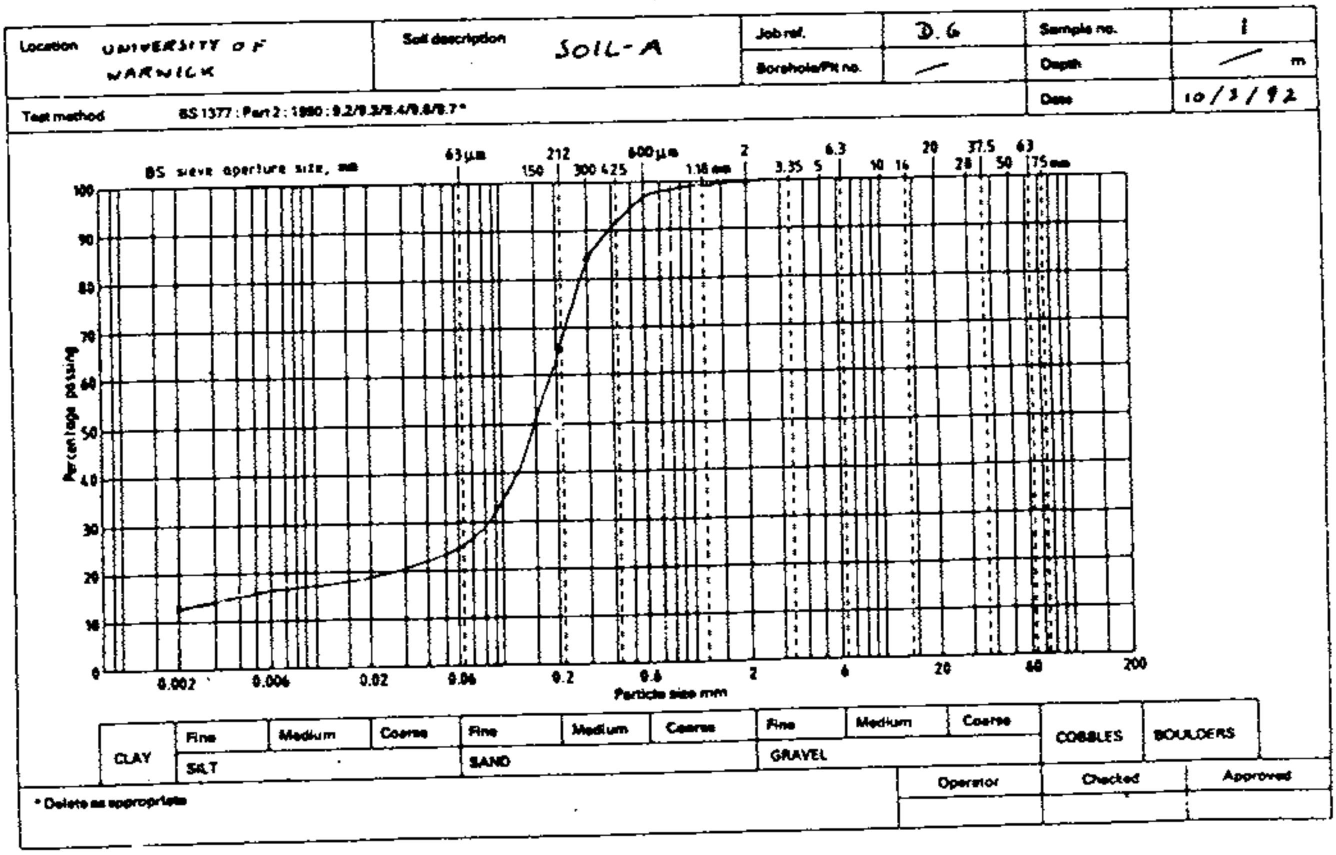
Kaoline Grade E powder:

Specific gravity	2.6
Specific Surface Area	8.0 M^2/g
Water soluble salts	0.15 %
SiO_2	50 %
Al_2O_3	35 %
Ph	5 ± 0.5

Soil-A mix proportions:	Building Sand	9 parts	(7.2 kg)
	Kaoline	1 parts	(0.8 kg)

Soil-A grading	76.5%	Sand
	8.0%	Silt
	15.5%	Clay

Soil-A grading curve:



APPENDIX C : EXPERIMENTAL METHOD

UNSTABILISED BLOCK PRODUCTION (290X140X100mm)

For each block a batch of *Soil-A* was manufactured and mixed for 5 minutes with distilled water to give a moisture content of 4% (for batch proportions see below). This batch was then left overnight to homogenise before remixing to 8% moisture content. All batch proportions were weighed to ±0.05g. All mixing was mechanical, using a large Hobart soil mixer.

The material to fill the mould was weighed out as three equal quantities into three plastic bags and sealed.

After oiling the mould with a release agent (engine oil), the soil was placed in the mould. The contents of each bag was lightly tamped before adding the next. The mould top was then placed on the soil and its height above the compression machine bed measured and recorded. A dial gauge was then positioned such that the block height during compaction could be measured.

The block was then compressed in 5 tonne force increments up to 40 tonnes. After each force increment the applied force was held constant long enough for the block height and both LVDT readings to stabilise and be recorded (typically 1 minute). The block was then decompressed in a similar manner.

The compressed block was ejected from the mould by pressing the mould walls down over the lower piston. The green block was then transferred to a wooden base plate and its final dimensions recorded.

UNSTABILISED MIX PROPORTIONS

7.200kg builders sand (0.5% moisture content)
0.800kg kaoline grade E powder (0.7% moisture content)
0.277kg distilled water (for 4% homogenisation)
0.318kg distilled water (8% moisture content for compaction)

Mass of 8% moisture content *soil-A* for block compaction 8.532kg

STABILISED BLOCK PRODUCTION (290X140X100mm)

For stabilised block production the above method was used but after homogenisation at 4% moisture content, 0.398kg of cement and 0.350kg of distilled water were added.

On ejection from the mould the green blocks were transferred to a plastic bag containing a damp tissue and sealed. The blocks were then left to cure for six days before immersion in water for the final 24 hours. Curing temperature was 22-24°C. After seven days the blocks were tested for wet compressive strength. Both the upper and lower block faces were capped with fibre board before compressive strength testing in a Denison concrete testing machine.

STABILISED SOIL-CEMENT CYLINDERS (ϕ 50mm, height 100mm)

The method given below is a copy of that used during manufacture. Six days after compaction each sample was soaked for 24 hours. On the seventh day after compaction the samples were capped with fibre board and tested for compressive strength in a Denison concrete testing machine. Although the Denison machine was operating below its range of grade 1 calibration (± 1 %) it had been recently recalibrated by an authorised testing house who indicated that the largest error given by the machine would be ± 3 % of the recorded value.

1. Measure out all ingredients for required batch. The water should be weighed into a pre-wetted container to allow for the quantity which remains in the container.

2. Place the 4% homogenised soil in the mixer. Sprinkle the cement onto the soil and note the time. Mix for 2 to 3 minutes or until the mixture looks uniform in colour, place a large plastic bag around the top of the mixer's bowl to reduce the evaporation of the water. Sprinkle in the weighed water, try not to pour the water onto the sides or the mixing paddle. Mix for a further 3 to 4 minutes or at least until the mixture looks uniform.

3. Weigh out 453.6g of the mixture, leaving the mixing bowl covered with a large plastic bag to reduce the moisture loss.

4. Oil the mould with the release oil and assemble for filling. Place approx one third of the mixture into the mould using the paper funnel. Take care not to spill any soil. Tamp the soil down with the steel bar. Repeat this for the next two thirds of the mix. Place the mould piston on top and try to centralise the main body between the end pistons.

5. Place the mould, red ring down, in the centre of the compression machine plated and compress to the required force twice (forces listed below).

6. Lift off the compression machine. Remove top and bottom pistons. Place the ejection ram in the base and the collars on top of the mould. Lower the compression machine to eject the sample. If you try and rotate the mould while compressing, it will be apparent when the sample has been ejected far enough for final removal by hand. Note the time that the sample was ejected.

7. Write the identification number on the top face and place it into a plastic bag. Repeat the above for the next two cylinders and then weigh and measure the length of each. Finally place them inside a plastic bag with one moist tissue and seal the bag.

Compaction forces:

1 MPa	= 1.960 kN	use 2.0 kN	(1.018 MPa)
2 MPa	= 3.93 kN	use 4.0 kN	(2.037 MPa)
4 MPa	= 7.85 kN	use 8.0 kN	(4.074 MPa)
6 MPa	= 11.78 kN	use 12.0 kN	(6.112 MPa)
8 MPa	= 15.71 kN	use 16.0 kN	(8.149 MPa)
10 MPa	= 19.63 kN	use 20.0 kN	(10.186 MPa)

ORDER:

Start with 10 MPa compression and 11% cement. Follow with 10 MPa 9% etc. This should minimise confusion with the compression machine!

BATCH PROPORTIONS:

All cylinders are to be compacted at eight percent water content and a dry mass of soil + cement of 420g, giving a fill mass per cylinder of 453.6g .

The figures below relate to batch mass measures. Each batch should contain enough material to make 3 cylinders with some material left over, approx 225g.

It is important to remember to note the time when the cement is added to the 4% moisture content soil.

BATCH TO MIX UP AND STAND OVERNIGHT:

(measure all mass to ± 0.05g)

9kg	lab-dry soil
1kg	kaolin from bag
0.347kg	distilled water

3% CEMENT BATCH:

1484.4g	4% homogenised soil
42.7g	cement
60.6g	distilled water

5% CEMENT BATCH:

1456.0g	4% homogenised soil
70g	cement
61.7g	distilled water

7% CEMENT BATCH:

1428.7g	4% homogenised soil
96.3g	cement
62.7g	distilled water

9% CEMENT BATCH:

1402.5g	4%homogenised soil
121.5g	cement
63.8g	distilled water

11% CEMENT BATCH:

1377.3g	4% homogenised soil
145.6g	cement
64.8g	distilled water

APPENDIX D : EXPERIMENTAL INSTRUMENTATION

THE LVDT TRANSDUCER

The LVDT transducer (see figures D1, D2 and D3) was designed to flush-mount in the mould walls. The main body of the transducer is machined from EN24T steel to form a circular spring. The thickness and shape of the spring are such that it will remain well inside the elastic region of the steel such that deflection is proportional to the applied load. The spring is deflected by a cylindrical piston mounted in a tubular guide. Both the outer faces of the tubular guide and of the cylindrical piston are flush with the spring body face and mould wall under conditions of no load.

The LVDT plunger is screwed into the rear spring boss. The LVDT body is clamped inside the transducer by the olive ring. Any deflection of the spring is sensed by the LVDT and converted into a voltage signal. The voltage output from the LVDT is fed into a conditioner and finally displayed on a digital voltmeter. The LVDT transducers and conditioners are both made by Schlumberger Industries and were supplied by RS Components Ltd, catalogue No 646-527 and No. 646-599 respectively. The Schlumberger Part numbers are LVDT SM1 and type OD3 911040 transducer conditioner.

Initially quite large hysteresis was observed on unloading. This was found to be caused by an air lock between the piston and the spring body. The design was subsequently modified by including a 1.5mm diameter vent hole. Two such transducers were manufactured and used during the experimentation.

Figures D4 and D5 show the calibration plots for transducer No.1 and No.2 respectively. Minor machining differences led to each unit having a different spring constant and hence a different gain. The gain of each unit was found to be constant over time and constant within normal laboratory temperatures. The zero offset was found to vary with time, typically 1mV per 30 minutes. The zero load voltage was recorded immediately before each experiment so that the zero offset could be determined, this offset was assumed to remain constant for the duration of each test (20 minutes). The transducers' hysteresis lead to a 0.1MPa over reading after four full cycles.

The pressure transmitted to the mould wall was found by entering the recorded voltage into the transducer equation:

$$y = mx + c$$

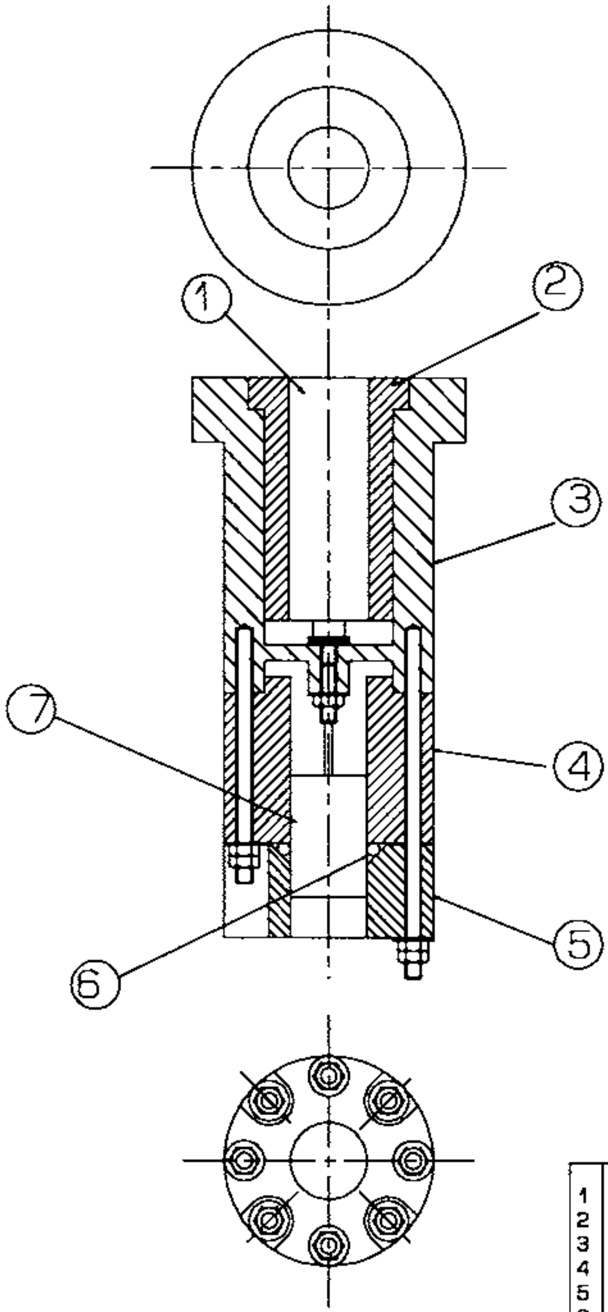
where y = transmitted pressure /MPa

m = transducer gain (found from calibration curves) /MPa/mV

x = recorded voltage /mV

c = zero offset correction (found by $c = -mx$ at zero load)

Figure D6 shows the possible locations for the transducers in the mould side wall.



	Part	Material
1	PISTON	SILVER STEEL
2	LINER	MILD STEEL
3	SPRING BOD	EN24T STEEL
4	TRANSD MTG	MILD STEEL
5	CLAMP RING	MILD STEEL
6	OLIVE	COPPER WIRE
7	LVDT	-

	Scale	NOT TO SCALE	Title SOIL CEMENT MOULD WALL	Drawn by	CEO
	Date	23-10-91	PRESSURE TRANSDUCER	Dwg No.	1

Figure D1 LVDT Transducer Assembly Drawing

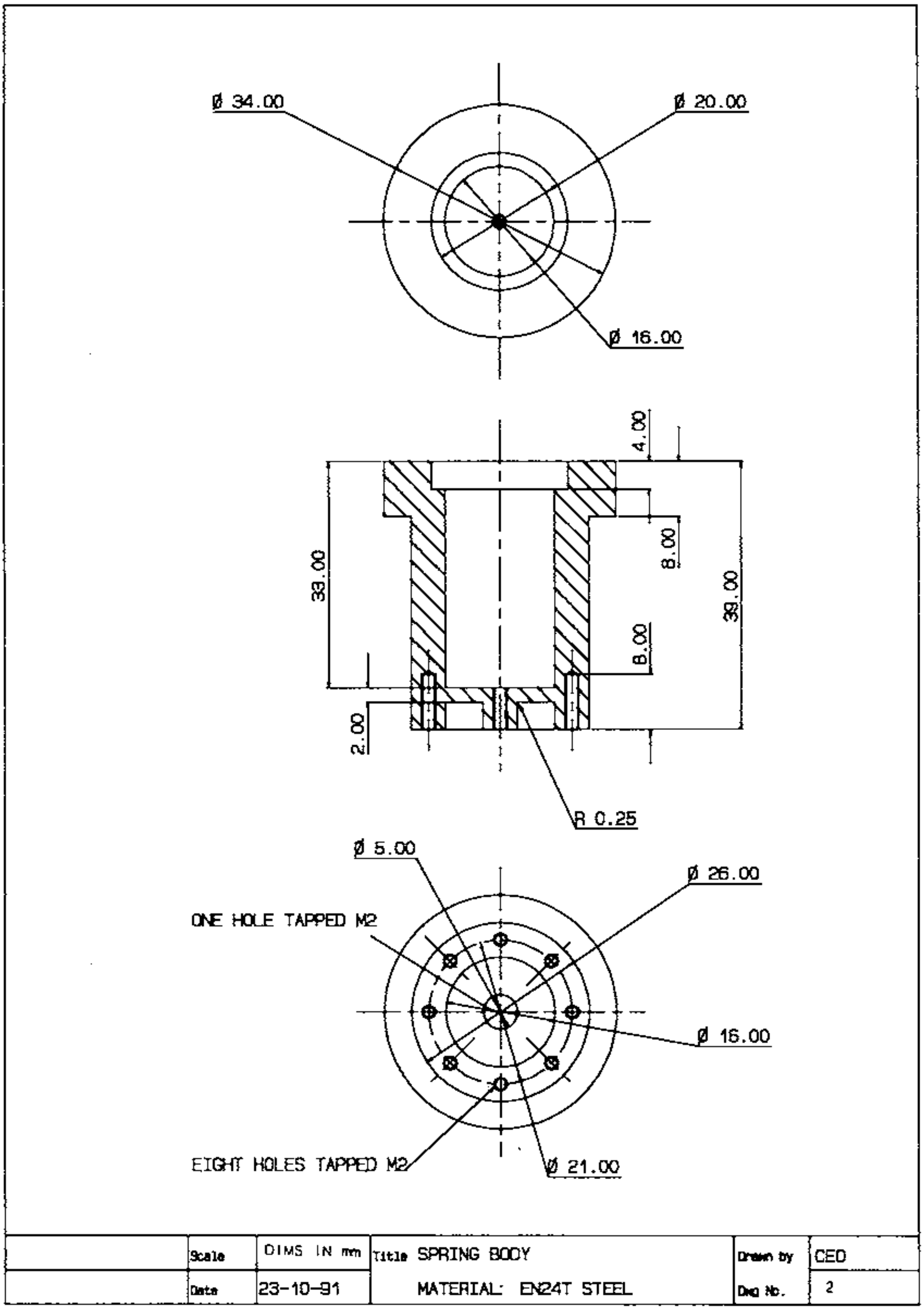


Figure D2 LVDT Transducer Spring Body

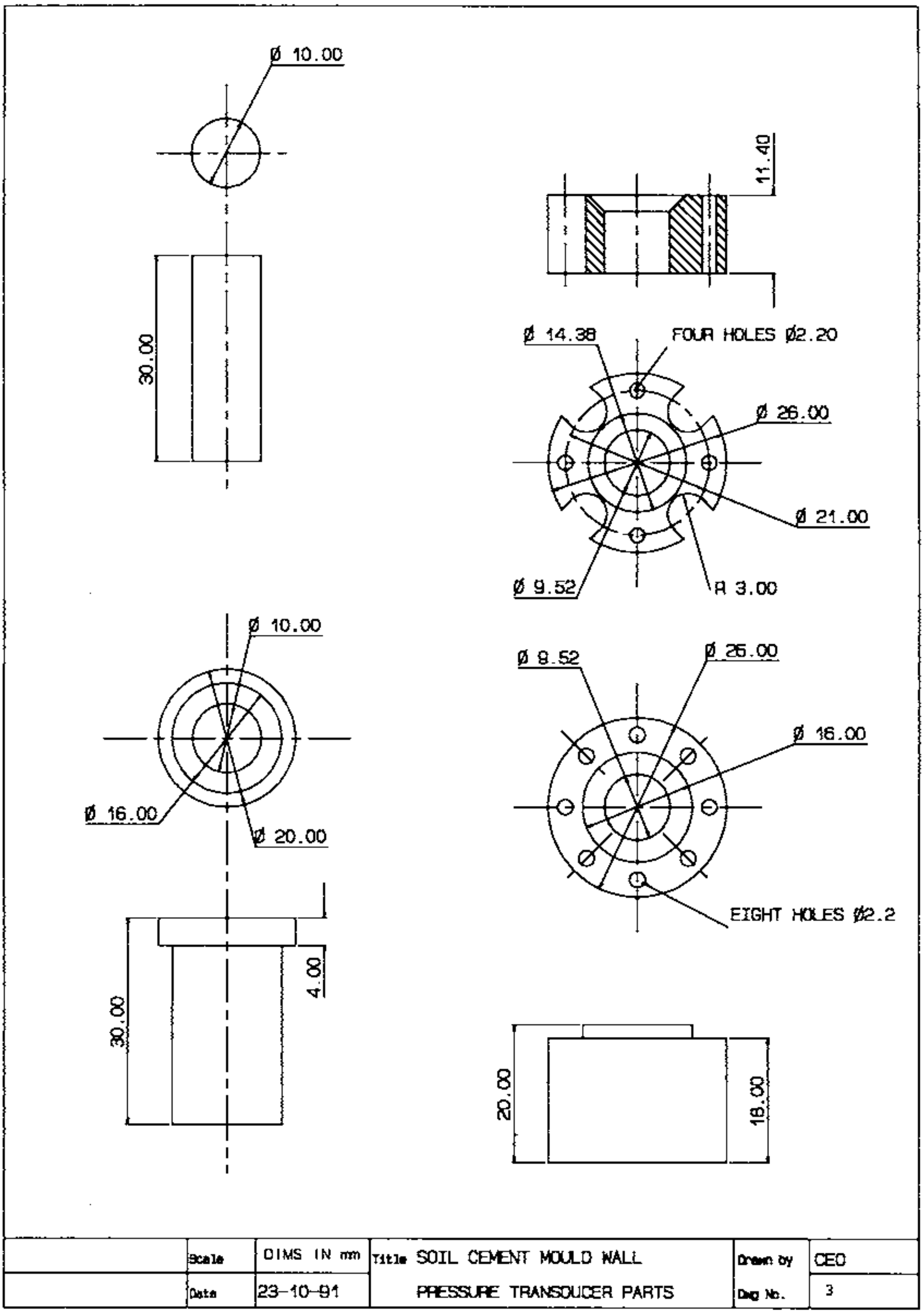


Figure D3 LVDT Transducer Parts

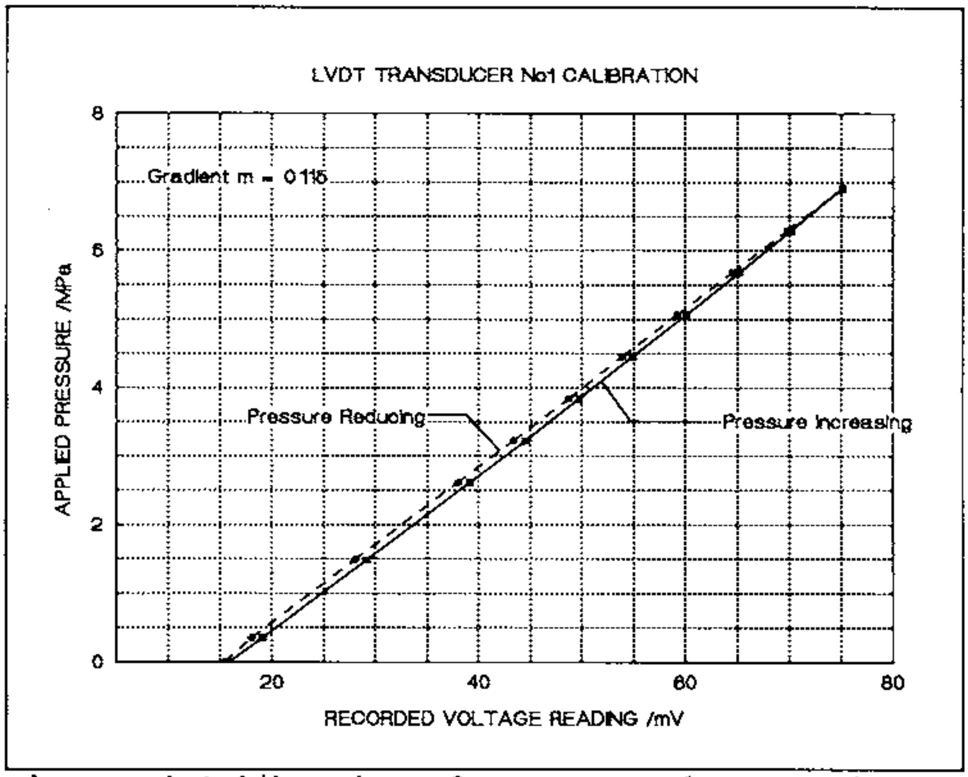


Figure D4 Calibration of LVDT transducer No.1

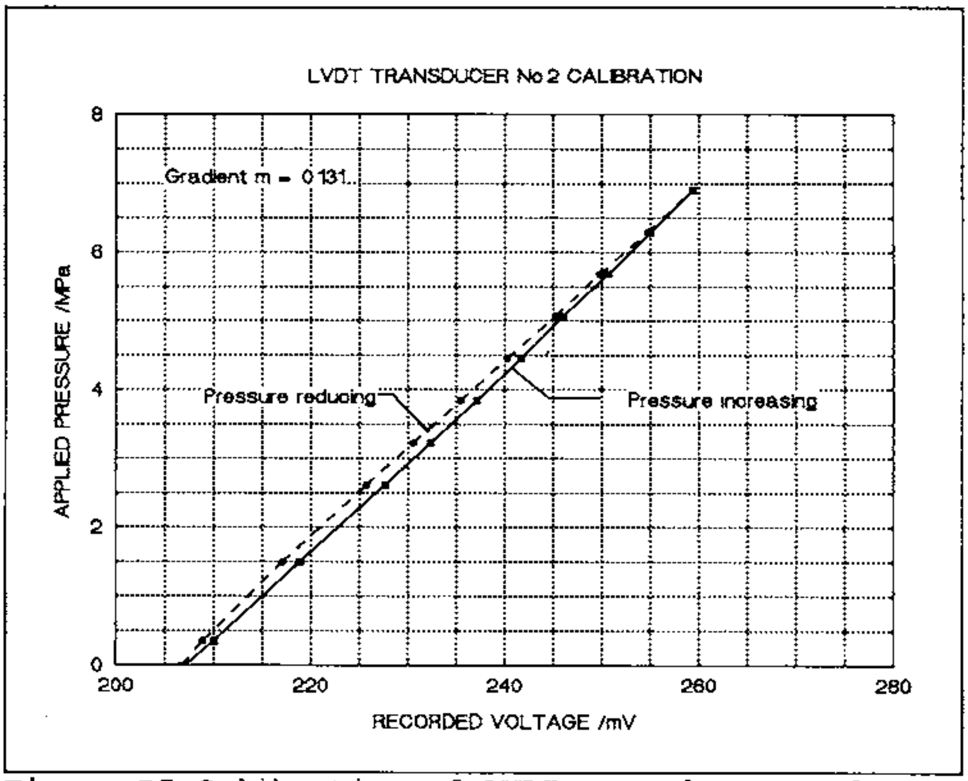


Figure D5 Calibration of LVDT transducer No.2

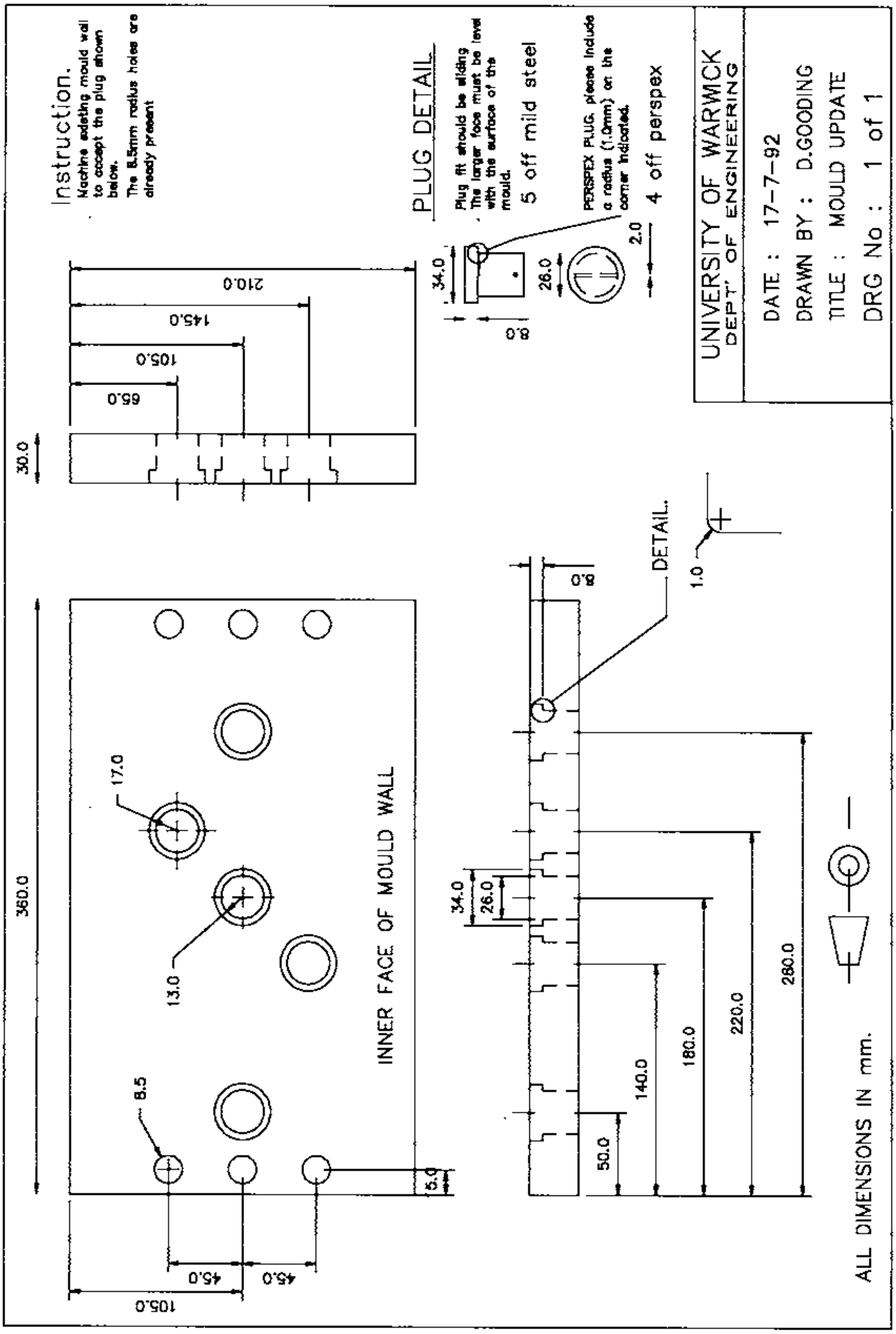


Figure D6 Transducer location sites in mould side wall

Testing Machine Details:

All wet compressive strength tests were made on a Denison Concrete test machine 7229/T91081, max load 100kN. Certified to grade 1 calibration at time of testing.

All soil-cement cylinders were compacted on a Monsanto Tensometer Type E (No. N120-79) with a 25kN load cell (No. 263). Certified to grade 1 calibration at time of testing.

All soil-cement blocks were compacted on an Amsler compression machine (No. ES1120), max load 40 tonnes. Certified to grade 1 calibration at time of testing.

DEVELOPMENT
TECHNOLOGY
UNIT

D
T
U

Working Paper No. 41

Algebraic Modelling of the Behaviour of Hydraulic Ram-Pumps

1994

T.H. Thomas

Development Technology Unit
Department of Engineering, University of Warwick
Coventry CV4 7AL UK
Tel: +44 (0) 1203 523122 Fax: (0) 1203 418922
email: dtu@eng.warwick.ac.uk

Working Paper No. 41

Algebraic Modelling of the Behaviour of Hydraulic Ram-Pumps

by Dr. T.H. Thomas

January 1994

Abstract:
The mathematical analysis of hydraulic ram-pumps began soon after their invention in the late 18th century. However simple models of adequate accuracy for use by system designers, pump manufacturers, installers and operators are still not available. This paper describes algebraic models of varying complexity for use by system and pump designers and by those involved in training installers and users. It argues that a pump plus drivepipe, rather than pump alone is the natural unit for modelling and for characterising performance in applications literature. Behaviour is shown to depend primarily upon three parameters. The first is λ, the ratio of peak drive flow (which depends upon tuning) to the pump's maximum flow with its impulse valve locked open. The second is μ, the ratio of peak drive flow to the 'Joukowski' flow just sufficient to achieve the system delivery head. The third is R, the ratio of delivery head to drive head. The analysis shows some of the trade-offs entailed in tuning, indicates the optimum choice of drivepipe and explains certain forms of malfunction observable in the field. Several 'rules of thumb' are derived. The paper also indicates areas where the greater precision of computer simulation over algebraic modelling is desirable.

CONTENTS

1. INTRODUCTION

This paper describes a number of analytic models, of varying degrees of complexity, for representing hydraulic ram pumps. It examines how such models can be used to optimise tuning, explain anomalous behaviour, aid system design and help in the design of better pumps.

The hydraulic ram pump, a water-powered water-lifting device, is of some antiquity, having been developed by Whitehurst, Montgolfier and others during the eighteenth century. Although its heyday was the late 19th century, there are over twenty manufacturers worldwide still producing machines. The ram pump performs the same role as a low-head high-flow turbine driving a high-head low-flow pump; it is however much simpler than a turbine-pump set as it contains only two moving parts, each a type of valve. The ram pump is mainly used to lift drinking water from small streams in locations where electricity is not available and engine-driven pumps would be costly to operate. The ram pump operates continuously but usually at a low power level (typically 50 to 200 watts), higher-powered pumping being more easily achieved by other means.

There is currently a revival of interest in ram pumps in developing countries and (due to changing tariff structures following privatisation of water supplies) in some industrialised countries too. Renewable energy devices in general are experiencing a come-back usually accompanied by design improvements resulting from new materials or new understanding. Ram pumps, not withstanding their simple construction, have also undergone some design changes affecting their cost and performance. More important to their wider use have been organisational changes (reducing the communication difficulties between manufacturers, installers and potential users). In developing countries population growth is causing expansion of rural settlements located higher than spring lines and intensification of agriculture through irrigation. Both of these increase the demand for water-powered pumping. The revival of interest has been reflected in several new publications (SKAT, Jeffery 1992, Knol 1992) and some transfer of manufacture to S. America and Africa from other continents.

The modelling of ram pump behaviour also has a considerable history (e.g. Eytelwein 1805, Lorenz 1910, O'Brien & Gosling 1933, Krol 1951, Rennie & Bunt 1981, Glover & Boldy 1990), although there is little evidence of such improvements in understanding affecting pump design or installation practice in the past. Two fairly recent developments have facilitated the devising of better models - improvements in instrumentation have made it easier to observe the complex movement of shock waves in actual systems, and the dramatic drop in computation costs have permitted the use of simulation models using short time steps (e.g. 0.1 millisec).

This paper arises from the work of the Development Technology Unit at Warwick University over nearly ten years on the identification of social and technical constraints to ram pump use, on the development of low-cost pumps for local manufacture in areas of use and on the transfer of system-installation skills into ten developing countries. Out of that work the need for, and the limitations of, modelling have become clearer.

2. REPRESENTING SYSTEM PERFORMANCE

The key components of a ram-pump system are shown in Figure 1. All of these have properties that affect system performance. There are some complex interactions between these components, so that little meaning that can be attached to the "performance" of the pump in isolation. When analysing the operation of a more conventional motorised pump it is possible to characterise the pump and the rest of the hydraulic circuit separately and to then combine these characterisations. The behaviour of a ram-pump is so strongly influenced by the nature of its drive pipe that this sort of analytic separation of system parts is not helpful.

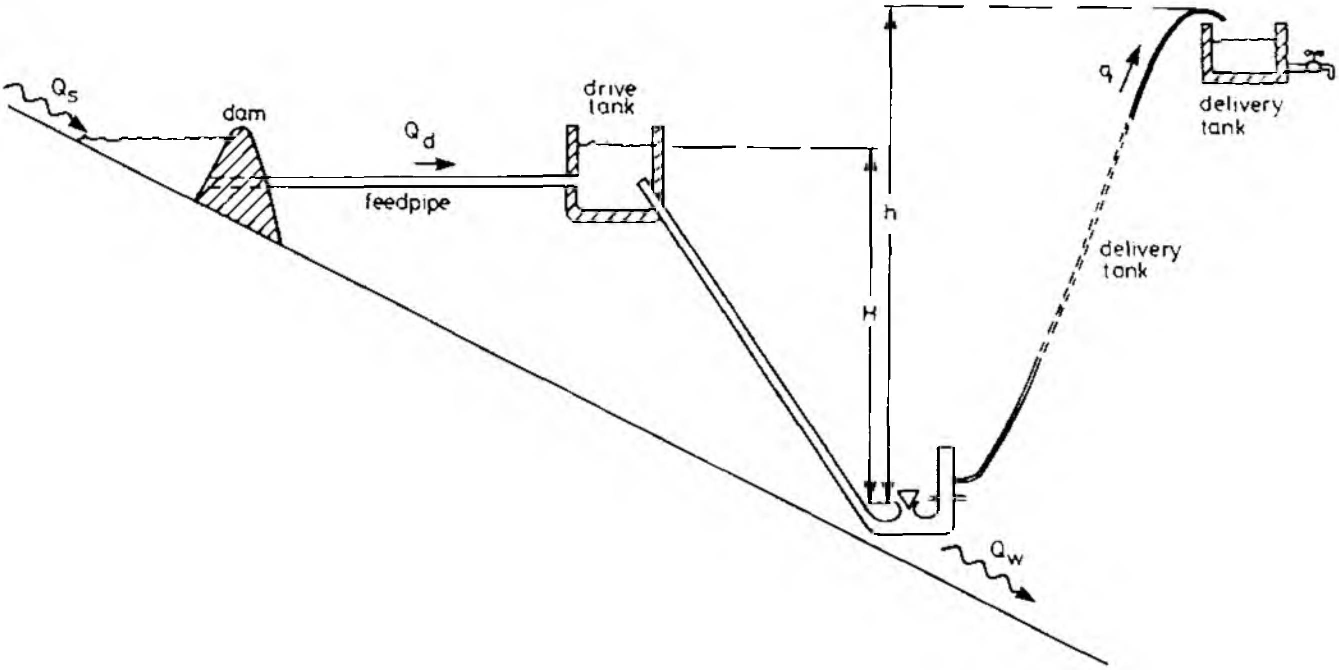


Fig. 1 Components of a ram-pump system

The components of the system have the following general characteristics that influence overall performance.

Feed pipe .. this has a *head loss* (dependent upon its length, diameter, roughness and the mean flow Q_d) which can be readily calculated independently of other system components.

Drive tank .. this is usually of sufficiently large surface area that it has negligible effect on system output and once sized can be left out of any further analysis.

Drive pipe ... this has a *drop* (H) that enters into even the crudest of performance equations, a *slope* (S) that primarily determines the water acceleration during the acceleration phase of pumping, an overall *friction factor* (relating friction head loss to instantaneous water flowrate) and an *area* (A) that relates velocity to flowrate.

Pump .... this has a *tuning control* that determines the driveflow Q_{vc} at the onset of impulse valve closure, an *impulse valve* whose inertia and fluid drag determines speed of closure once initiated and whose aperture when open detemines friction and kinetic energy losses during the acceleration phase, a *delivery valve* whose friction when open and whose inertia when opening and closing help determine pumping efficiency and finally a *flow-smoothing device* (e.g. pressure vessel) which if adequately sized has little effect on system performance.

Delivery ...... this comprises a *delivery pipe* carrying a steady flow (q) whose friction head loss is readily and separately calculable using standard formulae, and a *storage tank* which once sized (e.g. for 12 hours storage) has little effect on mean system performance. The *delivery height* (h) of course determines the outlet pressure at the pump.

When we come to characterise the performance of a system having a given site, pump, drive pipe etc. we normally treat the following as output (dependent) variables:

(i) delivery flow q

(ii) mean driveflow Q_d, taken as an average over many cycles

(iii) beatrate/(cadence) b, pump cycles per second
(iv) efficiency η, of the pump alone (not an easily defined entity), of the pump plus drive pipe or of the whole system. Efficiency is treated as a known constant (independent variable) in crude models and as a dependent variable in more elaborate models.

The normally determinable independent variables are
(i) delivery head h, or a higher head h' that includes friction head loss in the delivery pipe
(ii) drive head H, usually measured from the drive tank and hence already allowing for feedpipe friction head loss
(iii) drive pipe effective slope S, defined as drive head H divided by drive pipe length L (S has a somewhat more direct influence on performance than L)
(iv) pump setting Q_{cc}, the flow at which the impulse valve has been tuned to commence closing; this in combination with other variables determines the peak driveflow Q_p occurring during each cycle (a key variable in most analytic models)
(v) drive pipe area A
(vi) drive pipe type, and hence firstly its friction coefficient and secondly its wall stiffness that determines the velocity of sound in the water within it
(vii) pump type, and hence its internal frictions, exhaust velocity for a given driveflow and impulse valve-closure speed.

We have thus a plethora of independent variables whose number is tolerable for entry to a computerised model but is intolerably high if we intend to prepare graphs or tables to be used by a system designer in the field. (Indeed for a designer even the heads H and h are iterative variables since system design includes selecting between alternative water sources and pumphouse sites).

The list of variables above does not include feed pipe or delivery pipe characteristics and for the rest of the paper these will be assumed to be reflected in the values used for drive head H and effective delivery head h'. In practice these pipes will be sized, taking into account their probably large contribution to system cost, to give hydraulic 'efficiencies' of around 95% or even 90% for each: thus H might only be 90% of the drop from source to pumphouse and $h' = h/0.90$. In all our models we will be representing only that part of the system between the entry to the drive pipe and the entry to the delivery pipe.

Of the seven independent variables listed earlier, we might combine the last three for purposes of display output characteristics, producing graphs or tables for a particular combination of pump and drive pipe type (e.g. pump XYZ when used with a 110 mm OD, 10 bar, PVC drive pipe). This leaves for independent variables, two more than can be handled by a single readily readable graph or table. It is useful therefore to identify the degree of influence of these input variables upon the main output variables. Table 1 shows such an influence chart. The entries in the chart are based on experienced estimates of (the modulus of) sensitivity $|E|$

$$\text{where } \left|E_{yx}\right| = \frac{|\partial y / \partial x|}{y / x}$$

with y being regarded as the dependent variable. To emphasise the sensitivities, delivery head h' and head ratio $R = h'/H$ are used rather than h' and H.

Sensitivities $\left|E_{yx}\right|$ of each output variable to each input variable (acting alone) are allocated into four categories:

H = high sensitivity	$\|E\| > 0.8$	over most of both variables' ranges
M = medium sensitivity	$0.3 < \|E\| < 0.8$	" " " "
L = low sensitivity	$0.1 < \|E\| < 0.3$	" " " "
VL = very low sensitivity	$\|E\| < 0.1$	" " " "

Table 1: Influence of Independent Variables on Performance

Input variable (x)	Output Variable (y): Delivery flow q	Driveflow Q_d	Beatrate b	Efficiency η
Pump setting Q_{cc}	H	H	H	M
Head ratio R	H	VL	L	L
Delivery head h'	M	L	VL	M
Drivepipe slope S	VL	L	H	VL

Of these output variables, beatrate b is of no fundamental importance but may be used as a tuning indicator by operators. Mean driveflow Q_d may be constrained by the available source flow Q_s and hence should be known. Delivery flow is of primary interest to the system designer and knowledge of efficiency is useful when choosing between models of pumps. Of the independent variables, pump setting Q_{cc} and head ratio R are the main determinants of behaviour, with delivery head h' having a strong influence near the top of its range. Figure 2 illustrates the form of the relationship between inputs Q_{cc} and R and the outputs q and Q_d.

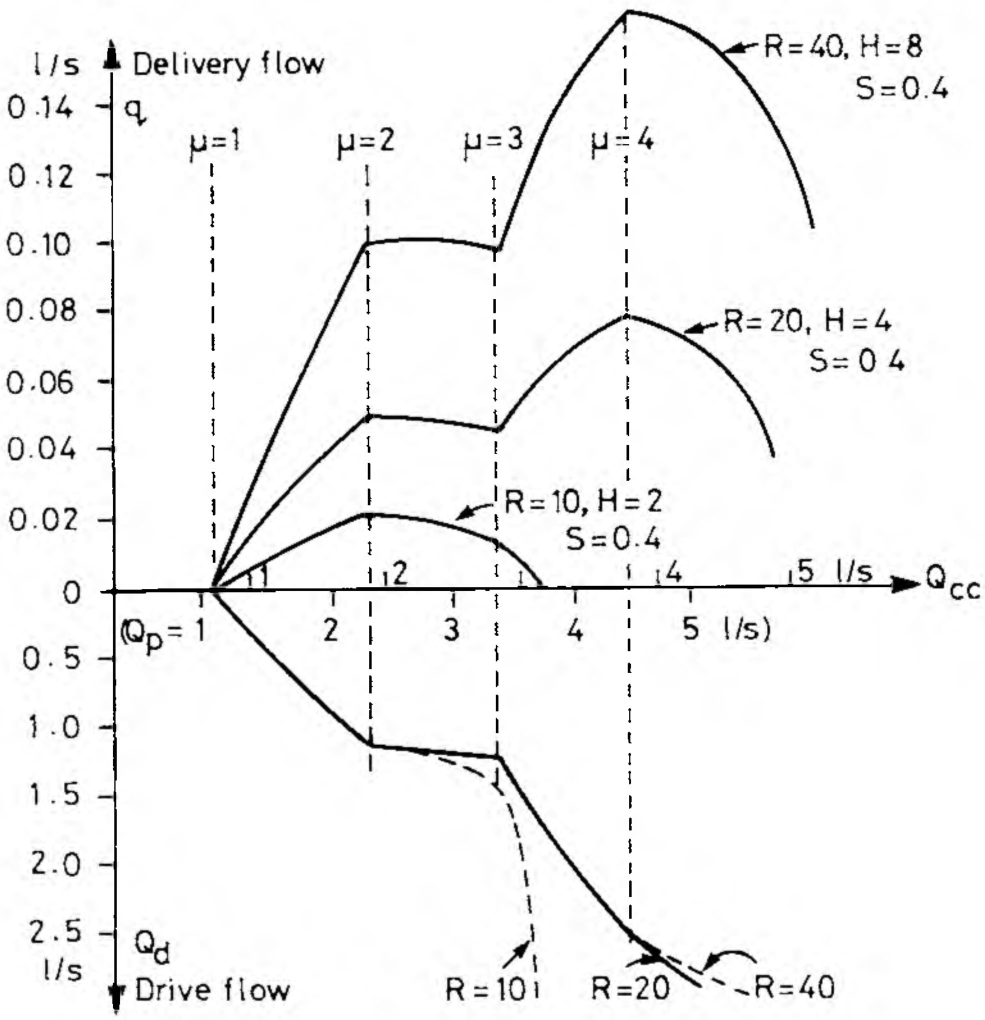


Fig. 2 Characteristic for pump type M8 with 50 mm drive pipe when tuning (Q_{cc}) is varied (delivery head of h' = 80m, drive pipe slope S = 0.4, drive head H = 2, 4 or 8 m)

3. MODELS

3.1 Algebraic and Simulation Models Compared

Algebraic models, invariable entailing the use of approximations to keep them manageable, are suited to application by 'hand' or by small computer to a range of activities - education, system design, pump selection and pump design. Because it is difficult to solve some of the equations, such models are often employed iteratively to indicate the performance, for example of a guessed-at system design, and how it might be improved by a change in parameters.

Users of ram-pumps are not, generally specialist engineers. Some installers have little education and cannot understand any algebraic notation. A major use of models therefore is to prepare design information even at the level of "rules of thumb". In fact three levels of model complexity can usefully be distinguished. Simple models are those operable by (some) water technicians. Standard models are usable by machine or system design engineers, by manufacturers to prepare charts and tables and for training demonstrations. Research models are primarily for pump designers. The Third World bias in ram-pump usage makes it particularly difficult to communicate and support models in the form of computer software.

Time-step (or-event-step) simulation models, usually based on the method of characteristics to represent transient flows in the drive pipe, have been developed for ram pumps (Glover). They are powerful but so costly of computing power that they are primarily useful for aiding pump design or explaining system phenomena rather than calculating routine performance graphs. Like all numerical techniques, these simulations deal with the particular - the influence of a particular variable can only be explored by repeated simulation 'runs' in which different values of that variable are used. With small computers, even with 486 processors, time-step simulations of ram-pump systems run at only a small fraction (e.g. 1%) of real time.

This paper restricts itself to 'simple' and 'standard' algebraic models and their use.

3.2 Simple Models

The simplest model of a ram-pump employs the concept of power balance and efficiency to give:

output power = input power × efficiency

$$\rho g\, q\, h' = \rho g\, Q_d\, H \times \eta$$

$$q = \eta\, Q_d / R \qquad \text{where head-ratio } R = h'/H \qquad [1]$$

The efficiency η so defined varies widely with operating conditions, falling to zero if Q_d is too small or R is too large. Manufacturers tables are however usually based on an assumed *constant* efficiency and users are often advised of constraints outside which they should not stray. Equation [1] probably represents the highest level of mathematical complexity one can expect any field worker or water technician to handle.

A model that can be employed in the teaching of field staff is the 'lumped-system' one whereby the pump cycle is broken into two phases. During an acceleration phase the water in the drive pipe accelerates under the drive head H, that is at rate $a = gH/L = gS$. During the deceleration phase the water decelerates under a reverse head of $h' - H$ giving $a = g\,(H - h')/L = gS(1 - R)$. This model neglects friction or the kinetic energy in the exhaust water; it also treats the water in the drive pipe as incompressible and hence fails to predict the negative pressures during end-of-cycle rebound that are important for pump operation. However this lumped-system model has several simple virtues that are not seriously affected by the rather drastic approximations inherent in it.

The velocity profile that the model generates is shown in Figure 3 superimposed upon a more realistic one.

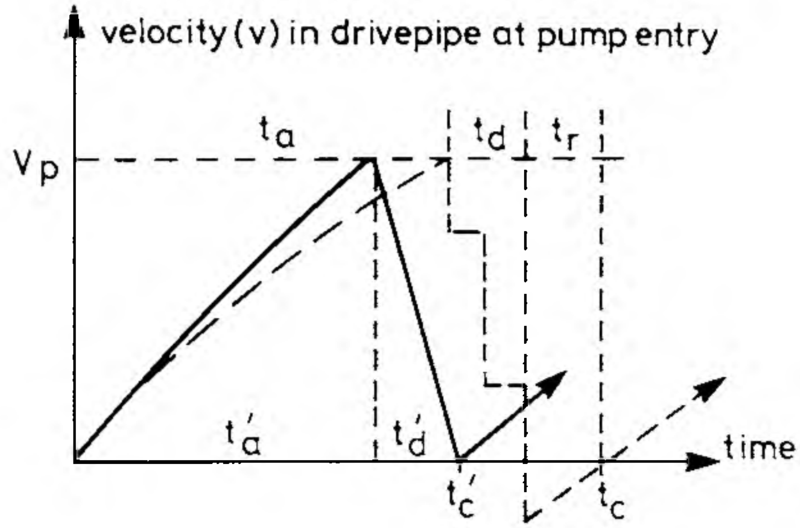


Fig. 3 Velocity profile predicted by frictionless lumped-system model
(Actual profile shown dashed)

The approximations result in predictions of acceleration phase time t_a a little shorter than reality t_a, a delivery phase of about the right duration ($t_d = t_d$) and no rebound time ($t_r = 0$). However in normal operation the drivepipe friction causing t_a to exceed t_a will be fairly small and the rebound phase may be short. Within about 10% accuracy this model correctly indicates the following.

(i) The cycle time is dominated by the acceleration phase.

(ii) For high head ratios ($R > 10$ so $t_c = t_a$),

peak velocity $v_p = g\,S\,t_c$ (t_c is cycle time) [2]
mean drive flowrate $Q_d = 0.5\ A\ v_p$

(iii) The shape of the driveflow-versus-time plot, Fig. 4 will be roughly a sawtooth (since $Q = A\ v$) for which the friction correction factor is

$$CF \equiv \frac{\text{average head loss with sawtooth flow pattern}}{\text{head loss at same } Q_d \text{ but flowing steadily}} = 2 \qquad [3]$$

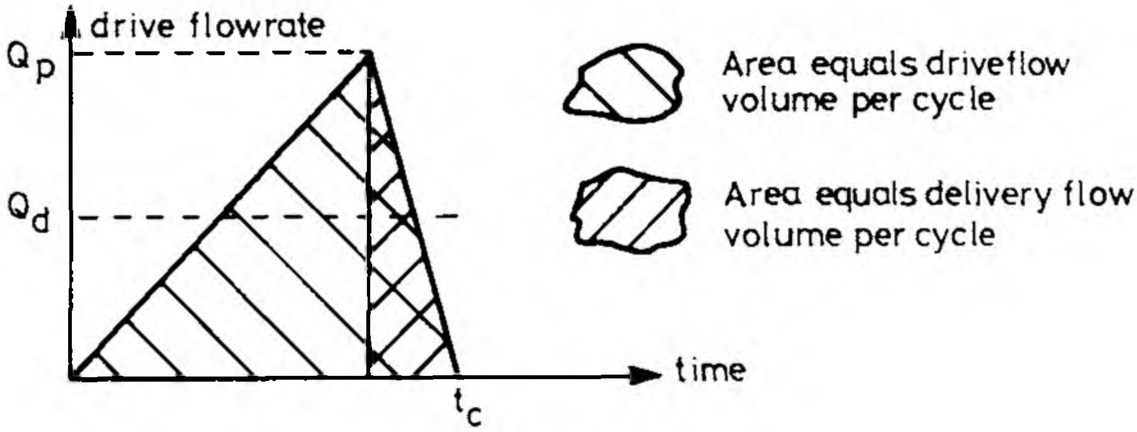


Fig. 4 Flowrate versus time predicted by frictionless lumped-system model

Thus drive pipe and exhaust kinetic energy losses can be first calculated assuming Q_d flows constantly and then doubled to allow for the waveform. Derived from this and the assumption that neither pipe friction loss nor waterflow kinetic energy should exceed, say, 10% of input energy gives two rules-of-thumb:

ROT.1 "Drive pipe size should be such that its headloss when carrying steady *flow Q_d does not exceed 5% of H".*

ROT.2 "The maximum height to which exhaust water sprays above the impulse valve (just before the latter closes) should not exceed 20% of H" (Height assumed proportional to KE, maximum flow rate Q_p assumed to be twice Q_d).

For any specific tuning, the pump manufacturer could describe the pump's friction and kinetic losses by specifying an equivalent length of (standard diameter) drive pipe L_{pe}. Drivepipe plus pump can be replaced by a simple but longer drivepipe (length $L + L_{pe}$), enabling a designer with access to pipe friction tables to apply:

ROT.3 "The headloss through drivepipe and pump together when carrying steady *flow Q_d should not exceed 10% of H".*

3.3 Acceleration Phase Models

The acceleration phase model above neglects various important effects, most noticeably factors which retard the gravitational acceleration. More accurate models were developed long ago but are not usually of a form convenient for a system designer.

The acceleration phase can be deemed to start when the water in the drive pipe starts to move downwards again (following its upwards recoil movement at the end of the previous cycle). Although shock waves may still be travelling up and down the water in the drive pipe, their amplitude should be small enough that the whole water column can be treated as having a single, initially zero, velocity.

It can readily be shown that the instantaneous acceleration of the drive pipe water satisfies

$$\frac{dv}{dt} = g\,S\,(1 - kv^2) \qquad [4]$$

So that acceleration terminates when

$$v = v_\infty = k^{-\frac{1}{2}} \qquad [5]$$

The retardation factor k reflects the headloss at drive pipe entry, plus that due to drive pipe friction, that caused by friction within the pump and the velocity head 'thrown away' in the waste water. The head loss due to all these factors can be expressed as a multiple of the velocity head in the pipe

$$H_{loss} = C \times \frac{v^2}{2g} \quad (\text{where } C = 2gH \times k) \qquad [6]$$

and $$C = C_1 + C_2 + C_3 + C_4 \qquad [7]$$

where C_1 is pipe inlet loss coefficient, typically 0.2 to 0.5

C_2 is pipe friction coefficient fL/D and f is typically 0.02

C_3 is pump loss coefficient, typically 1.5 but may be much higher

$C_4 = A^2/A_e^2$ where A is the pipe's cross-sectional area and A_e is the effective area of the discharge aperture of the impulse valve.

The first of these coefficients is usually negligible compared to the others, especially if the drive pipe inlet has a bell mouth. For practical purposes:

$$k = k_d + k_p \qquad [8]$$

where k_d (representing the drive pipe) equals $C_2/2gH$ and k_p (representing the pump) equals $(C_3 + C_4)/2gH$.

For the pump designer the minimisation of k_p is an objective, especially by maximising A_e to keep C_4 small and having large internal channels to keep C_3 small. k_p varies with pump tuning. For the system designer maximum economy comes from matching drive pipe to pump so that k_d is neither much greater than k_p nor much less. This fixing a pump (with impulse valve jammed open) onto a previously open-ended drive pipe should not reduce the flow through it by a factor more than 1.7 (implying $k_p < 2\,k_d$) or less than 1.22 (implying $k_p > k_d/2$). A rule-of-thumb that arises out of this analysis is:

ROT.4 "The area of the impulse valve's exhaust aperture should not be much smaller than the drive pipe's cross-sectional area".

Returning to equations [4] and [5], solving and relating all velocities to the maximum velocity attainable, gives

$$\text{normalised velocity, } \lambda = v / v_\infty = \frac{e^{2\tau} - 1}{e^{2\tau} + 1} \qquad [9]$$

where normalised time, $\tau = t / t_o$ (and where $\lambda = \lambda_p$ $\tau = \tau_m = t_a / t_o$)

and reference time, $t_o = v_\infty / gS$ (time to reach the velocity v_∞ were there no retarding influences)

The relationship is plotted in Figure 5 where the normalisation is also illustrated.

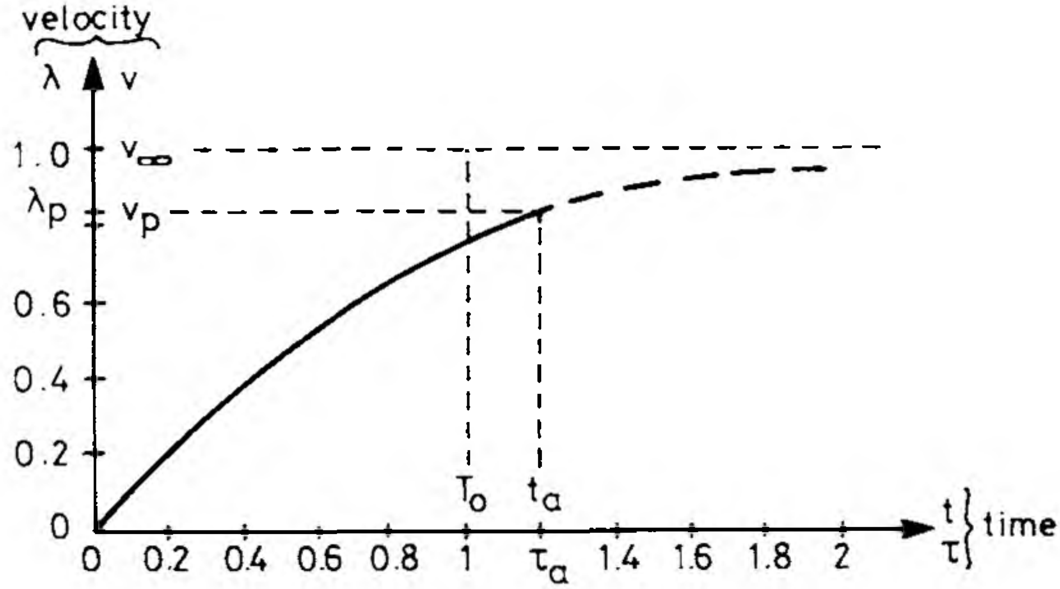


Fig. 5 Variation of water velocity during the acceleration phase
($\lambda = v / v_\infty$ and $\tau = t / T_o$ **are normalised values)**

This model is helpful discussing pump tuning. Tuning indirectly determines the peak velocity v_p at which acceleration terminates. As the purpose of the acceleration phase is to convert potential energy (height) into kinetic energy, we are interested in efficiency and power (rate of forming KE). Both of these are functions of v_p.

Figure 6 shows the efficiency and the normalised power of the acceleration phase as a whole, as functions of the normalised peak velocity at impulse valve closure ($\lambda_p = v_p/v_\infty = Q_p/Q_\infty$). The relevant algebraic expressions are:

$$\eta_a = -\lambda_p^2 / \ell n(1-\lambda_p^2)$$

$$P_a = \rho AL.\ \frac{gH}{2}.v_\infty \times \frac{\lambda_p^2}{\tau_\infty} \qquad [10]$$

$$t_a = \tau_a\, t_o = \tfrac{1}{2}\ \ell n\left(\frac{1+\lambda_p}{1-\lambda_p}\right) v_\infty / gS$$

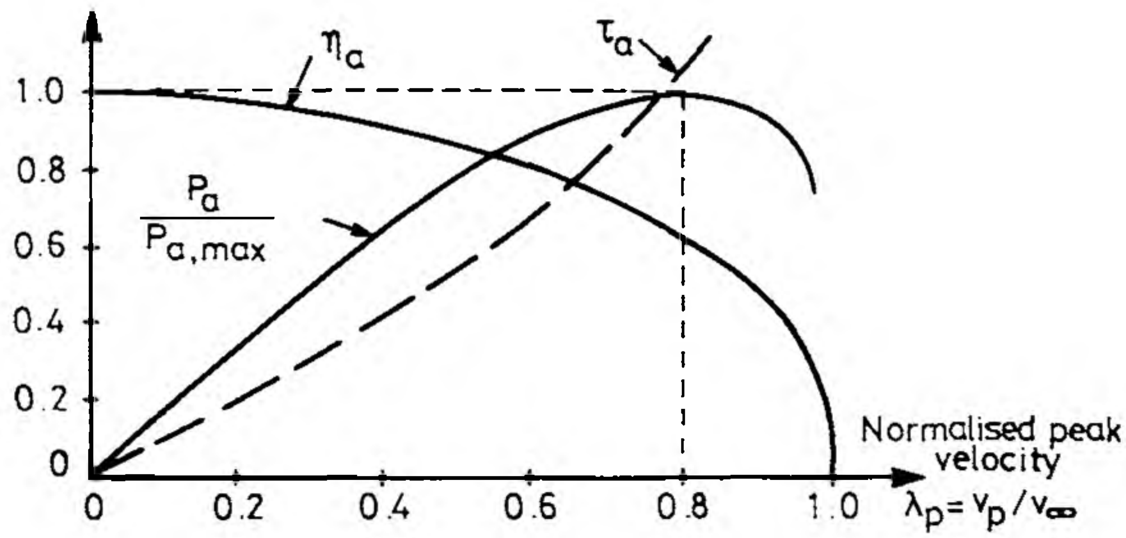


Fig. 6 Efficiency and power of the acceleration phase
(v_p is velocity at the end of the phase)

During the acceleration period the average velocity is about half the peak (final) velocity. The average and peak flows are similarly related. The acceleration time is close to $v_p/g\,S$ being actually:

$$t_a = \frac{v_p}{g\,S} \cdot \frac{\ln\{(1+\lambda_p)/(1-\lambda_p)\}}{2\lambda_p}$$

and the average velocity during acceleration is given by:

$$\frac{\bar{v}_a}{v_p} = \frac{\overline{Q_a}}{Q_p} = \frac{-\ln(1-\lambda_p^2)}{\lambda_p \ln\{(1+\lambda_p)/(1-\lambda_p\}}$$

Which tabulates as Table 2

Table 2 Mean acceleration flowrate and acceleration time

λ_p	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	(1.0)
$t_a \div g\,S/v_p$	1.00	1.00	1.01	1.03	1.06	1.10	1.16	1.24	1.37	1.64	(∞)
$\overline{Q}_a / \frac{1}{2}Q_p$	1.00	1.00	1.01	1.02	1.03	1.05	1.07	1.11	1.16	1.25	(2)
η_a	1.00	0.99	0.98	0.95	0.92	0.87	0.81	0.72	0.62	0.49	(0)

From Figure 6 it is obvious there is no purpose in setting λ higher than 0.8 (i.e. peak drivepipe flow equal to 80% of its maximum possible value). Maximum acceleration power occurs at $\lambda = 0.8$ but the efficiency is then low, $\eta = 63\%$. A much lower flow setting is usually preferable, for example $\lambda = 0.47$ will give 75% of maximum power but raise acceleration efficiency to $\eta = 90\%$. Although there are other factors to be discussed in the next section, Figure 6 illustrates the main trade-off between throughput and efficiency involved in tuning. It can also be used in reverse, in the sense that a designer could work back from a desired mean drive flow Q_d and desired acceleration efficiency to estimate the necessary limiting flow ($Q_\infty = A\ v_\infty$). From that the value of k in equations [4] to [8] is implied for which a suitable pipe and pump can be selected.

Refinements to this model are possible but do not greatly affect Figure 6. For example the previous recoil phase may have left the lower part of the drive pipe empty at the instant that forward motion of water recommences. This will have the effect of reducing the coefficients in equation [7] (e.g. $C_3 = 0$, $C_4 = 1$) until the water-air interface has passed back through the pump again. With normal recoils this lessening of friction at low velocities has negligible effect.

Representing, in fine detail, the flow behaviour during the last milliseconds of impulse valve closure is beyond the practical scope of algebraic models and is an area where simulation models give better insight. There is little evidence that the assumption of instantaneous valve closure will seriously falsify conclusions drawn from the acceleration model described above.

The acceleration phase model developed above does not readily yield a rule of thumb for pump tuning other than

ROT.5 "Pumps tuned to have very long cycle times (e.g. t_c over 3 seconds) will be very inefficient".

This rule is illustrated by Fig. 7: T_o is typically 0.5 to 1.5 seconds and t_a is typically 90% of measured cycle time t_c.

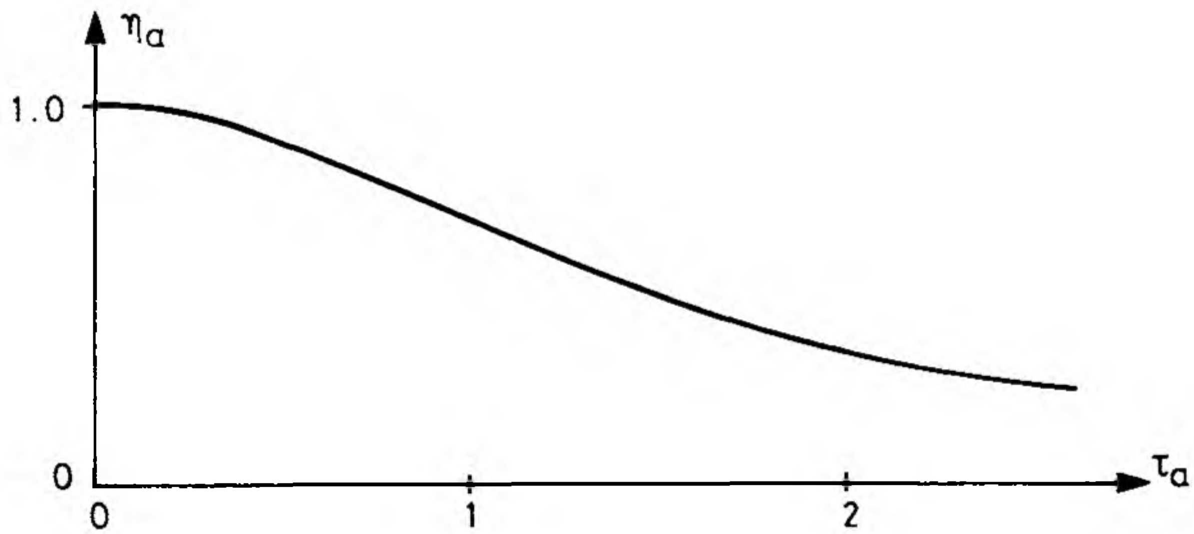


Fig. 7 Variation of acceleration efficiency with normalised acceleration time

3.4 Delivery Phase Model

During the acceleration phase the water in the drive pipe is building up its kinetic energy. It is accelerating because the impulse valve at the bottom of the pipe is open to atmosphere. During the delivery phase this impulse valve is closed and the flow is diverted through the delivery valve which it has forced open against a back head of h' (delivery head plus delivery-pipe headloss). The large net retarding force on the column of water in the drive pipe ($A \times \rho g(h'-H)$) causes it to decelerate rapidly. The detailed mechanism of this retardation is more complex than that assumed in Fig. 5 and comprises a sequence of shock waves travelling up and down the pipe. The overall effect of the delivery phase is to consume the kinetic energy formed during the preceding acceleration phase, using it to pump water. At the end of the delivery phase there is a small residue of kinetic energy that has not been used: this drives a water-recoil process namely the 'recoil phase'.

The transition between acceleration and delivery phases is very complex and must be idealised if algebraic models of tolerable complexity are to result. The sequence is as follows. Once the drive flow Q has exceeded the set threshold Q_{cc}, the impulse valve starts to close (conveniently visualised as a piston moving towards a hole smaller than itself). The flow continues increasing, beyond Q_{cc}, while the piston of the valve accelerates. As the piston gathers significant speed the fraction of the flow pressing *round* the piston decreases while the fraction *following* the piston increases (the latter is equal to piston area times piston velocity). The *velocity* of the former however increases, since the annulus through which it has to pass is rapidly getting thinner, causing the back pressure behind the piston to rise. The acceleration of the drivepipe water therefore diminishes and finally goes negative. By this time the piston is close to striking its stop and the first (annulus passing) fraction has a very high velocity manifest as a visible spurt of water above the valve. In the last millisecond of valve closure the exhaust flow falls to zero, the piston hits its stop and the pressure in the pump body rises precipitately to the Joukowski (or water-hammer) head h_J. Provided this is higher than the delivery head h', the delivery valve will be pushed open and the head will then fall back to $h' + h_L$ where h_L is the friction headloss through the delivery valve. The Joukowski head is thus maintained for a very short time whose duration depends on the velocity of sound, the distance from the impulse valve to the air-water interface above the delivery valve and the inertia of the delivery valve.

This sequence is portrayed in Figure 8 which represents approximately ten milliseconds of the change over between acceleration and delivery phases. The idealised model in Figure 8(b) is suitable for performance predictions but not for estimating the fatigue life of components. The boundary between acceleration and delivery phases has been taken as the instant the pump body pressure head has risen to the drive head H. Modern instrumentation provides ready confirmation of the 'actual' pressure transient but not of the flow transient: velocity sampling every 0.1 milliseconds is too expensive.

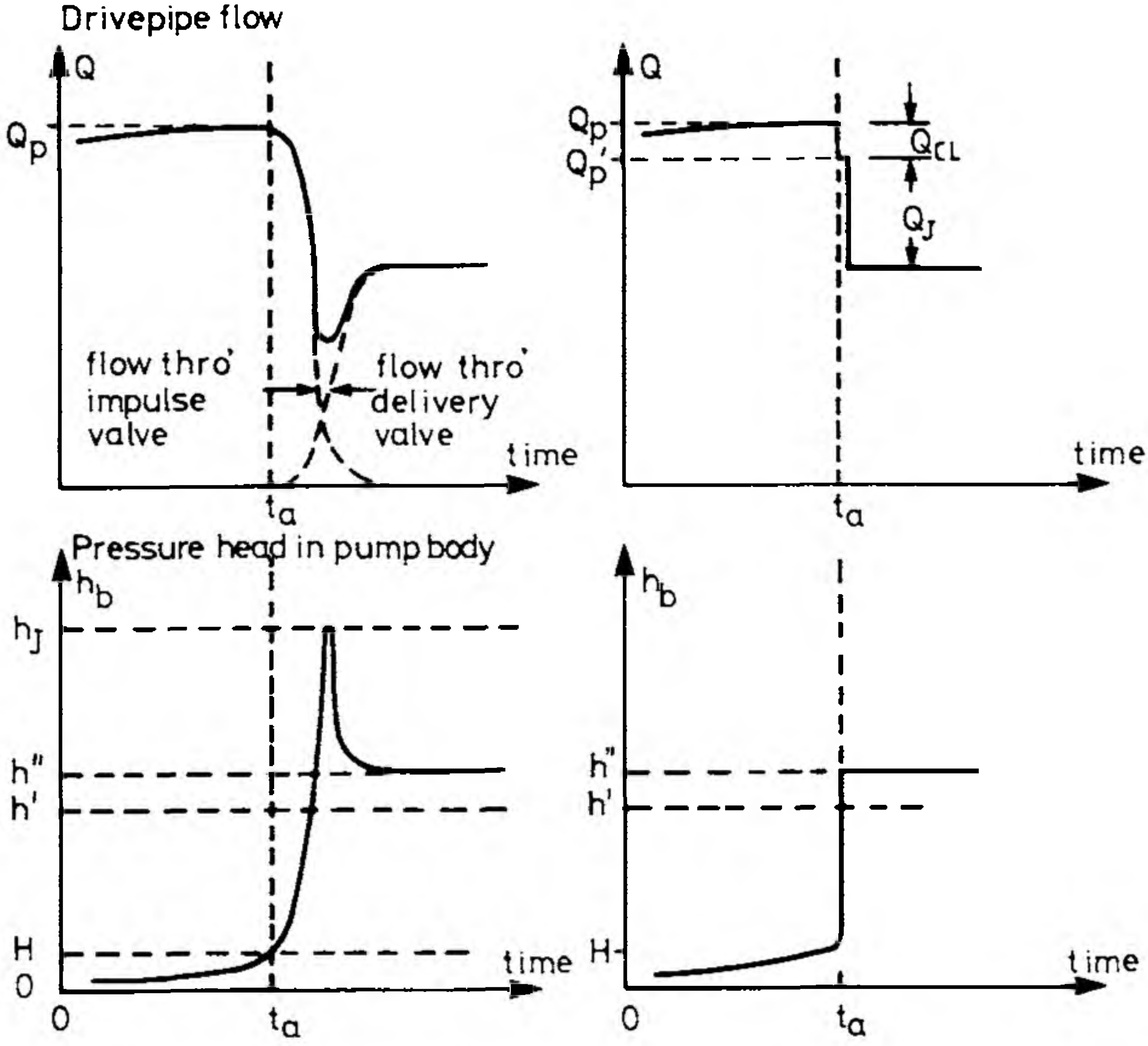
In Figure 8(b) the net effect of energy losses (e.g. lost piston KE) during the last instants of valve closure are replaced by a step reduction Q_{CL} in the drive pipe flow. The further drop Q_J corresponds to Joukowski velocity drop in a shock wave whose pressure differential is $\rho g \times (h'' - H)$

$$Q_J = A\, v_J \quad \text{and} \quad v_J = \frac{g}{c}\,(h'' - H) \qquad [11]$$

where c is the effective velocity of sound in the drive pipe and $h'' = h'$ + average delivery valve head loss. The quantities c and h'' will be discussed later. In our idealised model, which draws upon the work of O'Brien 1933 and Rennie 1980, the delivery phase starts with flow out of the drive pipe (i.e. into the pump body) dropping suddenly from Q_p' to $Q_p' - Q_J$ and the head rising suddenly from H to h''. This initiates a shock wave that travels up the drive pipe, at speed c to arrive at the drive tank after transit time

$$T = L/c \qquad [12]$$

The pressure distribution along the drive pipe is further assumed to have been the static distribution corresponding to steady conditions of no flow, i.e. head equal's zero at the drive tank surface and H at the pump entry. Effectively we are neglecting the effect of drive pipe friction during the delivery phase.



(a) **probable actual behaviour** (b) **idealised behaviour**

Fig. 8 Transition from acceleration phase to delivery phase
(over a few milliseconds around time t_a)

In the acceleration-phase model of section 3.2, behaviour depended on the peak drivepipe water velocity normalised with respect to its maximum possible value; $\lambda_p = v_p/v_\infty = Q_p/Q_\infty$. In the delivery-phase model behaviour depends on the velocity (and flow) normalised with respect to the Joukowski velocity defined in equation |11|. We therefore need to define another normalised flow

$\mu = Q/Q_J$ and its initial value $\mu_i = Q_p/Q_J$

Consider now the flow and pressure at a point in the drive pipe very close to the pump. Its form will be as in Figure 9.

In Figure 9 the shaded area is proportional to the water pumped per cycle. The dashed curve (A) is the behaviour we should expect if the delivery valve were to remain open; however to avoid reverse flow it is designed to close almost instantly. The dashed curve (B) is the projected behaviour following delivery valve closure. However the negative pressures so generated will usually take the absolute pressure below atmospheric, causing air to be drawn through the (high friction) snifter valve and shortly afterwards through the (low friction) impulse valve when it has reopened a little. Consequently trajectory (C) will be followed.

Figure 9(a) was drawn for $\mu_i = 3.3$ whose integer part is odd. Figure 9(b) shows the different behaviour when the integer part of μ_i is even.

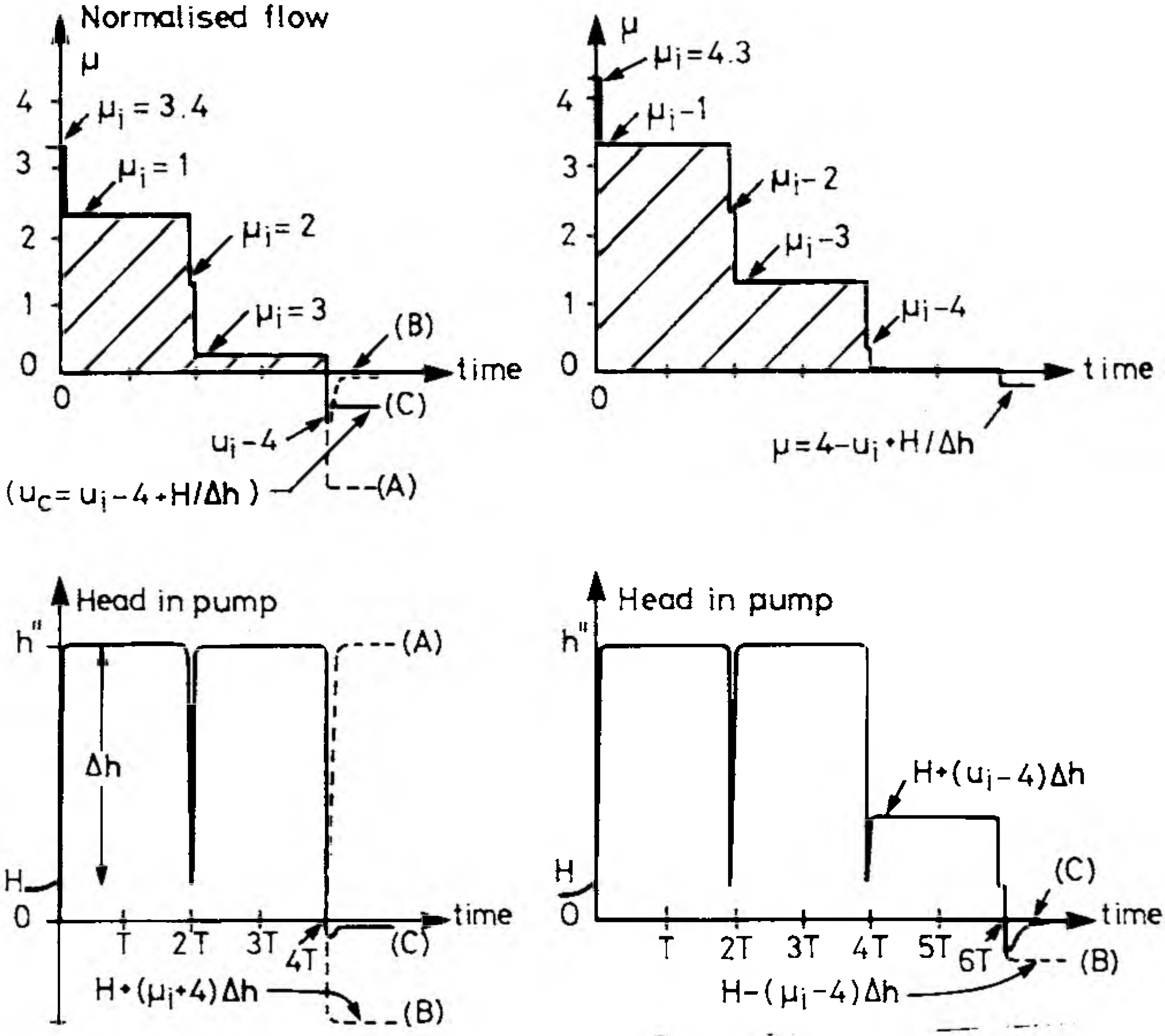


Fig. 9 Pressure and normalised flow at pump entrance

(A) delivery valve remains open

(B) delivery valve closes but impulse valve does not reopen

(C) actual: delivery closes and impulse reopens

Shaded area indicates normalised volume delivered per cycle

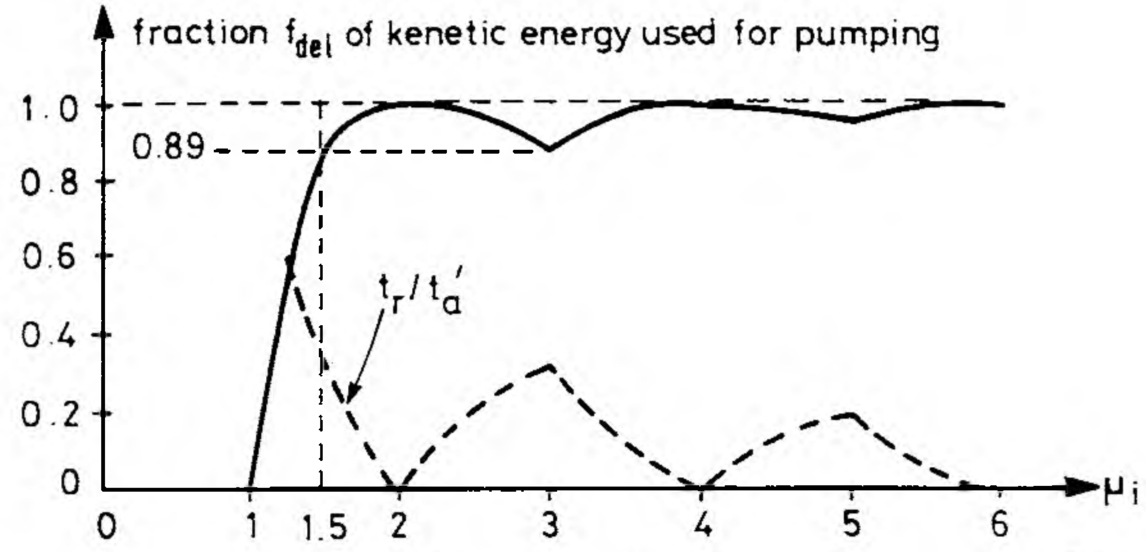
The discussions so far have emphasised the complexity of the delivery phase and the many approximations necessary to achieve a manageable algebraic model. The model developed - one of several possible - does however allow us to answer the following questions:

- how long t_d will the delivery phase last?
- what fraction of the kinetic energy developed during the acceleration phase is used in

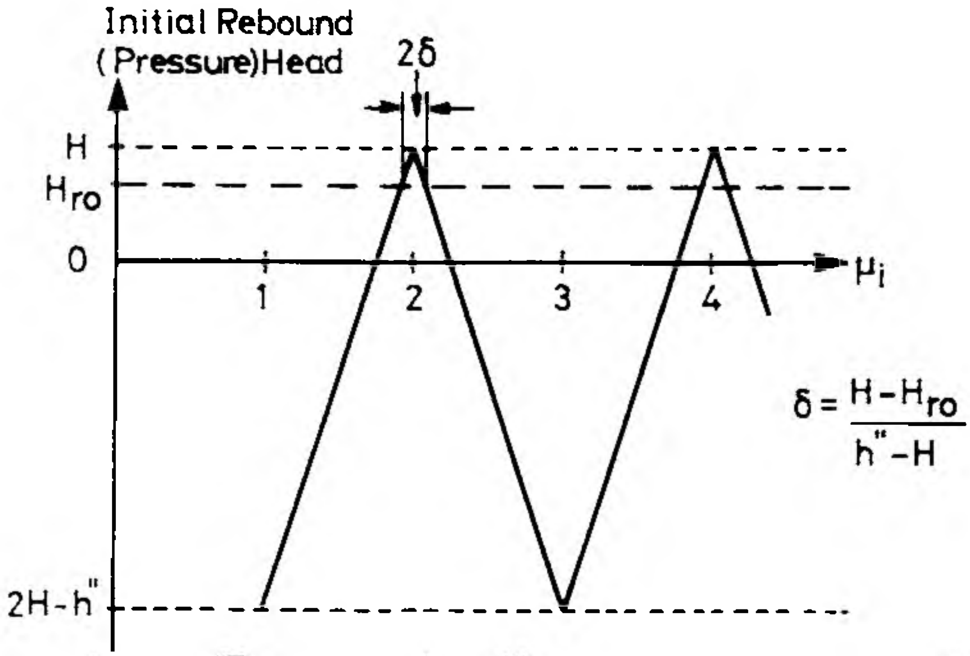
pumping (and what remaining fraction is therefore passed on to the recoil phase)?
- what therefore are the bounds within which μ_i should be tuned to lie?
- what efficiency has the delivery phase?

From the graphs we can see that the duration of the delivery phase is an integer number of 'reflections', where each reflection lasting $2T$ is the passage of a shock wave up the delivery pipe and back. By inspection

$$t_d = 2T\ (1 + \mathrm{Int}(\mu_i)) \qquad \text{where Int( ) denotes 'integer part of' and } \mu_i > 1 \qquad [14]$$



(a) **Fraction of available kinetic energy used during delivery phase and recoil time t_r** **(t_a' is acceleration time if no friction)**



(b) **Head in pump body at start of recoil phase** **(h_{ro} = head below which the impulse valve reopens)**

Fig. 10 Effect of peak flowrate on energy utilisation and rebound **(μ_i = Q_p/Q_J is normalised peak flowrate, where Q_J is the Joukowski flow rate corresponding to effective delivery head h''· H is drive head)**

It can be shown that the fraction of kinetic energy, f_{del}, applied to pumping is the shaded area (the μ - time integral) of Fig. 9 divided by $(0.5\ \mu_i\ T)$. (For various reasons it is correct with this model to exclude potential energy released by the water flowing down the drive pipe during the *delivery* phase).

The graph is of interest for several reasons. A low fraction (e.g. $f_{del} < 0.9$) indicates a reduced throughput and a reduced efficiency. The throughput is lowered not only because some of the kinetic energy is not used but also because the unused part contributes to a long rebound time (discussed later) which lengthens the cycle of operation and hence reduces the effective power. The efficiency is lowered because any losses while accelerating the water cannot be recovered even though the rebound water returns to the drive tank. The delivered fraction f_{del} is generally acceptable for $\mu_i > 1.5$ but falls sharply as μ_i drops below 1.5.

From [11] and [13] $$\mu_i = \frac{Q_p'\ c}{Ag(h''-H)} \qquad [15]$$

Low values of μ_i arise when delivering to very high heads (h'' large), setting the pump flow too low (Q_p' low), or using too large a drive pipe (A large) in combination with a low velocity of sound (c is low in plastic feed pipes). Clearly μ_i should not fall below 1 (no output) and it would be unwise to drop below $\mu_i = 1.5$ (at which use-fraction is only $f_{del} = 0.89$).

Lastly there is the matter of delivery phase inefficiency. This has been represented by an additional friction head during delivery, increasing effective delivery height h' to h''.

We can therefore define

delivery valve efficiency $$\eta_{dv} = h'/h'' \qquad [16]$$

where delivery valve headloss is

$$h''-h' = \frac{C_{dv}}{2g\,A_{dv}^2}\ \overline{q_{dv}^2} \qquad [17a]$$

The delivery valve water-flow area A_{dv} can be measured. The loss coefficient C_{dv} will lie between 0.8 and 1.2. The mean square value of q_{dv} is not easy to estimate. The instantaneous delivery flow q_{dv} has the staircase waveform shown in Figure 9, and therefore magnitude $Q_p' \times \mu / \mu_i$. The flow-weighted mean-value of $(\mu/\mu_i)^2$ rises from zero (at $\mu_i = 1$) to 0.45 (at $\mu_i = 3$) and then stays approximately constant for $\mu_i > 3$. In practice friction losses in the delivery valve are significant only at relatively low delivery heads and hence at large values of μ_i. For this condition and via a series of approximations we can come to

$$h''-h' = \frac{0.4\ Q_p^2}{2g\ A_{dv}^2} = \frac{Q_d^2}{12 A_{dv}^2} \qquad \text{which is accurate within} \pm 30\% \qquad [17b]$$

A pump designer can use equations [16], [17b] in combination with the maximum value of Q_d and minimum value of h' to decide whether the valve area A_{dv} is sufficient.

3.5 Recoil-phase Model

Water velocities during recoil are small so do not result in significant friction losses. Our interest in the recoil phase therefore centres on (i) whether the recoil suction is sufficient to reopen the impulse valve, and (ii) how long the recoil phase lasts.

From inspection of Figure 9 it can be seen that the initial rebound pressure satisfies Figure 11. However there exists a head H_{ro} above which the impulse valve will not reopen and the pumping cycle will not continue. In Figure 11 the zones of width 2δ indicate values of μ_i for which the rebound is inadequate. In an operational pump with a *weighted* impulse valve H_{ro} is never greater than the drive head H, and is usually much less. As a first approximation we can take H_{ro} as zero in which case:

$$\delta = H/(h'' - H) = 1/(R - 1) \qquad [18]$$

As $\delta = 1$ represents the valve not reopening at *any* value of μ_i (any flow setting), R should never drop below 2 and it would be risky to use values of R below say 5 (25% chance of impulse valve failing to reopen). In practice the approximation behind [18] is a drastic one; however the observed phenomena of pumps being difficult to run at low head ratios and very high drive heads is confirmed. A pump with a *sprung* rather than weighted impulse valve will have a much higher value for H_{ro}, usually higher than H in fact. Such pumps are therefore much less liable to failure of the impulse valve to reopen.

The simplest model of the rebound process is an energy one. An acceleration phase of duration t_a results in the drive pipe water having a certain kinetic energy. At the end of the delivery phase a fraction approximately $(1 - f_{del})$ of this energy remains which should cause a rebound of duration

$$t_r = t_a \times \sqrt{1 - f_{del}} \qquad [19]$$

This data is also plotted in Figure 10.

This analysis assumes constant acceleration/deceleration for a given pipe slope: it thus neglects friction effects which if included would reduce t_r/t_a by typically several percent. It also assumes instantaneous reopening of the impulse valve, whereas in practice this is so delayed by inertia that some of the rebound energy is expended in sucking air through the tiny snifter valve hole. More fundamentally, the model assumes that during rebound the water column recedes, unbroken, back up the drive pipe. In reality water and air mix and their interface is geometrically very complex. Equation [19] may be taken to give a crude estimate of the rebound time t_r. It shows that under many operating conditions t_r is not negligible and is especially large when μ_i is close to 1 (i.e. when the necessary delivery head can only just be achieved).

3.6 Combining Acceleration, Delivery and Recoil Models

In the previous sections three models have been discussed. For the acceleration phase a lumped-system model has been used, for the delivery phase a distributed-system model (with reverberating shock waves), and for the rebound phase again a lumped system model. In practice shock waves continue to roam the drive pipe water column during rebound and die away during the subsequent acceleration phase. An algebraic model to represent these would be intolerably complex and add little to accuracy. The transition from phase to phase is complex and could only be fully modelled if the dynamics (drag, mass etc.) of valves were included. The transition from acceleration to delivery and from delivery to recoil entail irreversibilities that cause loss of work energy. The first is represented by the drop Q_{CL} in delivery flowrate shown in Figure 8, which reduces both efficiency and throughput. The second energy loss, at the start of the rebound phase, arises from any backflow through the delivery valve, any air or water friction in the snifter, or cavitation. It affects efficiency unfavourable but throughput favourably (since rebound time will be reduced).

During acceleration and rebound the impulse valve is open to atmosphere, and pressures at the bottom of the drive pipe have been related to atmospheric. During delivery, however, the impulse valve is closed and the model used has related pressures (heads) to a datum at the surface of the drive tank height H higher. The change in datum has hidden certain minor energy transactions (kinetic energy to strain energy). The acceleration, delivery and rebound models can be combines as follows:

cycle time $$t_s = t_a + t_d + t_r \qquad [20]$$

where the constituent times are defined by equations [9] , [14] and [19] and Table 2.

efficiency $$\eta = \eta_{nr} = \eta_a \cdot \eta_{ad} \cdot \eta_{dr} \qquad \text{in the absence of recoil}$$

$$\eta = \frac{\eta_{nr} f_{del}}{1-\eta(1-f_{del})} \qquad \text{if recoil is significant} \qquad [21]$$

where η_a and η_{dv} are defined by equations [10] and [17], f_{del} by Fig. 10 and the acceleration-to-delivery handover efficiency η_{ad} is typically 0.97.

delivered power $$P = P_a \cdot \eta_{ad} \cdot \eta_{dv} \cdot f_{del} \cdot t_a/t_c \qquad [22]$$

where acceleration power P_a is given by equation [10].

drive flow $$Q_d = \frac{Q_p}{2} \frac{f_{del} + 1/R}{1 + 1/R + \sqrt{1-f_{del}}} \qquad [23]$$

obtained by dividing drive volume per cycle (allowing for recoil flow) by cycle time (including recoil time).

Putting the three phases together gives Figure 11.

From this graph it can be seen than mean drive flow Q_d is usually considerably less than the flow Q_{cc} at which closure commences. (Q_{cc}/Q_d is typically between 1.5 and 2.0). This yields the rule of thumb

Rot 6. *"If the source flow falls temporarily below the pump's mean drive flow Q_d the pump will stop, usually with its impulse valve open. To restart the pump without human intervention the source flow must increase to a level 1½ to 2 times Q_d".*

This rule implies that after a drive flow interruption pumps usually fail to restart without human intervention. This is a serious inconvenience in practice.

4. THE APPLICATION OF ALGEBRAIC MODELS

In the introduction to this paper reference was made to the need to explain behaviour, prepare performance graphs/tables and develop rules for system design, pump tuning and pump design. Six such 'rules' were developed in the course of model building (section 3 above). Many more might be devised. The purpose of this concluding section is to illustrate the use of algebraic models via some examples.

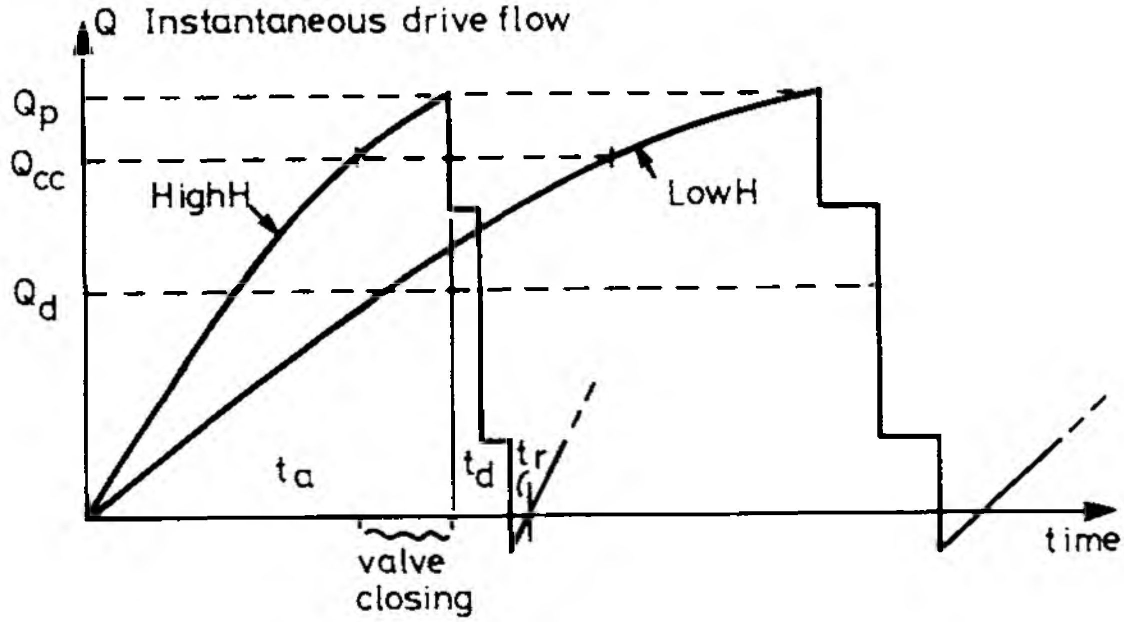


Fig. 11 Driveflow over a complete cycle for high and low driveheads
(Q_p is peak driveflow, Q_{cc} is driveflow to start valve closure, Q_d is mean driveflow)

4.1 Explaining Phenomena and Improving Pump Design

Ram-pumps, for all their mechanical simplicity, display complex and sometimes 'temperamental' behaviour. Reasons for the latter include operation too close to some limit, and blockages and leakages in pipes. Two major causes of erratic operation are the presence of excess air in the drive system (often due to excessive recoil at the end of the delivery phase) and the failure of the impulse valve to reopen (insufficient recoil). Algebraic models help us understand both phenomena.

The recoil can be expressed as an energy fraction ($1 - f_{del}$, where f_{del} is illustrated in Fig. 10(a)), a time fraction (Fig. 13) or a distance:

$$\text{rebound distance} \qquad L_r = (1 - f_{del})\,\frac{v_p^2}{2gS} \qquad [24]$$

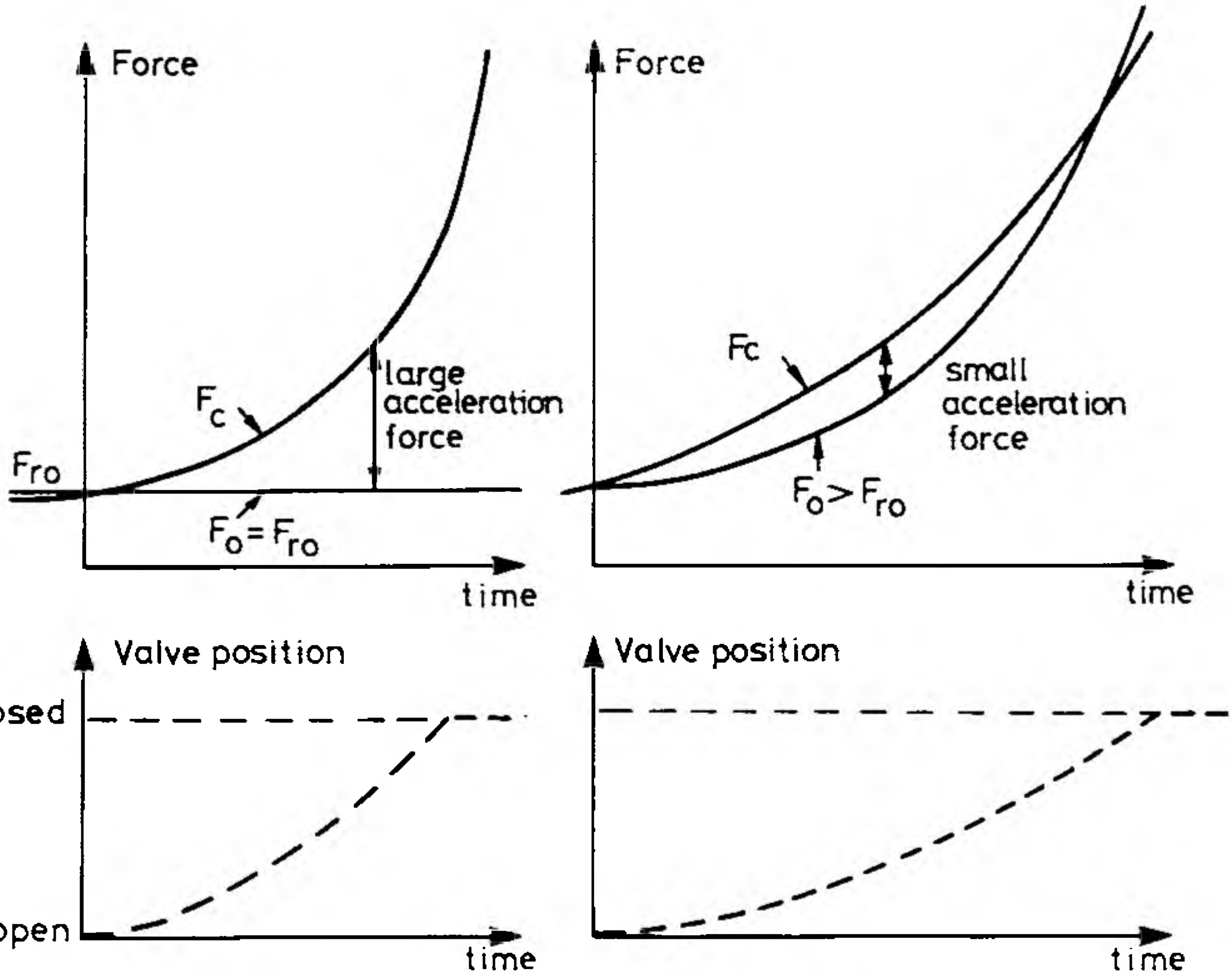
where f_{del} is determined by the ratio μ_i of Joukowski head to delivery head. The term $v_p^2 / 2gS$ is rarely greater than 100 cm, and $1 - f_{del}$ is less than 0.11 for all μ_i greater than 1.5 (see Fig. 10a). So L_r rarely exceeds 10 cm.

Excessive recoil is that which draws so much air back into the drive pipe that it is not swept out during the ensuing acceleration phase. Whether this occurs depends on L_r and the pump design. It is least likely to do so if the pump discharges under water or rises more than 10 cm from its drive pipe entry to its delivery valve. Conversely, air entrainment is most likely when the delivery head is near the maximum possible for the drive flow chosen (μ_i < 1.5), or when a high-flow pump has a low-set impulse valve.

Failure to reopen due to lack of recoil occurs when the pump is most efficient: $f_{del} = 1.0$ at $\mu_i = 2$ or 4 or 6 etc.. This is illustrated in Fig. 10(b), where a zone of width $\Delta\mu_i = 2\delta$ is shown around each even value of μ_i. Within this zone the pump may fail to reopen. Equation [18] indicates a relationship between δ and the head-ratio R, suggesting serious probability of failure to reopen at low head-ratios. However *any* such failure is inconvenient. During start-up, when a pump generally fills up a delivery pipe from the bottom, the Joukowski ratio μ_i falls with each pump stroke. Such start-up requires the continuous attention of an operator if the pump may stop whenever μ_i is close to an even number. Even worse the final full delivery head may correspond to $\mu_i = 2$ so that reliable operation is impossible until the pump is slightly retuned to change Q_p and (via the Joukowski head h_J) μ_i. Equation [18] is a special case of the more general relation:

$$\text{width of failure zone } 2\delta = 2(H - H_{ro})/(h' - H) \qquad [24]$$

So ideally we should like the head against which the impulse valve will just reopen (H_{ro}) to exceed the drive head H. The advantages and costs of raising H_{ro} may be observed from the following graph of impulse valve closure. Note that H_{ro} may be re-expressed as a force F_{ro} keeping the valve closed ($F_{ro} = H_{ro} \cdot \rho g \cdot$ valve area) which a spring force or the valve's weight must just equal and oppose.



(a) Weighted valve **(b) Sprung valve**

Fig. 12 Forces and position variation during impulse valve closure
(F_C = closure force due to water drag; F_O = opposing force; time is measured from start of valve closure)

Thus the use of a sprung impulse valve whose 'opposing force' rises as the valve closes (Fig 12(b) can give a much higher figure for F_{r0} and hence H_{r0} than a merely weighted impulse valve will (Fig. 15(a)). A pump with the former is therefore less likely to stop, provided the valve's much slower closure doesn't cause other problems.

For the pump designer many issues - and desirable proportions - have been raised in the course of the analysis. Impulse valve and delivery valve losses should be tolerable at maximum drive flow. The last stage of impulse valve closure should take less time than that for a shock wave to traverse up and down the drive pipe. An impulse valve that will reopen against full drive head should reduce the chance of maverick operation. Certain design details cannot however be decided using the algebraic models of section 3. In particular the dynamics of the opening and closing of the two main valves require more specialised models to predict velocities and accelerations, peak stresses, back leakages and so on. Time step simulations have application to these tasks.

4.2 Calculating Pump Characteristics

It was argued in Section 2 that the pump and its drive pipe should be treated as an inseparable system for performance characterisation. The many equations of section 3 can be combined to obtain performance predictions of varying degrees of accuracy. It is difficult to avoid iterative calculations since the equations are numerous and complex. However an approach of adequate accuracy for practical purposes is as follows:

A peak flow Q_p is selected.

The maximum possible flow Q_∞ is calculated from H, S, h', A, k_d and k_p.

The normalised peak flow $\lambda_p = Q_p/Q_\infty$ is obtained and used to predict acceleration efficiency η_a and mean acceleration flow $\overline{Q}_a$. The Joukowski flow is obtained (Q_J) and used to obtain the Joukowski ratio $\mu_i = Q_p/Q_J$. A modified value Q_p' may be used if the acceleration-to-delivery transition efficiency η_{ad} can be estimated. From this ratio μ_i the energy delivery fraction f_{del} can be derived and any possibility of malfunction can be identified ($\mu_i = 2, 4, 6$ etc. $\mu_i < 1.5$).

From Q_p and delivery valve geometry, the delivery valve efficiency η_{dv} is estimated. The efficiencies and energy fraction are combined using equation [21] to give an overall efficiency η. The drive flow Q_d is obtained to varying levels of accuracy using Q_a or better equation [23]. Delivery flow q can now be calculated using equation [1].

Using a sequence of values for peak flow Q_p, a table or graph of delivery flow q versus drive flow Q_d is thus obtained. Other equations can be used to obtain cycle time, although this is rarely tabulated.

4.3 System Design

System design can be undertaken using performance charts or graphs for a range of pump/drive pipe combinations: the lowest-cost option capable of meeting a specification should be chosen. This approach requires many such graphs and the ability to interpolate reliably between them. Alternatively, likely candidate systems can be evaluated using a computer programme to generate delivery flow versus driveflow loci. Unfortunately neither of these approaches fits the situation frequently met with in the field, where computers are remote and water technicians not very numerate. Rules of thumb also have their place.

Consider the difficult task of choosing drivepipe diameter and slope. The drivepipe pressure rating needs, incidentally, to be about twice the *delivery* pressure to limit fatigue failure and the drive pipe should be able to withstand negative heads down to $-H$.

Slope S is usually site-constrained: it is difficult to lay a pipe steeper than the valley side. Low slopes are expensive (the drive pipe is long) and inefficient. Very high slopes give a high operating cadence (and thus high noise and a short pump life), low efficiency through increased frequency with which valve closure losses occur, and possibility of impulse valve malfunction due to too short a drive pipe.

Too large a drivepipe diameter D result in excessive drivepipe cost and peak velocities too low to generate the required delivery pressure. Too small a diameter results, for a given drive flow, in excessive acceleration phase losses and low overall efficiency.

It has already been argued that for reliable operation the Joukowski ratio should not fall below 1.5. This can be expressed as a *upper* limit on pipe size. Noting that at $\mu_i = 1.5$, $Q_p = 3Q_d$ and $A = 0.67\ c\ Q_p/g\ (h-H)$, we need to know the effective wave speed c to derive any specific rules. For steel pipes we should use c = 1400 m/s and for PVC pipes (of wall thickness one-tenth of their diameter) c = 400 m/s. These lead to the rough rule of thumb.

ROT.7 "For steel pipes, drive pipe diameter in mm should not *exceed*
$600\sqrt{\textit{drive flow in litres / sec} \div \textit{delivery head in meters}}$;
for PVC pipes replace 600 by 300"

A *lower* limit for pipe diameter in combination with pipe slope can be obtained by ensuring Q_p/Q_∞ does not exceed 0.8, yielding $Q_\infty/Q_d > 2.5$. However Q_∞ is the maximum flow obtained when drive pipe and pump together are left to run with an open impulse valve. The flow through an open-ended drive pipe alone, laid down the slope, would be higher. By a series of assumptions one can arrive at a rule:

ROT.8 "The drive pipe diameter and slope must be sufficient that its flow open-ended, the pump having been removed, is at least 3.5 times the intended drive flow".

4.4 Tuning

The contribution of algebraic models to achieving good pump tuning is not very great. Tuning of ram-pumps on site is not essential, as a pump set for a mid-range drive flow will operate on most sites. Indeed with some operators the provision of a means of tuning may be unwise: the pump is then vulnerable to gross maladjustment. Where the pump is tuneable, by variation of the flow Q_{cc} at which impulse valve closure begins, heuristic methods are often adequate: the operator finds by experiment the setting that maximises delivery flow within the limits of available source flow, Only when several pumps are operated in parallel is this trial-and-error approach likely to prove difficult.

An operator usually has little data with which to work. On many sites the pump cycle time is the only easily measured variable; drive flow and waste flow may be inconvenient to gauge, delivery flow is measurable but only at the delivery tank sited many minutes climb away from the pump house. Unfortunately cycle-time is not simply related to the flow variables of interest. Figure 13 shows cycle time, drive flow and delivery flow as functions of Joukowski ratio μ; itself a measure of peak flow and hence of tuning. The plot is for a representative system and displays considerable complexity.

No simple tuning rule for finding a high efficiency point (such as $\mu = 2$) by observing cycle time suggests itself.

With knowledge of site parameters and in particular of drivepipe diameter, an installer or manufacturer can pre-set a pump's tuning Q_{cc} to a suitable value, or can constrain it to lie within a particular range of flows. A suitable range might be that which makes μ_i lie in the narrow band $1.25 < \mu_i < 2$, or the wider band $1.5 < \mu_i < 4$. The lower limit prevents the pump being tuned to give negligible delivery or to entrain air. The upper limit prevents use of unnecessarily high drive flow and also protects the pump for excessive over-pressures should its delivery become blocked. The optimum range will depend on the ratio of open-pump flow Q_∞ to Joukowski flow Q_J.

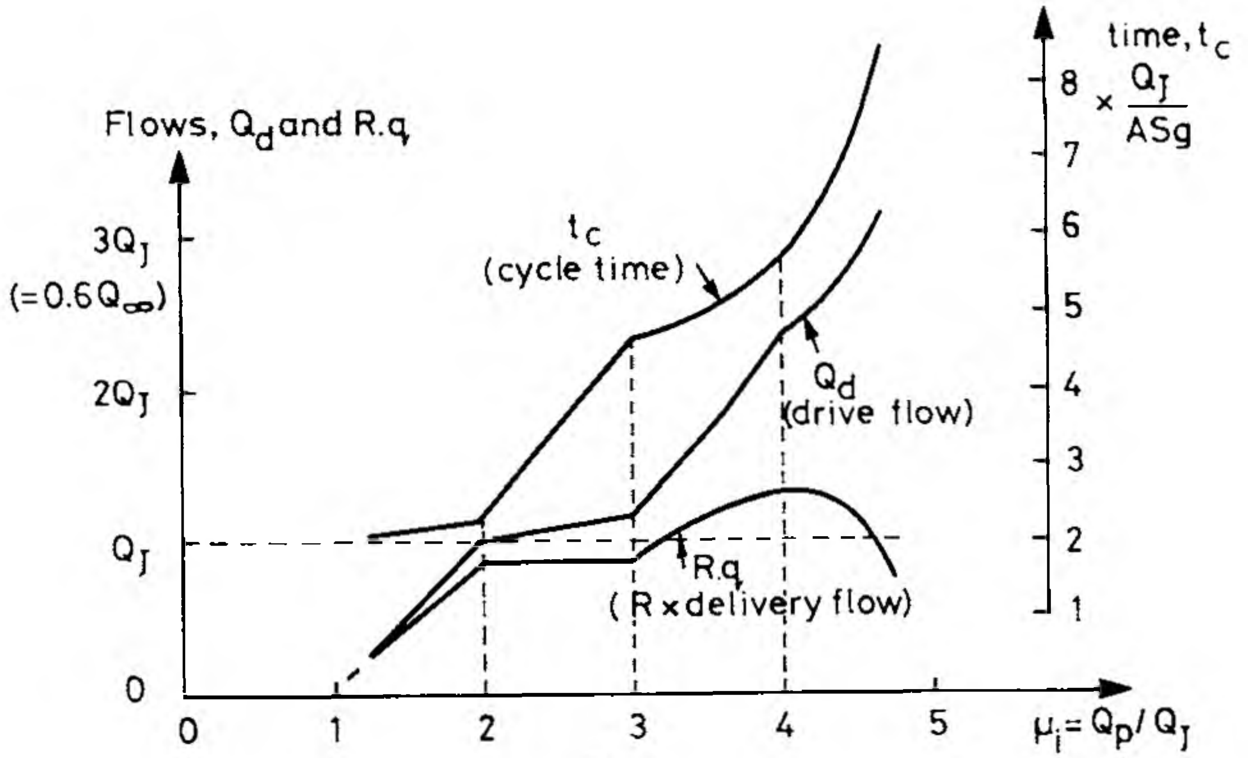


Fig. 13 Driveflow, delivery flow and cycle time variation with tuning

(Q_J is flow just sufficient to attain delivery head, t_c is cycle time, Q_d is driveflow, q is delivery flow, assuming $Q_\infty = 5Q_J$ $R = 20$ $\eta_{dv} = 0.9$)

5. CONCLUSIONS

The three main phases - acceleration, delivery and recoil - of a ram-pump cycle have been modelled using equations of manageable simplicity. Several rules of thumb have been derived. The paper has throughout stressed the application of algebraic modelling to solve common problems in system design and tuning.

Equation [1] in section 3.2 represents the level of modelling usually employed by manufacturers in the preparation of data sheets. This paper has shown that its assumption of constant efficiency is unreasonable and its objective of describing the performance of a pump in isolation (from its drive pipe) is unrealistic. Such simple models are incapable of explaining many phenomena important to pump users. Through the use of two ratios ($\lambda = Q_p/Q_\infty$ and $\mu_i = Q_p/Q_J$) much more can be explained, even though the peak flow Q_p itself can not be readily measured. The several approximations and omissions in the model developed make it inadequate for some aspects of pump design optimisation, for which complex simulation models are preferable.

DEVELOPMENT
TECHNOLOGY
UNIT

D
T
U

Working Paper No. 42

Low-cost rural transport for Sierra Leone

March 1994

C.E.Oram & T.H. Thomas

Development Technology Unit
Department of Engineering, University of Warwick
Coventry CV4 7AL UK
Tel: +44 (0) 1203 523122 Fax: (0) 1203 418922
email: dtu@eng.warwick.ac.uk

LOW COST RURAL TRANSPORT FOR SIERRA LEONE

TH Thomas and CE Oram

0. Abstract

This short paper reviews the demand for rural transport services and discusses the present inadequacy of such provision in Sierra Leone. It examines the main transport modes that might meet its transport needs and concludes that both animal power and assisted human-powered transport have the potential for a significant contribution. The focus of the paper is on the provision of vehicles rather than on the provision of roads and paths, and various interventions are suggested relating to the use and manufacture of such vehicles. In this context manufacturer training, marketing and vehicle design, are discussed. It is concluded that small town manufacturers and artisans are the most appropriate sources of vehicles for rural transport and that their needs for profit and return on labour must be met if there is to be any hope of sustainable development.

1. Introduction and general comments

Transport is essential to rural development and future prosperity. But in most countries in sub-Saharan Africa, from the Sahel to the Zambesi, the cost and inconvenience of transport for farmers is very high. Each person in a rural community in Africa is likely to spend an average of over two hours per day on transport tasks, despite walking less than 10km with a payload of only 20kg or so. This represents a poor level of effectiveness when compared with some of the more sophisticated transport methods shown below. Such ineffectiveness adds to the burden already carried by the farming community and severely inhibits improvements in productivity and hence development.

Improved transport is needed to allow ready movement of materials and tools around the farm to improve the timeliness and efficiency of farming operations and reduce wastage of crops. Farmers often generate surpluses which cannot command proper prices because the goods cannot be sold in a favourable market. Such produce must rather be disposed of at much less advantageous rates by the nearest roadside, itself often at such a distance that spoilage occurs. (Highly perishable goods may even be unmarketable.) In Sierra Leone the paucity of roads and the general absence of suitable vehicles makes these problems more acute than the African average. Clearly the rural community will operate at a depressed financial status, when compared to urban areas, and in such situations it is difficult to accumulate capital which might be invested in improved infrastructure and farming inputs.

One of the major factors contributing to the shortage of transport is the cost, and in particular, the initial outlay on vehicles. Farmers are understandably reluctant to increase their risk as this may increase their exposure to money lenders and credit schemes. Cheaper transport modes, of greater effectiveness, are likely to make smaller demands on scarce capital and allow faster and more thorough development. With limited resources and skill levels, most countries find however, that provision of suitable vehicles is difficult. Little effort has been invested anywhere, either in the developed or the developing world, into designing low-cost and, more particularly cost-effective vehicles, for use in the rural areas of developing countries.

What is required are vehicles which can be made locally and repaired locally. Many studies have suggested that mutual proximity benefits the user and the producer of goods in rural areas, by facilitating dialogue between the two (Starkey, 1988, Kanu, 1988). Users become aware of the limitations of the technology, and producers become familiar with the needs of the users and adapt designs to suit local conditions. Regarding the types of vehicle most beneficial to rural dwellers; analysis of the data in the table below suggests that animal carts represent a considerable increase in transport power or effectiveness over head-loading. They are relatively cheap and cost effective. They cannot compete however, with motorised transport, if sufficient capital is available to purchase it.

Whilst this position paper attempts to give a general view of low-cost transport in sub-Saharan Africa and of transport in Sierra Leone in particular, it concentrates upon the animal cart because this is the most powerful and accessible innovation which could be introduced into the country. It is recognised however that a major constraint to the growth of this transport mode is the very low numbers of draught animals in the country and the cultural resistance to their use. Only about1% of the national herd in Sierra Leone is used for draught purposes and by about only 5% of the farming population (Kanu and Sankoh, 1990).

A danger during any period of change is that the interests of women will be overlooked and that they will be left worse off as a result. To combat this danger, one trend over the last few years in other countries, has been to emphasise donkey-cart rather than ox-cart development because there is generally less resistance to women handling equines than bovines. Women are then able to retain a stake in an income generating activity. Sierra Leone has very few working equines and so it is likely that other measures will have to be taken to protect womens' income in the areas affected by animalisation schemes.

2. Present Transport Use

There have been a number of studies of rural transport in developing countries, (eg Barwell 1985). About two thirds of the total transport demand in rural Africa is met by women headloading payloads of up to 30kg. Headloading occupies the average adult for up to three hours per day and represents a daily transport output of about 100kg.km. Three-quarters of trips are less than 6km in length, but they absorb major proportions of rural dwellers' time - perhaps 20% of the average women's active day. Table 1 below (Dennis, 1993) shows typical labour time demand and usage of rural transport and Table 2 (Adeoti et al 1989) gives an idea of trip lengths in a Guinea Savannah environment similar to that in some areas of Sierra Leone. Of the remaining one quarter of trips, most are less than 15km and are over unsealed (ie dirt) roads.

TABLE 1: Household transport statistics for rural Africa.

purpose	time input [hrs/year]	transport output [kg×km/yr]
domestic: firewood+water	600-1500	20 000-60 000
agricultural	200-450	6 000-10 000
social	200-600	zero payload

TABLE 2: Trip lengths undertaken by rural dwellers.

purpose	avg distance [km]	relative frequency [%]	proportion total travel distance [%]
to farm	3	70	23
to market	25	10	25
for domestic needs *	6	6	4
social and religious	45	10	40
medical, educational	30	4	8
total	11	100	100

* The figures for domestic purposes are low and may perhaps reflect the difficulty of obtaining data from women in an Islamic society.

Demand for transport in connection with farming activities is very seasonal and transport owners use various means of spreading out total transport demand over the year. Thus building and maintenance work, often requiring the transport of significant amounts of material, is undertaken away from periods of high agricultural demand and there is normally also a great deal of informal transport hire or borrowing and barter. Cart operators appear very conscious of their animals' condition and nutritional status and usually regulate the load they carry to reflect this. It is fairly rare to encounter a badly overloaded cart.

3. Characteristics of Present Transport Modes

The main transport vehicles or transport modes used in rural areas in Africa are shown in the Table 3 below. The values shown in the table are estimates, but are believed to be representative of West Africa. They demonstrate the power of the more sophisticated modes. (The column 'power' is deliberately included to highlight the somewhat irrational attractiveness of the more powerful modes). Not shown in the table is the ox-drawn sledge, which is generally regarded as being too destructive of road surfaces and is banned in some countries.

TABLE 3: Characteristics of rural transport modes.

transport mode	average payload [kg]	speed [km/hr]	'power' (payload xspeed) [kg.km/hr]	daily range [km]	capital cost [$_US]	total cost/ day* [$_US]	'power'x hrs/day /dailycost [kg.km/$]
human-powered							
head loading	15	5	75	15	1†	2.0	112
wheelbarrow	25	5	125	10	30	2.0	123
handcart	100	5	500	10	30	2.0	494
bicycle	17	12	210	50	100	2.1	420
animal-powered							
four pack donkey train	100	5	500	20	100	3.1	648
donkey cart	150	5	750	20	125	3.1	966
ox cart	200	4	800	20	150	3.1	1 280
engine-powered							
2-wheel tractor+trailer	200	20	4 000	50	2 000	6.7	1 490
mini-bus/ pick up truck	750	80	60 000	200	10 000	20.5	7 320
lorry	7 500	70	525 000	200	50 000	64.5	23 300

* The daily cost includes the daily wages of an operator ($_US 2) or driver plus the cost of fuel/ feed and depreciation of vehicles.
† The price of a pair of shoes.

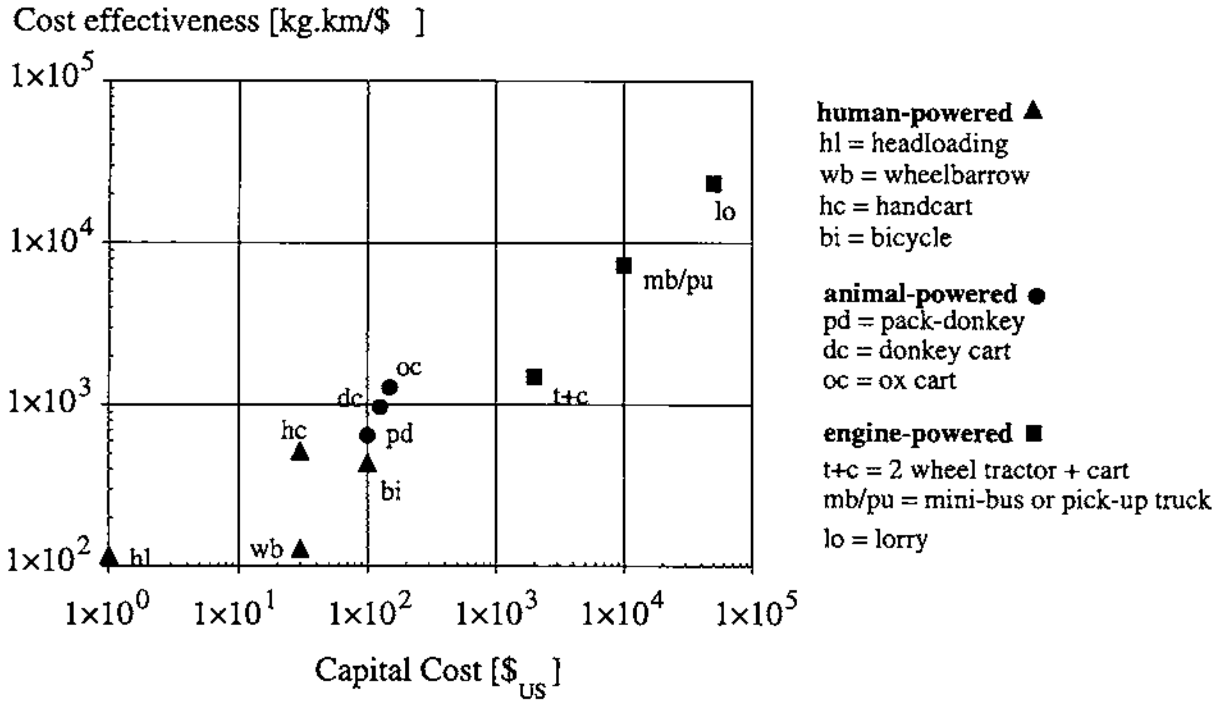


Figure 1: Cost effectiveness versus capital cost for a range of transport modes.

The graph below demonstrates that the cost-effectiveness of transport (the typical daily output divided by the cost per day) rises with capital cost. In other words more capital-expensive transport modes are more cost effective. This confirms the commonsense view that it is probably the high capital cost of more powerful transport devices that mainly prevents their more widespread use.

The graph clearly shows a progression in productivity and cost effectiveness from human to engine-powered vehicles. It does not however take into account the nature of the transport demand or the environment in which transport operates. For the great majority of trips, as illustrated by Table 2, the apparent economics of using motor vehicles are not obtainable for a number of reasons. Loads are usually small and users must wait before vehicles become available, suitable roads and tracks may not be available, and the costs in the table do not include maintaining vehicles in areas where spares are not available at any reasonable price.

The following three sections examine human, animal and engine-powered vehicles in turn, to identify opportunities for improvement in their performance, supply or operating economics.

4. Human Powered Transport

Head loading has been mentioned elsewhere as being a large user of time; it is also suspected of causing significant injury to neck and spine. Damage to neck joints may manifest itself later in life with arthritis and deformity. Support of a large load, whilst not requiring energy in a physicist's theoretical sense, involves considerable muscle tension and therefore much energy use and fatigue.

A fairly recent innovation, in many urban areas, is the introduction of wheelbarrows, usually operated by children. Wheelbarrows are able to carry higher loads over reasonable surfaces with slightly less chance of physiological damage to the user than headloading methods. It should be understood that wheelbarrows are deliberately designed to load the operator through his/her hands in order to provide better control and grip on poor ground surfaces. Patented and novel devices, such as the Chinese wheelbarrow, claimed by some agents to reduce operator fatigue by reducing handle forces, may not therefore secure an overall benefit.

Two-wheel handcarts are also used in large numbers in African towns where road surfaces allow low traction forces to be effective. In rural areas the narrowness of paths as well as their roughness discourages their use. Two types are widely in evidence; the water bowser and the heavy wooden-body cart. The former appears, like the wheelbarrow, to be a relatively recent innovation. Two light motorcycle wheels support a body made from angle iron and steel reinforcing bar and carry up to 500 litres of water, propelled by several children. Design of the the latter carts is very similar to that of oxcarts, but payloads are inevitably less. The economic status of these carts is, if anything, lower than that of animal carts and most are made from scrap vehicle components - very few are made new.

Bicycles are popular where road surfaces are adequate and where they are affordable - prices are usually high by European standards. Punctures in very worn and old tyres are frequent. It is not in general realised how variable is the susceptibility of rubber to puncture - old brittle rubber may be one thousand times more likely to puncture than new rubber. The major benefit from bicycle use is increased speed - bicycle adaptations to carry high loads do not appear very successful. By contrast with the Asian situation there is very little if any use of cycle rickshaws and cycle trailers and this probably arises from the poorer road conditions and the greater distances. Indeed the spread of rickshaws into the rural areas of India followed the progress of rural road building.

5. Animal Powered Transport

5.1 General Characteristics

The introduction of animal power into most of sub-Saharan Africa, with the exception perhaps of Ethiopia, is a phenomenon of the twentieth century. Rural dwellers in many areas are still unfamiliar with the care and feeding of animals and this, and the problems of harnessing and disease, have acted to restrict their introduction. Where these problems can be overcome, animal-powered transport is usually a cost-effective and appropriate technology by which to improve rural productivity. This has been shown by its success in certain other countries in west Africa - Senegal and Niger, for example.

Whilst animal traction has not, in the past, been popular with the farmers of Sierra Leone because of cultural attitudes and government policies (Bangura 1988), the realisation that there is little alternative to their use is beginning to gain currency. Technical and engineering literacy among many farmers is at only a modest level and there is considerable attraction in a device that 'makes and repairs itself', and which is able to use almost any plant material as fuel. Draught animals too are unusual in another respect: few industrial products appreciate in value, but that is the case with a draught animal. Up to about ten years of age, animals gain weight and add value - the European taste for young tender meat is not common in Africa.

Many different animals are used for draught purposes including goats, camels, horses and buffalo, but the main ones are donkeys and oxen. These two are sufficiently disease resistant and adapted to tolerate heat stress, to be widely popular. Draught animal physiology and performance has been investigated by a number of expert teams worldwide and quite a good understanding of their behaviour, characteristics and needs is beginning to be assembled. Draught animals cannot usually eat whilst working and may require high quality supplementary feeding if they are not to lose weight during periods of sustained labour, such as at harvest time. The nature of this supplementary feeding and its quantity have been investigated in a number of different countries and animal breeds. A fairly recent innovation to be investigated by modern scientific methods is the use of cows as draught animals even when pregnant. Such comprehensive use of an animal requires a considerable skill and understanding of its physiology, but does appear to give an improvement in the economic return from many animal breeds.

Until the turn of the century in the now industrialised countries, animals were not only used for transport purposes, they were used to provide shaft power for both portable and stationary machinery as well. Animal power units (APUs) or animal 'gins' were extensively used to drive hay elevators, grain kibblers and flour mills for example. These traditional machines are unsuitable for production in Sierra Leone and most other countries in Africa, but the Sierra Leone Work Oxen Programme has been investigating some of the recent German assisted attempts to design locally manufacturable animal power units. The importance of APUs to Sierra Leone's transport programme lies in their potential to increase year-round utilisation of animal power and thereby improve the economics of operation. Keeping animals in work throughout the year preserves training and fitness, which are otherwise lost quickly. However the present and future supply of animal carts is probably the key determinant of whether animal-powered transport can be expanded in Sierra Leone.

5.2 Present Supply of Animal Carts

Farmers' carts come essentially from three sources:

i) manufacturers outside the country, operating in the international market, who produce new carts, either assembled or more often, in knocked-down form;

ii) urban manufacturers in-country, with perhaps 50 employees, also producing new carts;

iii) local artisans who produce carts using scrap automotive wheels, tyres and axles for the moving parts, and new timber and steel for the body.

International manufacturers produce high quality products, sometimes at attractive prices (one UK/Italian supplier is quoting about $\$_{US}80$ for a 2½ tonne axle suitable for fitting to animal carts and equipped with commercial roller bearings). International suppliers are able to benefit from a ready supply of materials at the best prices; they are able to submit credible tenders to international development organisations and thus obtain large orders; and they are able to smooth their production over many different markets, probably not all experiencing poor economic performance at once. But they do have to support a large international sales team and cover high distribution costs both internationally and in-country. Furthermore they cannot afford to produce anything less than the highest quality, which may well not be affordable or attractive to farmers. (Honouring warrantee/guarantee failures with poor communications is very expensive.) There is another important point too: because of the high capital costs an essential component of any improved transport scheme involving major imported products such as carts, is credit. Credit managers are not farmers and engineers, but often end up deciding which products to support, thus distorting any local preference or opinion. The result is often inappropriate products, lack of sustainability, local debt and high foreign exchange costs.

Local supplier companies (type ii above), are usually able to benefit from the lower labour costs in-country, but are often faced with material prices above those paid in industrialised countries because of local import duties. They do not have the expensive international sales team however, nor the international transport costs, but they are still faced with in-country communications difficulties and distribution costs, which may render the concept of dealerships and spares networks completely unworkable. They are able to make lower quality and cheaper products and those more appropriate for local conditions and tastes, but the quality is often reduced more than the price.

The availability of scrap automotive parts in developing countries for cart production is usually poor and scrap may even be imported from industrialised countries. There are additional problems with the quality and methods of use of such components. Although the loads and speeds to which these scrap components may be exposed in animal carts is usually within the original design envelope, such components are often badly worn or partly broken, and such damage may be compounded when the parts are adapted for cart use. A very common procedure for example, with scrap vehicle axles for animal carts, is to remove the 'banjo' unit - the central final drive unit and differential - to lighten the axle and reduce friction. Often the hole is not covered, leaving the remaining bearings unlubricated and exposed to abrasive dust. More significantly, these bearings are not designed to carry wheel loads without the support of the banjo unit bearings and consequently they fail fairly quickly. The poor supply, and the consequent high price of scrap, has acted as a lid on the activity of indigenous carpenters and blacksmiths engaged in the cart construction business.

Cart bodies built by local artisans are usually of fairly complicated design - perhaps attempting to copy the sophisticated products which their customers would like to obtain from overseas - but construction is often poorly executed, with the result that overall utility and longevity is low, whilst construction costs and times are high. Considerable improvement could be made here, but only if there is good communication between customer and user. The education and training of users is necessary both to help them make best use of what is available and for them to understand the costs and difficulties of making what they want, as opposed perhaps to what they need.

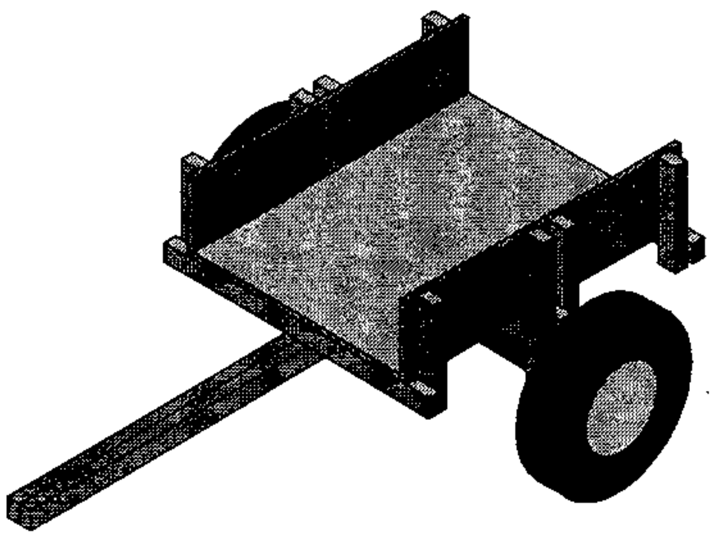


Figure 2: DTU low-cost wooden oxcart with clenched re-bar fixings.

5.3 Animal Cart Design

Current standards of design of animal carts are fairly low, which may arise from a variety of causes. In general, there is at best, only a very weak market; purchaser choice is almost invariably very restricted, and price is a very poor indicator of quality. Most purchasers appear technologically unsophisticated and unlikely to be able to judge desirable qualities in their tools. There is a massive lack of what, in the industrialised countries, would be called salesmanship, so that communication between customer and manufacturer is poor or non-existent. The producer does not know what is wanted or needed, and reluctantly or otherwise, must guess. The user is unaware of what is expensive and difficult to produce and, if asked, requests adjustability and complicated features. The result is expensive, inappropriate and inefficient products.

Compounding these difficulties is the need of the manufacturer to avoid failure and a bad name. There is thus a reluctance to try lightening, cheapening or unconventional features and no natural mature product can develop as a result. Where there is intervention, in the shape of animalisation programmes or the like, the outside scheme administrator cannot afford failure either and buys his way out of it with expensive products, often with imported components which need credit arrangements and foreign exchange.

Only a very few organisations have devoted any significant effort, whether to quantifying existing standards, to measuring needs, or to developing guidelines for carts. Recently however there has been activity in Zambia (Dogger, 1990) and Malawi (Malawi Bureau of Standards, 1986), which has measured cart usage in terms of distance travelled and has developed standard tests for durability and strength.

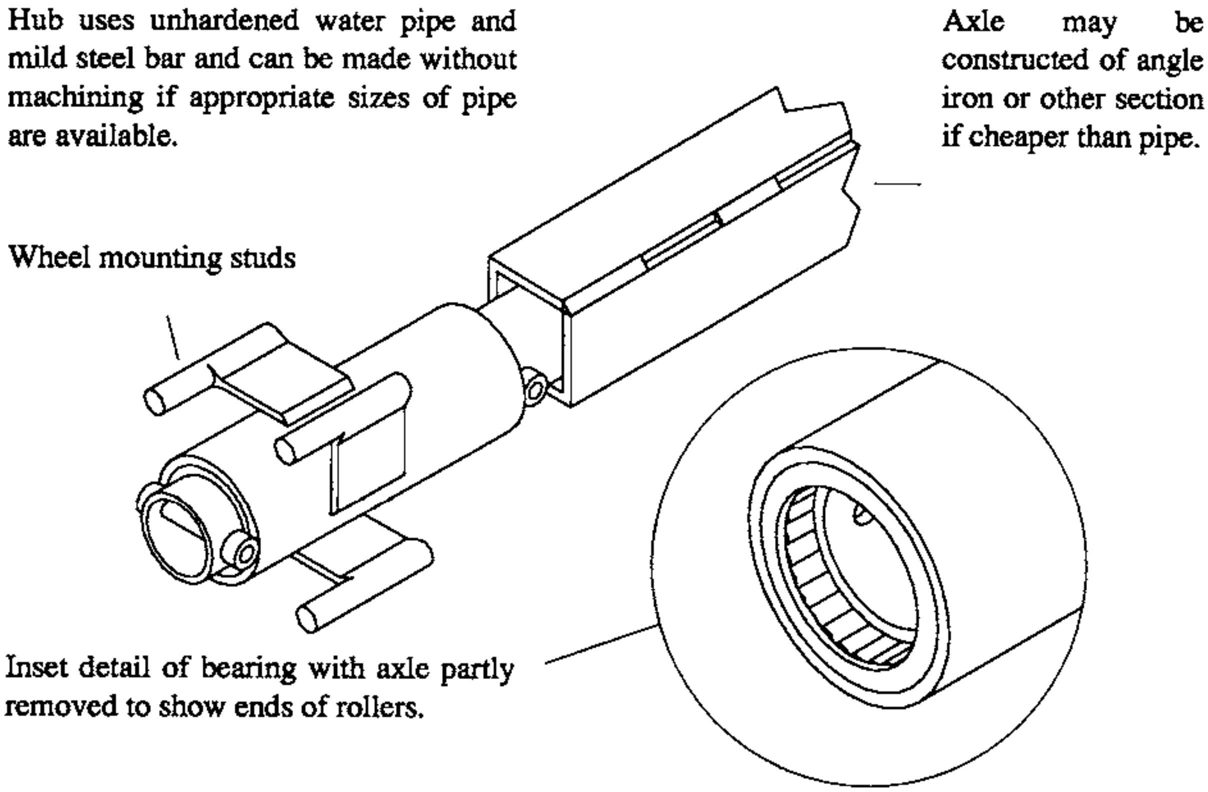


Figure 3: low-cost axle, rolling element bearing and hub for animal carts.

There exists no clear understanding or rationale for even the most basic requirements of cart design including:

- lifetime of running gear (wheels, axles and bearings),

- longevity of bodies,

- height of load trays (oxcarts are usually made with low trays, whilst donkey carts usually have high ones),

- optimum load tray lengths and widths,

- the need for head, tail and sideboards.

High load tray height, and the consequent high load centre of gravity, might be expected to have an effect on animal fatigue in areas with poor road surfaces. Not only are animals then subject to changes in draught as the wheels traverse bumps, but they also experience rapid changes in vertical load. This is because of the offset between centre of mass and any retarding/ acceleration force acting on the wheel, this causing a fore and aft pitching effect on the cart. On the other hand low tray height may allow load wetting whilst fording rivers, and does not necessarily ease loading if the people engaged in loading must bend their backs twice; once to pick up the cargo from the ground and then again to set it down on a low tray.

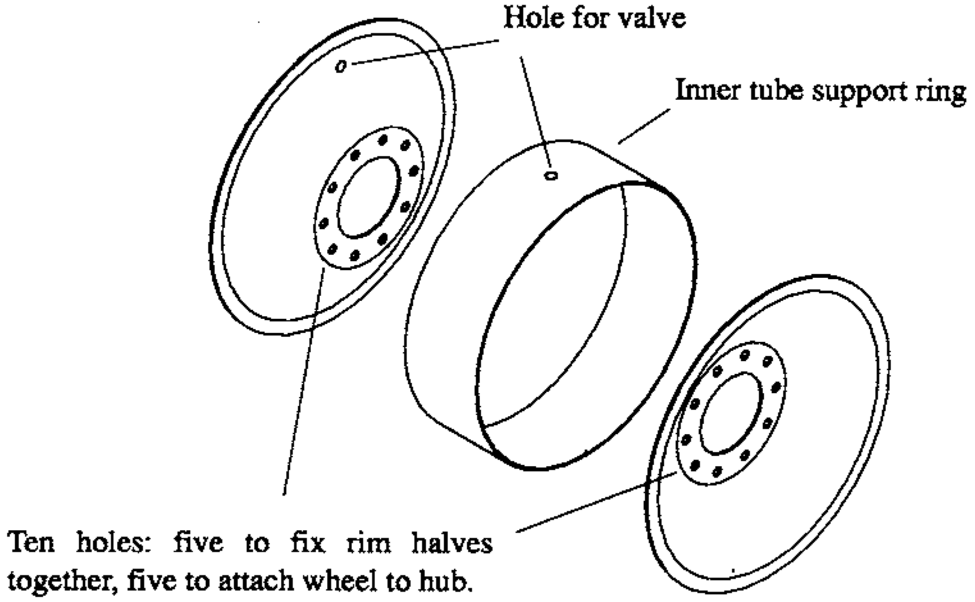


Figure 4: pressed steel wheels for local manufacture.

Notwithstanding the comments above regarding the rationale for existing designs, there appear to be several extant cart styles in west and southern Africa. Body height has already been commented upon, body lengths are around 1.9m, and widths around 1m. Most bodies are equipped with sides, sometimes removable. Wheels are generally pneumatically tyred and large sizes such as those from Land Rovers are preferred because of their reduced rolling resistance. Where there is no established axle supplier like the SISCOMA plant in Senegal, most carts use a scrap vehicle axle with a locally built body. In west Africa this is usually in timber but in eastern and southern Africa steel is preferred.

6. Motorised Transport

The most basic level of motorised transport in most countries in sub-Saharan Africa is the light motorcycle. Such machines, up to 200cc, are used both privately and for hire as taxis. Relatively little goods transport is undertaken by them, though it is possible to see things as large as beds and mattresses transported on them. Just as there are no rickshaws in Africa, so too are there very few of the light three-wheeled trucks and people-carriers seen widely in Asia. Another popular innovation in China and elsewhere, the single-axle tractor and trailer combination, is also almost unknown. Most mechanised transport in Africa is by mini-bus, pick-up truck and rigid lorry. Overloading in all vehicles is almost universal - thirty-five people plus baggage on a pick-up is commonplace, and vehicle breakdown as a result of this practice, poor maintenance and a shortage of spare parts, is frequent.

The most obvious potential improvement to motorised transport in Sierra Leone is to fill the gap identified above; that occupied by the three-wheeled light truck in India and the single-axle tractor and trailer combination in China. Such machines might be copied from Asian sources or developed specifically for Sierra Leone conditions and might be popular especially if the cost can be reduced significantly. A number of organisations are involved in improving or developing new small motorised vehicles for rural use and it is not inconceivable that a light single-axle tractor and trailer combination of adequate strength to withstand the abuse discussed above, could be marketed for $\$_{US}$1500 or even less.

7. Interventions

To improve the status of rural and low-cost transport in Sierra Leone a number of different interventions might be considered. A choice has consciously been made below to discuss only those interventions which require modest investment and which can be made in an incremental manner. For completeness some other interventions, which are considered to be less appropriate, are also listed. But the preferred interventions are as follows.

i) Introduce new designs and production processes appropriate to the needs of Sierra Leone, which will make local manufacturing of animal carts easier to carry out. A number of innovations in the design and production of animal and human-powered carts have been made in the last few years and promise to change the cost effectiveness of such transport significantly. Such improved products and methods include pressed steel and cast aluminium wheels, low-cost ball and roller bearings, some of which require no machine tools (Oram 1992), and low-cost body designs in both steel and wood using cheap fixing techniques. These modified cart designs can be produced with only two or three days labour per cart, and with a total cost including body, axle and bearings, and scrap wheels and tyres, of as little as $\$_{US}$ 50.

ii) Provide technical training of metal and woodworkers to complement these improved cart manufacturing techniques, This is very important if they are to advance and adapt the designs to cater better for their customers needs. The designs suggested above are specifically tailored to require little improvement in skill level to make adequate working products. Nevertheless training of artisans and manufacturers, including rudimentary book keeping, would be very beneficial to improve financial viability and longterm sustainability.

iii) Improve the communication between producer and user by providing training to extension agents and farmers. The market for low-technology equipment for the farming sector has already been described as poor, and the maintenance of a sales and spares network too expensive for companies supplying low-cost vehicles. As a consequence the customer often has unrealistic expectations and has no good idea of the cost of the provision of particular features.

iv) Undertake a programme of standards testing of vehicles and equipment by a government body and make the results widely available. The various parties would then become aware of inadequacies and should be able to make informed purchasing decisions. Such an approach is being followed in Zambia and has already been mentioned above.

These measures to improve animal traction equipment no doubt hinge on good working relations and cooperation with the Sierra Leone Work Oxen Programme and with its drive to increase the numbers of available draught animals in the country. The farmers' preference for engine-powered machinery is probably more difficult to support in the country's present circumstances and their technical proficiency may be too low to allow effective and sustained economical use of such machinery.

Other interventions considered less realistic or easy to introduce are listed below.

- Introduce extra credit facilities to allow farmers to buy already existing vehicles, either in-country or more probably, internationally.

- Introduce hire facilities through government or other agencies, so that suitable vehicles and resources are shared effectively over the rural population. This indeed was the method by which tractors were provided by the Sierra Leone government up until 1986 for field operations.

- Guarantee raw material availability to encourage the engineering and manufacturing sectors. But subsidies and special imports of materials are likely to be expensive and difficult to achieve, and to require intervention at high level. An easy-does-it approach using existing materials suppliers may be more productive.

It is the DTU's opinion that any sustainable improvement in rural transport must rest on local manufacture of appropriate vehicles. Interventions solely with users have not succeeded, nor those that assume the availability of imported equipment. Interventions must be concerned to improve the lot of farmer and that of the rural artisan and small town manufacturer. Because the linkage between the manufacturer and the user is often unsatisfactory, one feature of all interventions should be to strengthen them - to support what would normally be the work of the sales team.

8. Conclusions

The rural transport needs of Sierra Leone appear to be poorly met and to be a severe check on rural development. This document has set out to provide some analysis of the situation and suggest some interventions. Firstly the present transport needs in rural areas have been examined and an overview of present transport modes, including those based upon human muscle power has been given. Then animal-based transport modes, in particular animal carts, have been examined in more detail and small motorised options discussed briefly. The document closes with an examination of the interventions which might readily be made by an agent, either governmental or non-governmental, to improve rural transport provision. The report specifically avoids issues surrounding the provision of track (paths and roads) and notes the difficulties which have been (and might be) experienced with the provision of motorised facilities to the rural sector. It notes also the difficulties facing animal traction - the general lack of familiarity with draught animal husbandry and the poor status of animal power in the rural community.

Some points stand out from this analysis and from other considerations.

i) Improvements in rural transport demand increased availability of vehicles and lower prices. Any sustainable development in vehicle production must secure a profit for the artisanal and small town manufacturing sectors involved. Too often there has been a concentration on the requirements of the user and on how these may be satisfied, whilst relatively little effort has been expended on expanding local provision of equipment. Resisting urban drift, a problem in almost all the developing nations, requires that wealth be generated locally and spent locally. Income generated in the farming sector is better spent on local infrastructure, including transport, and on rurally produced vehicles if possible.

ii) An important cause of low profitability in the small manufacturing sector is the preponderance of fairly poor designs of vehicle, both from the manufacturing and from the use points of view. Significant improvements could be made here to reduce the cost of vehicles and their demand on foreign exchange.

iii) Inspection of the existing transport infrastructure of Sierra Leone suggests that, in common with much of sub-Saharan Africa, a significant gap in the capacity (payload×speed) spectrum exists. On the one hand, the major portion of the transport demand is met by headloading, whilst on the other, mini-buses, pick-up trucks and lorries, handle a much smaller proportion. Apart from handcarts, small and intermediate capacity vehicles (such as bicycles, rickshaws and three-wheeled motorised vehicles) are scarce or absent.

iv) A survey of the availability of small engines in Sierra Leone would be appropriate to determine the potential to supply small-scale motorised vehicle manufacture. There is little doubt that small motorised vehicles could be developed to provide a useful alternative to cheap but slow headloading and expensive but fast pick-up trucks.

v) Training of artisans and small entrepreneurs, particularly in elementary book keeping and marketing, is of great importance to their longer term survival, let alone prosperity. Their failure to understand the consequences of poor pricing and sometimes even of the difference between sale price and profit, means that artisans often struggle to cover their costs and renew their tools.

By these means, the DTU feels that the level of rural transport in Sierra Leone may brought up to a level from which more sophisticated development can take place. There is little doubt that the present difficulty experienced by farmers attempting to market their spare produce, acts as a severe constraint to their drive to improve their standard of living. Incremental and evolutionary changes such as those suggested in this document are probably the best way to help them.

9. References

Adeoti JS, Kaul RN, Olukosi JO, Adebisi O. Characteristics of rural transport needs - a case study of the savannah region of Nigeria. Proc International Seminar on Rural Transportation, New Delhi,Indian Roads Congress and ILO, Geneva, April 1989.

Bangura AB. Constraints to the extension of draught animal technology in the farming systems of Sierra Leone. Proc of the third workshop of the West African Animal Traction Network, Saly, Senegal, July 1988.

Barwell I, Edmonds GA, Howe JDGF, de Veen J. Rural Transport in developing countries. Intermediate Technology Publications, London, UK, 1985.

Dennis RA. Human and animal powered transport in rural communities in sub-Saharan Africa. Proc human and draught animal power in crop production, Harare, Zimbabwe. Silsoe Research Institute, UK, January 1993.

Dogger JW. Final report: ox-cart testing activities: animal draught power research and development project. Magoya testing station, Republic of Zambia, July 1990.

Kanu BH. Village level engineering: the importance of the blacksmith in supporting animal traction and agricultural production. Proc of the third workshop of the West African Animal Traction Network. Saly, Senegal. July 1988.

Kanu BH, Sankoh FAR. Management of work oxen in Sierra Leone and its implication for research. Proc of the fourth workshop of the West African Animal Traction Network. Bagauda Lake, Kano, Nigeria. July 1990.

Malawi Bureau of Standards. Animal drawn cart specification. DMS 95:1986.

Oram CE. Low-technology rolling element bearings for animal powered transport and equipment. Proc Animal Traction Network for Eastern and Southern Africa. Lusaka, Zambia. January 1992.

Starkey PH. Animal-Drawn Wheeled Toolcarriers: Perfected yet Rejected, GTZ/Vieweg, Eschborn, Germany, 1988.

DEVELOPMENT

TECHNOLOGY

UNIT

Working Paper No. 43

African Oxcart Design and Manufacture.

December 1994

HS Pearson, CE Oram & TH Thomas

Development Technology Unit

Department of Engineering University of Warwick

Coventry CV4 7AL UK

Tel: +44 (0)203 523523 extn 2339   Fax: (0)203 418922

email: esceo@uk.ac.warwick.eng

Contents

FIGURES

TABLES

APPENDICES

PART A: ANIMAL CARTS IN AFRICA & THE BORNO CART.

SECTION 1: GENERAL COMMENTS.

1.0 Introduction.

This paper covers work carried out by HS Pearson and colleagues in northern Nigeria in 1993 and 1994. The paper is divided into two parts: part A (sections 1 to 4) discusses animal carts in general, the design options available for their various components, the particular circumstances affecting the development of the DTU Borno oxcart, and its main design features; part B is a set of full manufacturing instructions for the basic cart and some minor variations of it. We have written the paper because there is a dearth of detailed information about cart design and manufacture - especially information that identifies the context of use and manufacture and the assumptions underlying the approach used.

The DTU Borno cart incorporates a mix of novel and traditional features, and it draws heavily of course, upon designs used elsewhere. We believe it to be substantially cheaper than most alternatives of comparable performance. It has been moderately tested in the laboratory and in the field where production in significant volume started only in late 1993. However certain components were already well proven and others have been fairly easy to test for performance. Every reader of this paper intending to use the design elsewhere should compare their own context and objectives with those described below and feel free to adapt the design to their own circumstances. We believe nevertheless, that the specifications and constraints worked to in NE Nigeria are representative of those in many other parts of Africa.

1.1 Animal-Drawn Carts in Africa.

The use of animal-drawn vehicles for agricultural purposes in rural areas of Africa is relatively new when compared with the millennia-long history of using animals to pull carts, coaches and wagons etc in Europe and Asia. In Europe by the late 19th century, at the height of its development, the animal-drawn wagon required very skilled artisans for its construction. Though examples of these 'high tech' wagons can be found in some African countries (in particular the Republic of South Africa) the technology for construction never took hold amongst African artisans. Perhaps, because the technology took hundreds of years to develop in Europe, insufficient time was available to pass on the relevant skills into Africa before the motor vehicle swept away animal traction in Europe itself.

Some areas of sub-Sahara Africa will not support the use of animal-drawn vehicles because the topography is unsuitable, e.g. very hilly terrain, or because the area is not suitable for draft animals, because of disease. The most common impediment has been sleeping sickness transmitted by the tsetse fly, which is found in rain-fed forest areas.

Animal-drawn vehicles were first used in Africa at ports for the transportation of goods to and from ships. Even today, on balance probably more animal-drawn carts are used in urban areas than rural ones. There is little doubt that if they were more readily available and were cheaper and of better quality, animal-drawn carts would be used in much greater numbers by rural people.

In the absence of carts, rural people carry goods on their backs or heads - work often left to women. For moving goods and agricultural produce from the farm or homestead to the nearest town or motorable road, pack animals such as donkeys are sometimes used. Even where carts are available, women do not traditionally operate them, and still find themselves carrying loads.

After African independence, development organisations, in cooperation with African governments, have had many programmes aimed at improving designs of animal-drawn carts and increasing their availability to the poorer members of farming communities.

It has always been difficult to justify the expense of a purely agricultural cart, an investment perhaps larger than that of any other equipment the farmer uses (for example a cart can cost about three times as much as a medium sized plough). The ox-cart on the farm does not directly contribute to increased crop production as does theplough, where the relationship to an increased crop is more readily obvious. Moreover the farmer would require extra animals if he wished to use them for transport at the same time as preparing land.

But any farmer operating above subsistence level will need transport to get his/ her produce to the nearest town or line of road or rail. Outside the harvest season farmers living in villages or near towns often hire out their carts and animals, to be operated by a relative, for use in hauling goods other than traditional farm produce. Optimum use of the cart and trained animals is important in terms of income generation, but it does not eliminate the need to reduce substantially the costs of carts in Africa from their present high levels.

Since the moving of heavy loads, crops, water, building materials etc, forms a major part of the working life of African people, the importance of transport is self evident. Programmes aimed at cart development have generally been more successful than efforts in many other technologoes. The construction of animal-drawn carts is one of the few industries that can be found readily in most African countries and is undertaken by a broad spectrum of agents - from the blacksmith operating under a shady tree, to large urban companies.

Some African countries have a long history of cart making. For over sixty years in Madagascar, professional cartwrights have been making wooden spoked wheels with metal rims, of a type very similar to those which were formally made in Europe. In other countries, such as the Central African Republic, less skilled artisans with more limited tools, make solid wooden wheels by building up layers of timber and cutting them to shape. In recent years in most African countries however, parts from old motor vehicles have formed the basic materials for cart construction.

Many areas of Africa are heavily dependant on animals for transport and for agricultural purposes. Rural people in Ethiopia rely heavily on the use of animals for crop production - donkeys are extensively used as pack animals, as they are in semi-arid areas in West Africa. Because of the mountainous terrain and the lack of reasonable rural roads in Ethiopia, animal-drawn vehicles are usually used in urban areas. Donkey carts, horse-drawn two-wheeled taxis and flat-bed four-wheeled commercial carts, drawn by three horses are widely used here. The Sahel and Botswana are also rich in mule and donkey carts - in fact over much of Africa there is a realisation that animal traction is a sustainable technology that can deliver useful results without the costs associated with motorised transport. It is often forgotten that animal traction is new technology in many areas in Africa, ie it is *novel* and the detail of the technology must be developed from a low base. This present work is part of that technology development.

1.2 Main Types of Animal Cart in Africa.

Most carts now available in Africa originate from one of three sources:

- importation from overseas;
- importation of axle assemblies, complete with wheels and tyres, and sometimes including metalic body parts - wooden floor and sides added locally;
- local construction of entire carts using scrap parts from motor vehicles or simple locally-made bearing systems combined with wooden bodies.

The ideal would be full local manufacture using designs at various technological levels to suit different manufacturing facilities and financial situations.

Animal carts are only one of many types of non-motorised vehicle for carrying freight. Such vehicles may be grouped into three classes:

- Human-propelled devices such as wheel-barrows, water-carts, bicycle-carts and hand push or pull carts.
- Horse-drawn wagons and carts. Horses are not widely used in African countries because of disease and high feeding costs, also horses are not as robust working animals as donkeys and oxen.
- Other animal-powered vehicles such as donkey carts, ox carts and wagons for one or two animals.

This report focuses on the third class: donkey and ox-carts and wagons for use with one or two animals.

SECTION 2: THE DTU BORNO OX-CART.

2.0 Background to Development.

Borno State (the site of the activity for the present project) was until recently a very large part of Nigeria but has recently been divided into two smaller states, Borno and Yobe. The Northern part of Borno is dry Sahel Savanna containing three towns, many agricultural villages and Fulani (nomad) camps. The landscape is flat, the soil is sandy and the population density is quite low. There are a few sealed roads, but most villages are at least 10 km from the nearest one. Motor vehicles and motor fuel are quite plentiful in Nigeria (although there are serious fuel shortages from time to time), but there is a substantial demand for animal carts for local movement of crops, fuel, water and building materials.

In 1990 the Nigerian Government and the European Community established a large interlinked rural development programme in North Borno entitled the North East Arid Zone Development Project (NEAZDP). Improving rural conditions, services and incomes were major aims written into the programme. The DTU Borno Cart was developed out of cooperation between the DTU (Warwick University), a new Centre of Appropriate Technology (RAMCAT) at Ramat Polytechnic in Maiduguri (the Borno State capital), and NEAZDP field staff. NEAZDP has also been involved in increasing the number of draught oxen in the arid zone, training urban and rural artisans, and establishing a market distribution system for 'rural equipment'.

Carts are already made in several villages in the arid zone. They use wheels and axles from old pick-ups which are combined with a timber body. The supply is inadequate to meet demand and prices are quite high, yet the assemblers (carpenters) get a poor living from making them. Poor availability of components is a major constraint on output.

Nigeria has a substantial formal industrial sector and, even in relatively isolated Borno State, there are several machine shops in Maiduguri and a couple in smaller towns, such as Gashua. There is also some informal industry, including aluminium casting. Trends in the Nigerian economy discourage ongoing dependence on imported components and articles.

2.1 Specification of Borno Ox-Cart.

The DTU Borno Cart was therefore developed against the following specification:

- to be assembled by village carpenters who will also continue to produce cart bodies (an urban fabricated alternative body was also required);
- to carry loads of up to 1 tonne (1 metric ton = 1 000kg - very nearly the same as an Imperial ton) when pulled by two well fed oxen over flat sandy tracks:
- to use pneumatic tyres, as these are readily available, repairable and non-damaging to sealed roads:
- to make full use of Borno State's existing industrial capacity and not be heavily dependent on imported items:

- to cost less than $\$_{US}150$ to produce (in 1993):
- to reach volume production within 3 years (of 1991) and therefore to use techniques and components that are rapidly provable or already proven elsewhere in Africa.

This working paper has been written in the belief that these conditions in NE Nigeria are not markedly different from those in many other parts of Africa and therefore that both the design and our experiences from other countries will be of interest to organisers of animal drawn transport programmes elsewhere in Africa.

2.2 Interaction Philosophy during Project.

The technology-transfer philosophy adapted in this programme was:

- to aim at durable products of moderate price rather than short-lived ones of extremely low price;
- to employ the highest levels of technology sustainable in the area- for example to use jigs and fixtures to aid production, which themselves require a certain amount of skill to reproduce or modify;
- to upgrade the general skills of the casters, fabricators and assemblers involved in cart production so that they might also improve their range and output of other products;
- to encourage interchangability of parts by adopting local standards wherever possible - to multi-source components and avoid reliance on any single producer;
- to minimise dependence on imported materials particularly those whose market availability is erratic.

2.3 Output from Present Project.

The Nigerian economy and the institutions involved were subjected to many changes in the period 1991/94 and the objectives adopted in 1991 were not fully achieved. However the cart, whose manufacture as described in detail in Part B of this paper, had been produced in some numbers (over 20) by July 1994 and some hundreds of operating hours had not revealed any major problems in use. One manufacturer of axle sets, one caster of aluminium hubs and two cart body builders were trained and supplied with the necessary jigs and moulds to attain good quality production. However extension of the production of running gear (axles, hubs, bearings, wheels) to several competing urban manufacturers has still to be arranged, as has the training of significant numbers of village carpenters to incorporate the new cart components into their production.

SECTION 3: CART DESIGN OPTIONS.

3.1 Wheels.

In general, fabricators of animal drawn vehicles in sub-Saharan Africa, use second-hand wheels from motor vehicles. Such wheels often come attached to an old rear or front axle, which is also used in the cart's construction. Wheels entirely manufactured in African countries specifically for use on animal drawn vehicles are not so widespread. Several designs however, have been adopted and used with various degrees of success and popularity. These are identified as follows.

3.1.1 Solid wood wheels.

Solid wooden wheels with solid rubber tyre made from old motor vehicle tyre outer cover. This wheel is constructed using layers of flat planks that are cut and bolted together to give the required thickness of about 100mm - with the grain of successive layers set at right angles for strength. The diameter of these wheels is about 1 meter, or a little more. Bearings are likely to be of a simple bush type incorporated in the hub of the wheel.

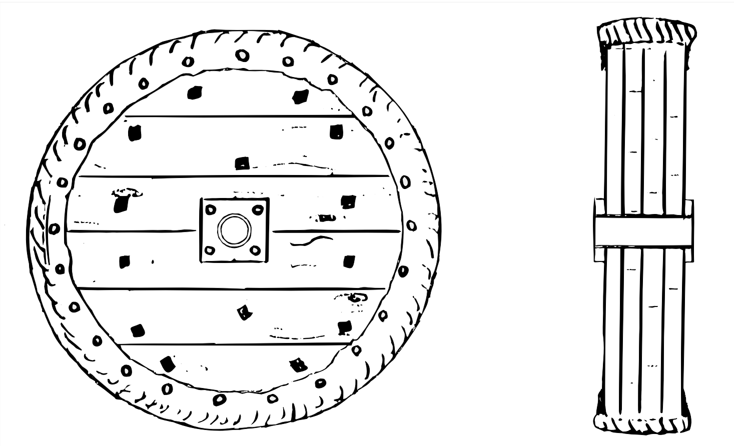


Figure 1: solid wood wheel.

Advantages: easy to make with readily available materials, simple to repair. Good for use over rough terrain or in remote areas where punctures to pneumatic tyres may be a problem to repair.

Disadvantages: there are limitations to the types of bearings that could be used successfully as a more complex hub structure would be required. On hard road surfaces the cart would ride very roughly, transmitting large shock loads to the bearings. The wheels are very heavy (100kg).

3.1.2 Fabricated wooden wheels.

Fabricated wooden wheels of traditional European design (Fig 2) with a heavy wood hub supporting wooden spokes fitted into a wooden sectioned rim with a narrow steel band as a tyre. The European spoke wheel is normally dished, whereas the Asian and African versions are not.

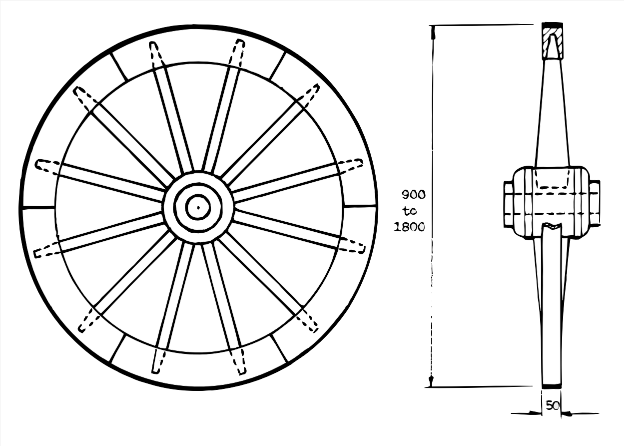


Figure 2: wooden spoked wheel.

This design of wheel has not been produced in large numbers in Africa. They were most prolific in South Africa where they were used by settlers from Europe, in particular the Boers, on their large ox wagons. Cruder versions of the spoked wooden wheel are however still made in some African countries. These wheels are usually fitted with thin solid rubber tyres cut from old commercial vehicle tyres.

Advantages: good for use in remote areas or over rough ground. Much lighter than solid wheels and more flexible under shock loads.

Disadvantages: requires considerable woodworking skill in manufacture and repair. 19th century European cart makers underwent a five year apprenticeship. Uses well-seasoned high-quality wood, that is becoming scarce in many countries.

3.1.3 Fabricated steel wheels.

Fabricated steel wheels with flat steel rims, 800mm to 1000mm diameter with a 100mm wide rim. Welded or blacksmith riveted construction, Figure 3. The fabricated steel wheel is still occasionally manufactured by large companies in developed countries for use on tractor-drawn agricultural equipment. Designs suitable for use on animal drawn carts have been produced in many African countries. Given reasonable construction they will last for many years without giving trouble, but the joints between spoke and rim can give trouble.

Advantages: fairly easy to construct and obvious.

Disadvantages: shock loads from bumps fed to bearings, fatigue of welds between spokes and rim, damage to road surfaces, noisy, bumpy ride.

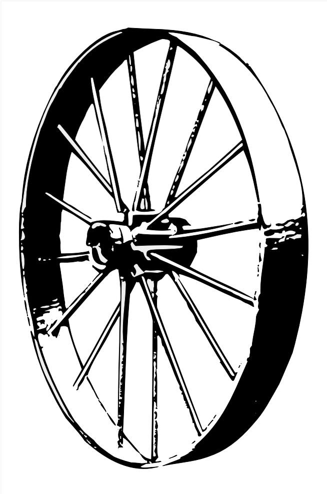


Figure 3: spoked steel wheel.

3.1.4 Fabricated all-steel split rim.

This rim is designed to be fitted with a commercial inflatable tyre, and is shown in Figure 4. Joseph Wirth, a development engineer working in Tanzania, designed a split rim fabricated wheel in 1965. Many hundreds have been manufactured in Tanzania and other countries since then and used on animal drawn vehicles. Very similar is the design by Intermediate (Technology) Transport. Both models use flat strips and round bars in their construction. Most popular sizes are for 16" (Land Rover) and 14" (Peugeot) tyres.

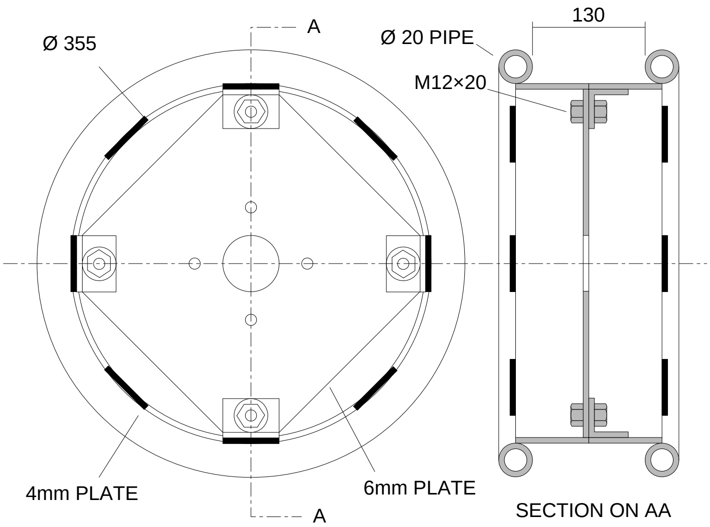


Figure 4: ITDG steel split rim for use with pneumatic tyre.

Advantages: split-rim design simplifies tyre repair. May be used with a selection of bearing and hub configurations. Use of pneumatic tyres gives an improved ride and a better rolling action.

Disadvantages: requires a workshop equipped with electricity for welding and drilling. The metal sections of the wheel have to be bent in a special, and quite expensive bending tool (e.g. that designed by IT Transport for the purpose) or a conventional rolling machine. The wheel is material intensive, using up to 5 different metal sections, and is heavy weighing about 25kg.

3.1.5 Pressed steel wheels for local manufacture.

This design (Figure 5) was developed at CAMARTEC in Northern Tanzania in 1988 and several hundred are in use. The construction of the wheel is very simple and is of a split rim design. Rims for two popular tyre sizes are available, 16" (Land Rover) and 14" (Peugeot).

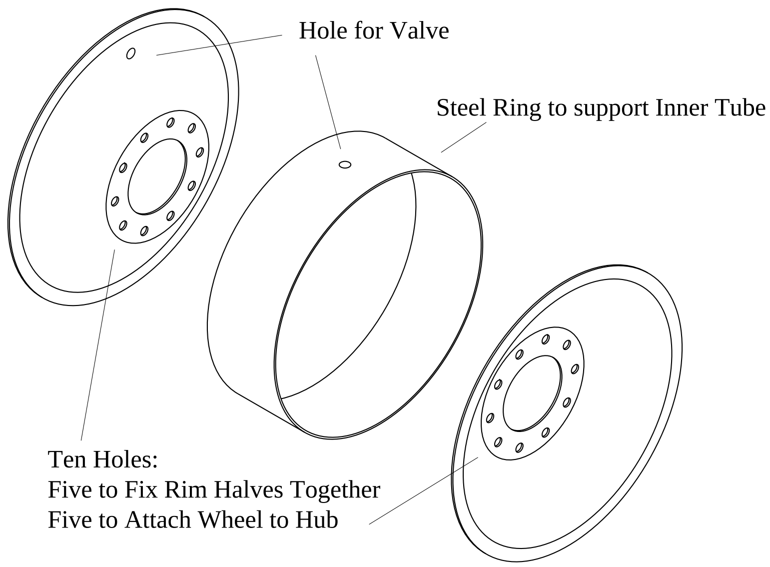


Figure 5: DTU pressed steel wheels for pneumatic tyres.

Advantages: split rim design simplifies tyre repair. May be used with a selection of bearing and hub configurations. Use of pneumatic tyres gives an improved ride, a better rolling action and lower shock loads on bearings. One size or material, 3mm steel plate, is used throughout the wheel's construction. Simplicity of design will allow many different sizes to be made using the same material and methodology. Adapts easily to small or large batch production. Light weight 12 kg.

Disadvantages: Requires a workshop with electrical power and access to an hydraulic garage-type press of not less than 40 tonf capacity (1 tonf is a unit of force = weight of 2240 lb). Not suitable for manufacture in most rural environments.

3.1.6 Wheel Chosen for Borno Cart.

The pressed steel wheel design was selected as the most suitable design for use on the DTU Borno oxcart. This decision was based upon previous experience in Tanzania and the availability of facilities and materials availability in the project area, and was taken to reduce wheel costs whilst retaining the advantages of pneumatic tyres already experienced in NE Nigeria.

3.1.7 Other Wheel Developments.

An extension of aluminium casting technology is presently being researched is the all-in-one cast aluminium wheel and hub shown in Figure 6. In this design the bore of the hub is unmachined and rollers bear directly on it and on the pipe axle. Test wheels have survived over 4 000 km of continuous running to date.

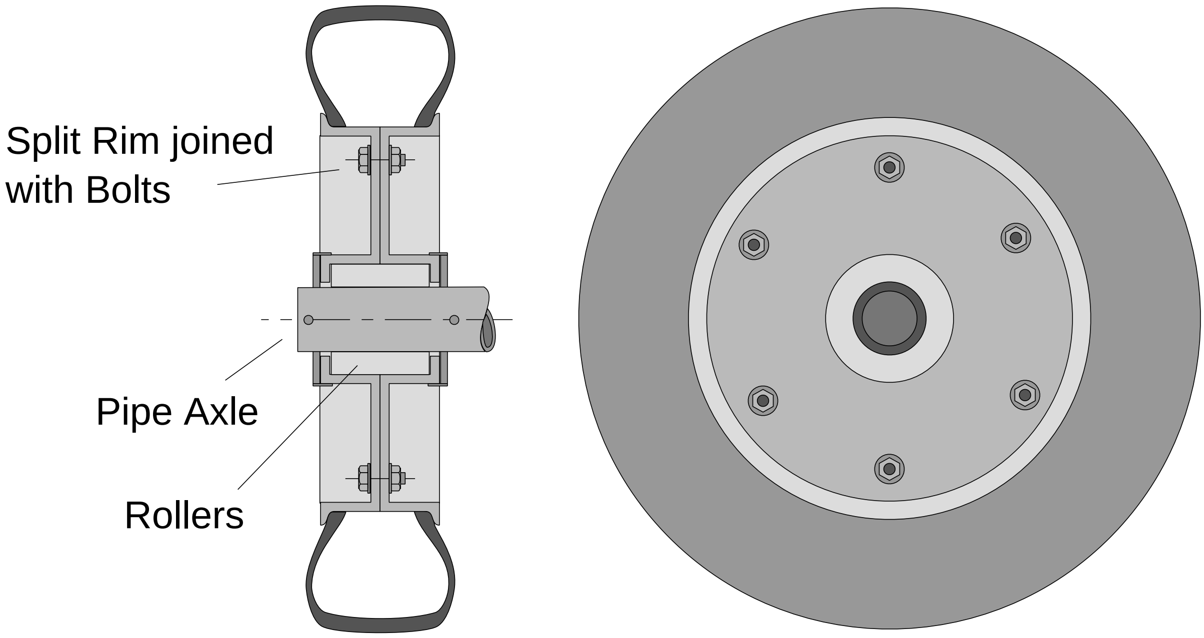


Figure 6: cast aluminium split rim for use with pneumatic tyre.

3.2 Hubs.

The design of a hub for local manufacture will depend upon the manufacturing climate, materials availability, bearing system, and wheel fitment. Some of the more well known designs are discussed below.

3.2.1 Traditional wooden hub.

Traditional wooden hub used in conjunction with wooden spokes and rim. Used on carts, wagons and coaches. Popular at the turn of the century in the northern industrialised nations and fitted to carts used by European settlers in Africa.

Advantages: strong and easy to maintain, will support heavy loads at slow speeds.

***Disadvantages*:** requires skilled craftsmen and good quality wood for its construction. Limitations to bearing designs that can be used.

3.2.2 Simple pipe hub.

Simple pipe hub with a flange welded to it for mounting the wheel. This design is often used in conjunction with a wood bearing pressed into the hub pipe and used in conjunction with rigid wheels of wood or steel.

Advantages: can be manufactured locally. Very simple in design and construction and cheap to build. Easy to service.

Disadvantages: depends on availability of reasonable workshop facilities and materials supplies. Limitations to the types of bearing that can be fitted.

3.2.3 Fabricated machined steel hub.

Fabricated machined steel hub but with more sophisticated mounting facilities to incorporate bronze bush, roller or ball bearings.

Advantages: can be manufactured locally. This design is adaptable for various bearing designs. Used with roller or ball bearings can run at higher speeds.

Disadvantages: advanced workshop facilities required with more complex fabrication and increased manufacturing costs. Regular supplies of raw materials may be a problem.

3.2.4 Cast or forged hub.

Cast or forged hub fitted with bush, roller or ball bearings. Cast hubs are used on all commercial high speed vehicles, with the bush-bearing hub used on slow speed agricultural equipment.

Advantages: provided that foundry facilities are available, this is very cheap to produce in a wide variation of designs for different bearing configurations and wheel fittings.

Disadvantages: requires foundry and machining facilities. Unsuitable for rural manufacture.

3.2.5 Hub Chosen for the Borno Cart.

A cast hub design using aluminium was selected for the Borno oxcart because of the availability of facilities, the raw materials cost, and the product quality required.

3.2.6 Other Hub Developments in the Programme.

A development of the fabricated hubs discussed above is being pursued by the DTU and is shown in Figure 7. Here low steel requirements have been achieved by mounting the wheel fixing studs on struts welded tangentially to the hub tube. This method has been shown to be cheap and reliable. A further important development has been the use of non commercial roller bearings, discussed briefly in a later section. The hub and bearing are in advanced stages of development and testing, but are not yet ready for widespread use.

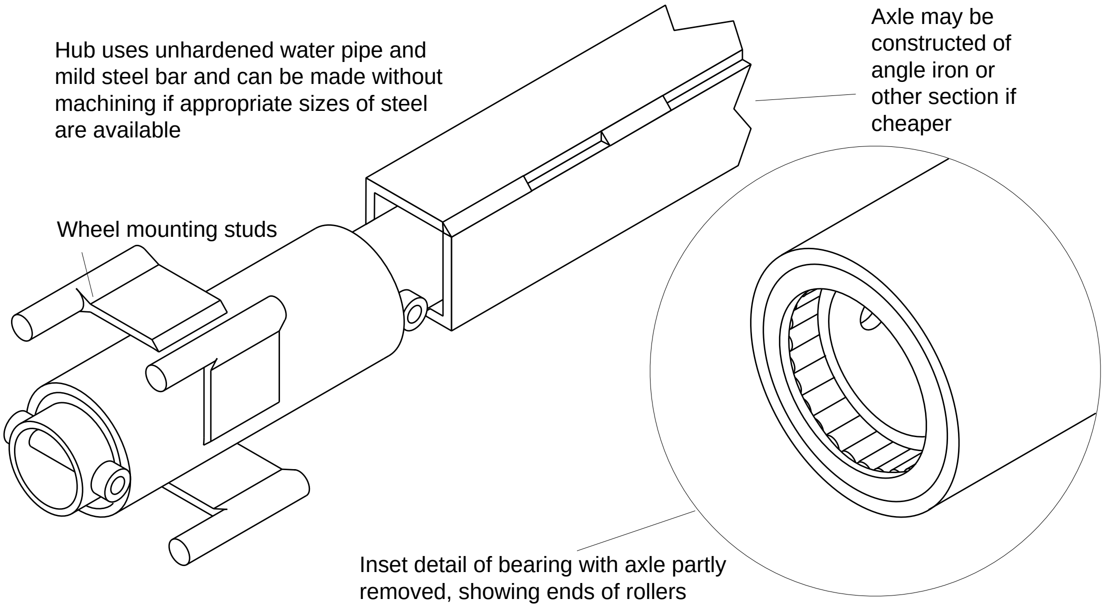


Figure 7: fabricated pipe and roller axle.

3.3 Bearings.

There are many bearing systems available that have been tested for use on animal-drawn carts, the following is a list of the more successful.

3.3.1 Steel pipe or solid axle running in oil soaked wood bearings.

These hubs can be arranged with the bearing (Figure 8) fixed in the hub of the wheel and rotating on a fixed axle, or with four sets of split block bearings arranged on two short axles, these rotating with their respective wheels.

Advantages: simple to make and will last reasonably well, providing hardwood is used and lubrication is adequate. Easy to service in rural areas where skills and tools may be limited.

Disadvantages: suitable hard wood may not always be readily available (eg many tropical hardwoods have silicate inclusions which cause rapid wear to mating shafts). The hub-fitted bearing requires some skill in manufacture, particularly if the spokes of the wheel are incorporated into the hub. The bearings show quite high friction, especially under load, which makes the cart hard to pull and reduces popularity with users. With four block bearing and overhung wheel designs, the outer bearings carry loads greater than the wheel load, which gives rise to high wear. This wear problem is often aggravated because most wood bearings are open to the ingress of dust and dirt.

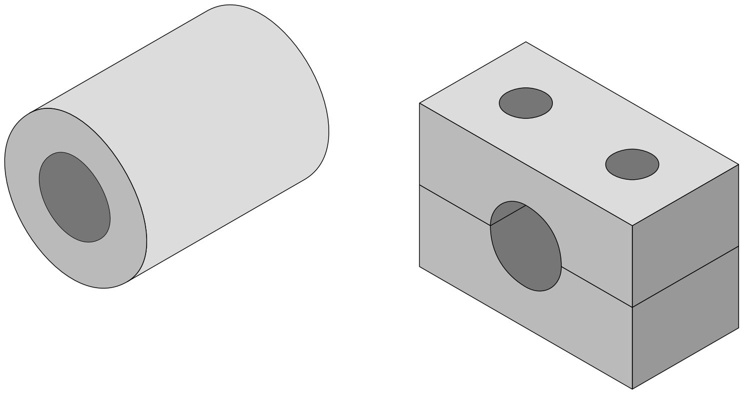


Figure 8: wooden bearings.

3.3.2 Solid steel axle running in a heavy-duty cast iron hub or bearing.

Figure 9 shows the next generation of bearing design after the wooden bearing. Because of its free graphite, cast iron is to some extent self lubricating, and runs well against other bearing materials such as steel, bronze or even itself. This type of bearing has been extensively used on agricultural equipment such as planters, harvesters and harrows.

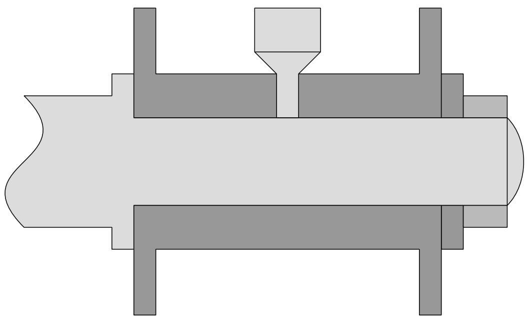


Figure 9: steel axle in cast iron hub.

***Advantages**:* using cast iron allows for intricate hub shapes with the bearing integral. It wears well provided that lubrication is adequate.

***Disadvantages**:* requires iron foundry facilities and skills such as pattern making and moulding and a source of raw materials. The bearing system should be well lubricated and sealed against dust and dirt. There are some frictional losses and excessive wear in the integral bearing could necessesitate replacement of the entire hub.

3.3.3 Steel axle in cast iron/ steel hub with bronze bush bearings.

This system (Figure 10) is a natural development from option 3.3.2 above. For interchangeability separate bearings are made and fitted into a cast iron or steel hub. Until the advent of rolling element bearings this system was used extensively in carts, wagons, agricultural equipment, and stationary steam engines worldwide.

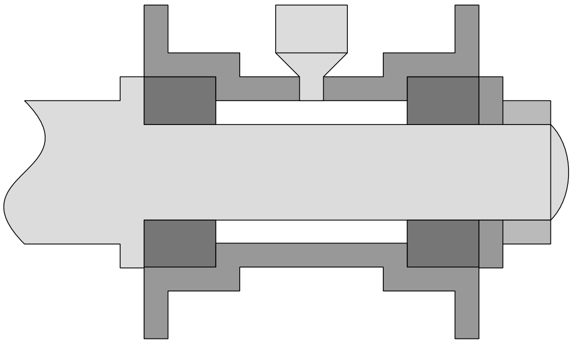


Figure 10: steel axle in cast iron hub.

Advantages: bearings are cheap to manufacture, fairly easy to replace and will tolerate heavy loads at low speeds. Different combinations of bearing materials can be used.

Disadvantages: regular lubrication and the exclusion of grit and dirt are very important. A breakdown in lubrication can lead to damage to the hub, and rapid wear. These bearings show significant frictional losses and require extensive workshop facilities and skills in manufacture.

3.3.4 Steel axle with ball or roller bearings.

Such an arrangement is shown in Figure 11. Rolling element bearings operate by interposing rollers or balls between the fixed component (usually the axle and the rotating component, (usually the hub). This reduces the frictional losses and results in a very free running bearing with a long life.

Advantages*:* Tapered-roller and cup-and-cone (or angular-contact) designs are adjustable and will accommodate axial as well as radial loads. With minimum maintenance or lubrication they can last a very long time if correctly sized and are particularly good when used in slow vehicles such as oxcarts.

Disadvantages: such bearings require complex machinery and accuracy and good quality control in their manufacture. Bearing components must be heat treated and fitted correctly. Heavy shock loading during installation or use can cause rapid destruction of these bearings.

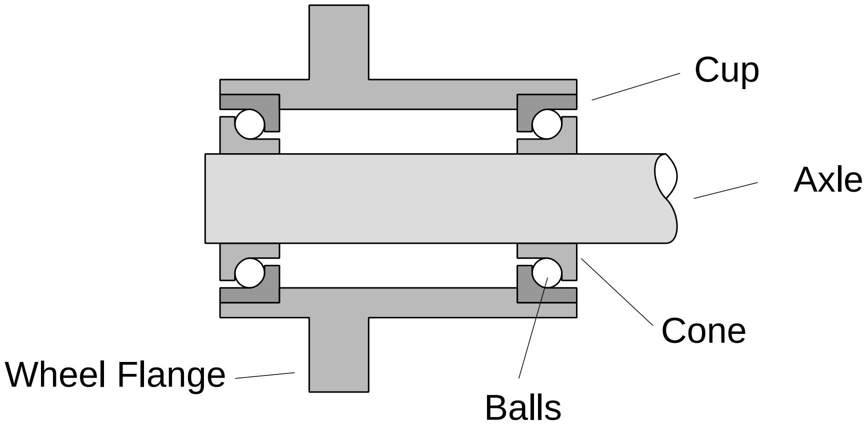


Figure 11: cup and cone bearings.

3.3.5 Bearing Option Chosen for Borno Oxcart.

Locally manufactured cup-and-cone ball bearings were selected as most suitable for use on the DTU Borno oxcart. This decision was based on availability of manufacturing skills and materials, performance and potential lifespan of the finished product.

3.3.6 Other developments in programme.

As mentioned above and shown in Figure 7, the DTU is currently developing a system of low-cost rolling-element bearing which can be made without machine tools. Limited trials of these axles have been made in Africa with encouraging results, but further work is needed before designs can be fully released.

3.4 Bodies.

Once a suitable axle has been obtained or built a body will be required. Some of the more popular options are listed below. The dimensions of cart bodies vary from country to country but are approximately as follows: width 1.3 to 1.5 m, length 1.8 to 2 m. Underbody clearance should be as much as possible but should not put the loading platform of the cart unrealistically high, ie above 700 mm. Variation in vertical load on the animal, as bumps are traversed, also increases with load tray height. The main types of cart body used in Africa are discussed below.

3.4.1 Scrap motor vehicles.

Scrap motor vehicles are a common source of oxcart bodies utilised in African countries. A scrap body, salvaged from an old small truck or pick-up, is often used in conjunction with the rear axle assembly from the same or similar vehicle.

Advantages: generally found in towns or near main roads where there is a good supply of scrap vehicles. Strong and able to outlast most axle systems, the owner will transfer the body as the axles wear out. Suitable for urban construction.

Disadvantages: use of scrap bodies can only be considered in the construction of limited numbers of non-standard carts, because of lack of local availability in large quantities. Often very heavy and best used in conjunction with a ball or roller bearing axle.

3.4.2 Wooden flat bed.

Wooden flat bed without sides made up of 50 mm ×100 mm wooden beams and 25 mm thick planks bolted and nailed together. Often mounted on a scrap rear or front axle from a motor vehicle.

Advantages: easily made and repaired under rural conditions. Can be built to suit different axle sizes, can handle bulky but light loads and may be inexpensive.

Disadvantages: without sides the underbody requires rigid bracing. Not suitable for carrying loose loads such as sand or grain.

3.4.3 Wooden box body.

Wooden box body similar to that above, but with sides, head and tail-board.

***Advantages*:** all the advantages of that above but capable of carrying a wider range of loads.

Disadvantages: can become material intensive and heavy if the construction and design is not carefully thought out.

3.4.4 Bodies of composite construction.

Bodies of composite construction using steel sections of angle iron, flats or box tubing welded and/or bolted with wooden planks for floor and wooden sides bolted on.

Advantages: providing the design is well thought out the body will be strong and light.

Disadvantages: requires the constructor to be skilled in metal fabrication as well as having some woodworking skills. Less suitable for rural construction, where steel sections and welding facilities may not be readily available.

3.4.5 Choice of Body for the Borno Oxcart.

The Borno ox-cart can be fitted with either of two DTU designed bodies shown below, one of welded steel box tubing and wood planking (Figure 12); the other of all-wood construction (Figure 13). Both designs have short construction times by virtue of their small component counts and simple manufacturing processes. For example only square cuts are required at the ends of all components and none are required to be cut more accurately than about 5mm.

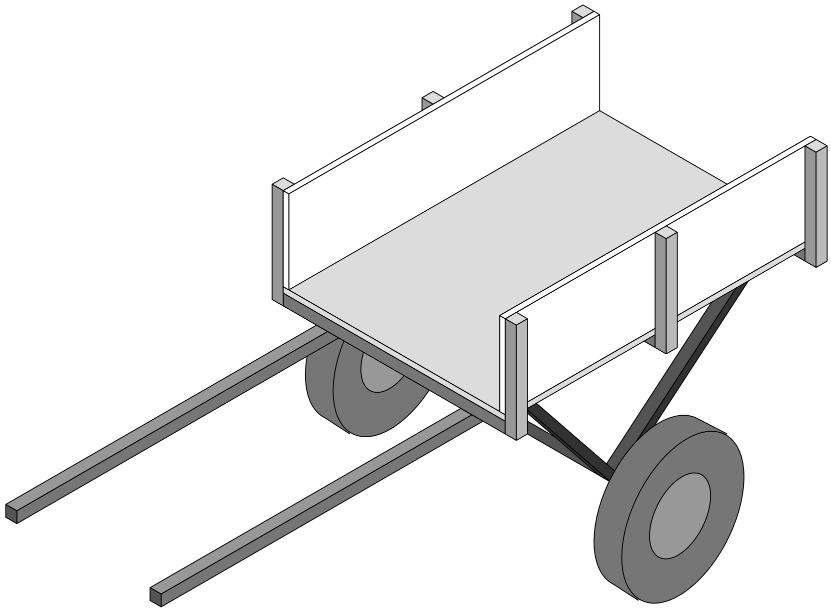


Figure 12: steel box tube framed cart with wooden planking.

In both carts, clenched fixings are used to hold the wooden components together or to the steel frame. This method is quick and sraightforward. Lengths of 6mm diameter steel wire or rod (such as that widely used in concrete reinforcement) are inserted into holes drilled in the wood (the drill can be made from the same rod) and the ends of these bent over using a hammer. The wire can then be tightened using a second hammer as if tightening a rivet.

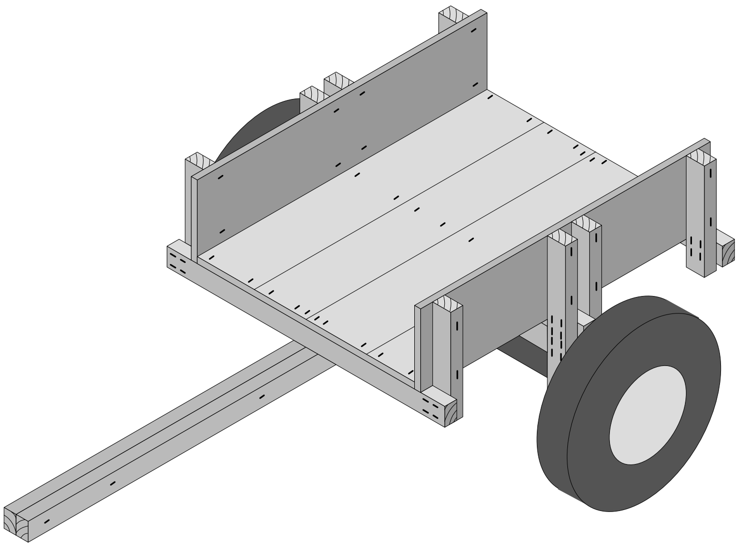


Figure 13: all wooden cart using clenched wire fixings.

3.4.6 Overall Cart Performance.

Testing both in the UK and in Nigeria of all-wood and of combination designs of cart have shown few problems so far. The all wooden design has been loaded with 400 kg of sandbags and towed over a repeated-kurb track at speeds of up to 15 km/h without damage, and the combination design has been supplied to about twenty farmers who have used the carts for up to one year at the time of writing.

SECTION 4: DESIGN DEVELOPMENT AND TESTING.

4.1 Pressed Wheels: Previous Experience.

Development of locally manufactured pressed steel wheels began when the first author was working at CAMARTEC, Arusha, Tanzania in 1988/90 with the United Nations. Development demands in Tanzania called for an improved wheel design to supersede the dated steel fabricated one similar to that shown in Figure 4 that had been manufactured locally since the mid 1960's. Between 1988 and 1990 tests were carried out on pressed steel wheels by running them, complete with an inflated tyre, on a static test rig. The wheel under test was run, day and night, at a speed 9 km/h (double that of an animal drawn cart) with a load of 300 kg. A bump was introduced to simulate partially rough road conditions. Tested wheels covered over 3 000 km trouble free. Figure 14 shows the Tanzanian test rig.

Pressed wheels were also tested in Tanzania by use with two wheeled trailers towed behind Station's tractors. These trailers were often loaded in excess of one ton and taken over very rough ground. Since 1990 several hundred wheels have been produced and used on animal-drawn carts.

The DTU programme in Nigeria has further developed and tested pressed steel wheels over the period 1992/4. Two sizes of wheel (16" and 14") are now available, both manufactured from 3 mm steel plate.

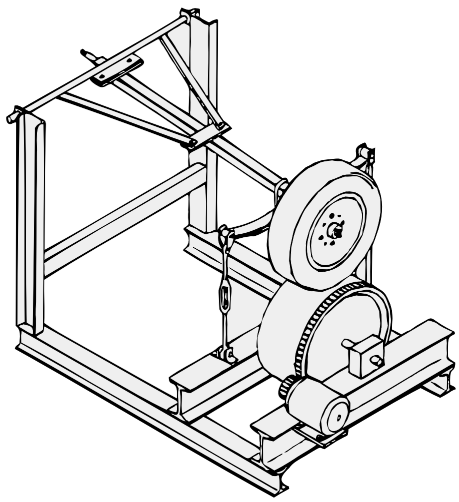


Figure 14: rolling road test of wheels and bearings.

4.2 Safety Tests on Wheels.

The DTU pressed steel wheel is a split rim design with the two halves of the wheel bolted together. With a tyre fitted and inflated to 2 bar (30 psi) a force of several tons is exerted on the rims, trying to push them apart. Concerns have been expressed that a potential danger exists to anyone attempting to split the rim halves with the tyre inflated. Clearly this is unlikely because the wheel is not usually taken off the cart unless the tyre is punctured. Nevertheless initial tests were conducted in Nigeria to determine the results of an attempt to split the rim with an inflated tyre in place.

These tests and findings are discussed below.

4.2.1 Test of complete wheel with fully inflated tyre at 2 bar.

Purpose of test: to determine the restraining capabilities of the four 10 mm Ø (diameter) bolts.

With the wheel securely held in a safe manner, an attempt was made to undo the four 10 mm Ø bolts. The force on the bolts was high and progressive and could be felt in the force required to undo them. It was obvious to those observing that all was not well. Three of the bolts were removed completely and the fourth undone half-way. At this point the two halves of the wheel sprang open on the unsupported side, allowing the inner-tube to escape into the inside of the rim and there rupture. The single 10 mm Ø bolt maintained the wheel's integrity.

Conclusions: the four retaining bolts are strong enough to hold the wheel together whilst inflated. A single bolt is strong enough to keep the wheel assembly together, albeit with some distortion to wheel and bolt.

4.2.2 Complete wheel with fully inflated tyre at 2 bar.

Purpose of test: to determine at what point the inflated inner tube would rupture upon separation of the wheel halves.

The wheel was placed in a hydraulic press with the two halves of the wheel held together by the force of the press. The tyre was then inflated to 2 bar (30 psi) and the restraining force on the wheel gradually released, allowing the two halves of the wheel to come apart. At a point when the wheel half discs had separated by 25 mm the inflated inner-tube began to migrate into the inside of the rim, rupturing on an unprotected sharp edge.

Conclusion: It was found that once the wheel discs had separated by 25 mm or so the inner-tube would expand into the inside of the wheel and possibly rupture against a sharp edge.

4.2.3 Complete wheel with fully inflated tyre at 3 bar.

Purpose of test: to measure wheel rim movement as tyre is inflated.

A complete wheel, supported at its centre, was slowly inflated to 3 bar (45 $lbf.in^{-2}$) and deflection measurements taken at the extreme edge of the of the wheel rim. A dial indicator showed that, under inflation, the rim halves moved apart 1.5 mm in total. The movement was progressive during inflation, but stopped as soon as the compressed air flow ceased.

Conclusion: the very small amount of movement was expected. Any movement during service occurs under the tyre bead and is therefore unable to induce puncture in the inner tube. If movement must be eliminated the width of the inner-tube support ring shown in Figure 5 can be increased slightly and the wheel discs given a small pre-load during assembly of the wheel.

4.2.4 Recommendations.

1) The four bolts holding the two halves of the wheel together should be long enough that if undone with the tyre inflated, the rim halves will come apart more than 25 mm before the nuts can be removed from the bolts.

2) A sharp component (a small spike) should be welded inside the inner rim to rupture the inflated inner tube before the bolts being fully undone.

3) A notice should be printed clearly on the wheel in the appropriate language, warning of the dangers involved in splitting a wheel with the tyre inflated.

4) Develop the design to make it impossible to split the wheel with an inflated tyre fitted (these design changes have been made but not practically implemented as yet).

4.3 Mk 4 Cast Aluminium Hubs: Testing.

The use of cast aluminium as a material for a low speed vehicle hub as been questioned on the point of strength. In order to address the issue some basic tests have been carried out in Nigeria and at Warwick and these are described below.

4.3.1 Shear test on lugs.

A Mk 4 hub was mounted (axis vertical) in a 60 tonf hydraulic press so that the ram applied load to one wheel mounting lug. In two tests the average force required to fracture the lug from the body of the casting was 17.5 tonf.

4.3.2 Casting strength around wheel studs.

A Mk 4 hub was mounted (axis horizontal) with one 12 mm Ø (diameter) wheel stud under the ram of a 60 tonf hydraulic press. In tests with two different presses the average force required to push the 12 mm Ø steel stud sideways out of the casting was 2.5 tonf. The stud suffered some bending before the casting gave way.

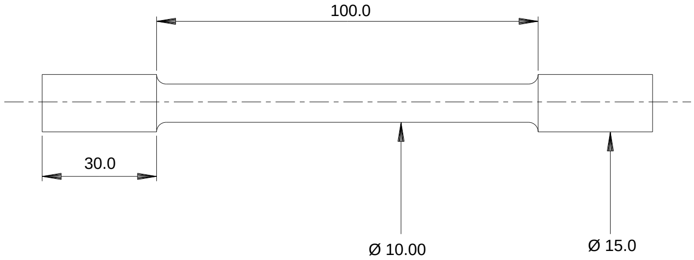


Figure 15: test piece: strength of aluminium castings produced in Nigeria.

4.3.3 Tensile test on samples taken from a Mk 4 hub.

In these tests carried out at Warwick, two samples taken from the hub were machined as shown in Figure 15 and tested in tension. Failure occurred at about 165 MPa at approximately 3% strain - well within the range expected for good quality alloy castings.

4.3.4 External analysis of five samples taken from a Mk 4 hub.

Results of these tests on five samples are presented in Appendix 1, and show the alloy to have about 8.5% silicon, 2% copper, 0.8% iron and 0.6% zinc, plus traces of many other elements. This is typical of the sand-casting alloys used in automotive components from which the scrap metal is obtained.

4.3.5 Conclusions.

The strength of cast aluminium hubs is well up to the loads, even at extremes, likely to be encountered in animal drawn carts, or even tractor drawn trailers. A regular good casting quality can be expected from the small aluminum foundries in the Nigeria project area. Monitoring of casting quality would be required if this technology were used in other countries for the production of hubs.

4.4 Bearings.

4.4.1 Recent Developments.

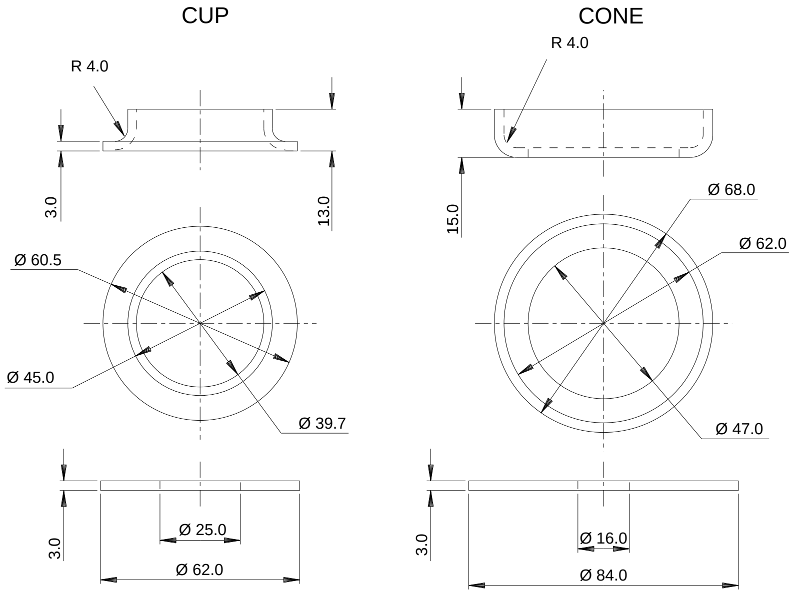
Prototype cup and cone bearings made at CAMARTEC, Tanzania in 1989 were put on a test rig (Figure 14) and run at 9 km/h with a load of 300 kgf. The drive roller of the test rig had a bump on it to simulate something of rural road conditions. An inspection was made after 3,000 km continuous running. The bearings were in very good condition with very little sign of wear. These bearings had cups and cones machined from solid mild steel bar, case hardened in charcoal as described below, and fitted with 17 × 8 mm diameter balls.

An identical set of bearings were fitted to a tractor drawn trailer for use by CAMARTEC in Sepember 1989. In just under one year the cart covered 500 km, often with loads in excess of 750 kg. Whilst the distance covered was not great, the speed, load and road conditions were very severe. The bearings fitted to this cart were inspected a year later in September 1990 and found to be in good order. Communications with CAMARTEC in 1993 confirmed that the cart is still in daily use, fitted with the original bearings. By 1990 an initial order for 25 ball bearing axle systems was in production and to date about 200 axles have been produced.

4.4.2 DTU Cup & Cone Bearings.

Development and testing of cup and cone bearings continued under the RAMAT/ Warwick linkage programme.

Early in 1993 two complete sets (for one axle) of cup and cone bearings were made in the Warwick Department of Engineering workshops to the dimensions shown in Figure 16 and heat treated with 'KASNET' (a proprietary carbon bearing formulation specifically developed for case-hardening).



NB Blanks (shown below components pressed from them) must be annealed before pressing. Case harden after pressing in charcoal plus 1% sodium carbonate for three hours at 850/900°C. Allow to cool in the carburizing box, then remove, re-heat to 800°C and quench in oil.

Figure 16: cup-and-cone bearing components.

Loaded with 17 × 8 mm diameter balls the bearings were mounted in DTU design cast aluminium hubs imported from Nigeria. The hubs were fitted with Land Rover wheels and run on the DTU bearing test rig (Figure 18) with a load of 200 kgf per hub at 120 rpm for 32 000 km. Inspection showed that wear was advanced, in particular on the cones, which had worn severely (approximately 1.5 mm) on the loaded side. The 8 mm diameter balls did not show any signs of wear. Grease had been packed into the bearings before the start of the test - the rig being stopped only to check for bearing free play and for adjustment of the bearings about mid way.

Pressed cups and cones, made from 3mm thick steel sheet and carburized with charcoal as described below, were tested next. Being very thin, these pressed components allowed an increase in the number of 8 mm balls used from 17 to 20, whilst also allowing thick walled pipe to be used for the stub axle. (More expensive 30 mm diameter solid bar was required with the cones machined from solid material.)

The bearings were fitted into Nigerian cast aluminium hubs and run at 80 rpm at Warwick over a period of seven months loaded to 200 kgf. A complete inspection showed very little wear with lubricant still in place, when the rig was stopped at 30 000 km.

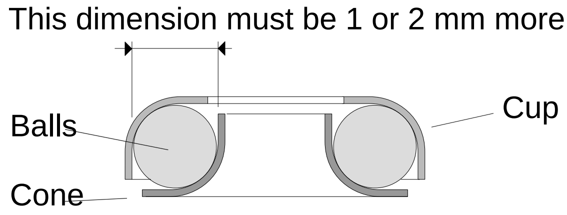


Figure 17: essential cup-and-cone bearing dimensions.

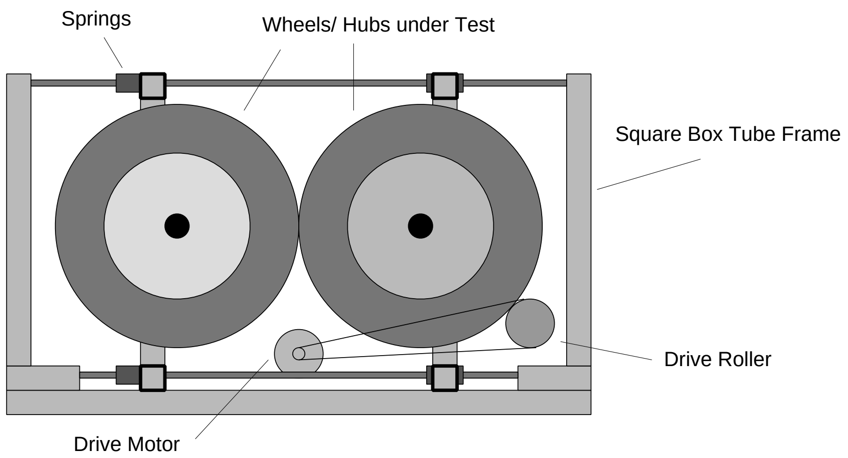


Figure 18: DTU wheel, hub and bearing test rig.

4.4.3 Tests on Locally Available (Nigeria) Bicycle Balls.

Commercially available bearing balls normally have a hardness of about 55 Rockwell C. The most readily available balls in Nigeria were those sold widely and cheaply for use in bicycles. These balls are 1/4" diameter and come in small tins of about 150 units from China and sometimes from India. Experience in Tanzania in 1988/90 with a product of the same origin, had been disappointing, with balls failing after a few hours of running under test conditions.

However low cost and ready availability in Nigeria tempted the author and his Nigerian colleagues, to conduct further tests on Chinese and Indian 1/4" balls. Samples brought back to Warwick were sent to GKN Materials Ltd testing laboratory for analysis and the results are shown in the test certificate (Appendix 2). The important constituents to note are C (carbon), Cr (chrome) and Mn (manganese). Carbon content in the samples is much too low at 0.29% and should be in the range 0.9% to 1.0%, Chrome content is similarly extremely low at 0.02% - commercial bearing steels would normally be from 1.0% to 1.5%.

Cup-and-cone races case-hardened in charcoal, were run at 80 rpm and 200 kgf load at Warwick with 26 × 1/4" Chinese balls. Only the equivalent of 14 km was covered before the test rig began to omit nasty grinding noises. On inspection it was found that several of the balls had disintegrated totally, although the cups and cones were intact.

Back to the drawing board!

Thus the Chinese/Indian balls were not hard or strong enough for the purpose. An attempt was made therefore to treat some balls to bring them up to satisfactory hardness. Balls were heated to 900°C and quenched, some in cold water and others in oil. Some Chinese balls that had been carburised by Nigerian staff, but not hardened, were sent to Warwick and were treated in the same way as the 'off the shelf' samples above. Hardness test results for these balls are shown in Table 1. As can be seen there is much variation, even within the groups of five balls, suggesting that quality control during production is poor.

This variation and poor hardness rendered unviable the use of Chinese and Indian balls in the present cup-and-cone bearing systems for animal-drawn carts. The suggestion that better quality 1/4" balls be used was rejected because of the strong temptation for unknowledgable or unscrupulous constructors to revert to the use of inferior balls. Guarding against such a possibility would not be possible and the decision was therefore taken to standardise on 8 mm diameter balls, this size being available new or second hand in Nigeria only from quality manufacturers.

4.4.4 Conclusions.

The development of locally pressed cup-and-cone bearings can be seen as a breakthrough for indigenous manufacture of animal drawn vehicles and other equipment in Sub-Saharan Africa. Technicians in Nigeria were most pleased to realise that local manufacture of quality rolling-element bearings was within their capability. Important or "key" technologies have also been introduced including simple press tooling and heat treatment processes. The fundamental understanding of such key processes is a very important step forward for small manufacturing industries in Africa.

4.5 Concluding Remarks on Design Development and Testing.

Key components of the Borno cart are the results of previous developments and testing. Wheels, bearings and to some extent hubs, have a development history dating back to 1988 in Tanzania. The basic technology applied in the making of pressed steel wheels is not new and was used in the early days of wheel development in Europe to carry pneumatic tyres. Cup-and-cone ball bearings represent some of the earliest developments of rolling-element bearings - though fairly simple in construction they are capable of withstanding heavy loads and giving good bearing life because of their tolerance of dirt and poor manufacturing standards. Such qualities have been proven by their universal use in bicycles for example. Improved bearing technologies needed better and more versatile hubs to accommodate them. Cast-iron hubs are cheap to produce if the technology is available, but aluminium castings make a very acceptable alternative.

Table 1: Hardness Tests: Nigeria purchased Chinese 1/4" balls - July 1993.

Ball #	Treatment	Rockwell C #	Vickers #
1	As purchased	46	460
2		35	350
3		40	400
4		48	490
5		47	480
6	Heated to 800/900°C	43	430
7	Quenched in oil.	33	330
8		26	275
9		33	330
10		45	450
11	Heated to 800/900°C	55	600
12	Quenched in water.	45	450
13		60	700
14		55	600
15		52	550
16	Carburized in Nigeria	41	410
17	for 8 hours in charcoal.	57	650
18	Re-heated (at Warwick) to	55	600
19	800/900°C and Quenched in	52	550
20	oil.	54	580
21	Carburized in Nigeria	58	660
22	for 8 hours in charcoal.	54	580
23	Re-heated (at Warwick) to	52	550
24	800/900°C and Quenched in	63	780
25	water.	60	700

1 to 5 Average = 436,	Standard Deviation = 60
6 to 10 Average = 363,	Standard Deviation = 74
11 to 15 Average = 580,	Standard Deviation = 90
16 to 20 Average = 558,	Standard Deviation = 90
21 to 25 Average = 654,	Standard Deviation = 93

PART B: MANUFACTURING THE BORNO ANIMAL CART.

SECTION 5: MANUFACTURE OF PRESSED WHEELS.

5.1 Introduction.

Second-hand wheels from motor vehicles are the usual choice of the animal cart builder in Africa. Considering the number of motor vehicles imported or assembled in Africa since the second world war - particularly since independence in the 60's - it is not surprising that many discarded wheels are available. Although the use of such wheels is a reasonable solution for the needs of cart builders constructing small numbers of carts, they should not be thought of as the only alternatives to wooden or heavy steel rimmed wheels. In fact they have several disadvantages for cart builders including a lack of controlled quality from one wheel to the next, a lack of consistency and standardisation in size of mounting and centre holes, and a shortage of supply leading to rapid rises in price as soon as any number are purchased in one area.

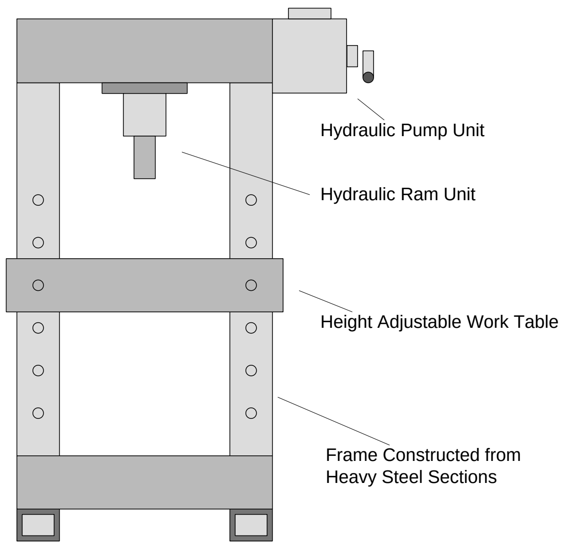


Figure 19: hydraulic press used in pressed wheel manufacture.

Manufacture of the pressed steel wheels described in this document utilizes a hydraulic press of minimum 40 tonf capacity. In fact such presses are not so rare in developing countries, as may be thought. Rail systems have never really developed in Africa and bulk goods and people are transported along roads. It is inevitable, therefore that a network of auto-repair workshops and garages has sprung up along roads carrying motorised traffic of any density. Buses and lorries in Africa break down regularly, ensuring plenty of work for the workshops. Breakdowns often involve damage to heavy suspension and transmission parts, and the repair of these cannot be carried out solely by hand as the forces required are considerable and must be carefully controlled. Thus it has become essential for many garage workshops to have an hydraulic press. These presses are not to be confused with the power presses used in manufacturing industry - the garage press (Figure 19) is a simple affair which uses a hydraulic unit similar to a hydraulic jack, set in a very robust frame, to exert large forces slowly in a controlled manner.

Before proceeding with pressed wheel development in Nigeria, a brief survey was made of Maiduguri town to determine the availability of a suitable press. It took only a couple of hours to locate six hydraulic presses of 60 tonf capacity. Three of these presses were located in Government controlled workshops, the other three were in the private sector. All were available for use by one means or another - the government presses free, the private sector presses by paying a small fee. All but one of the presses was manually pumped - probably an advantage because manual operation is likely to be safer than electric operation in unskilled hands.

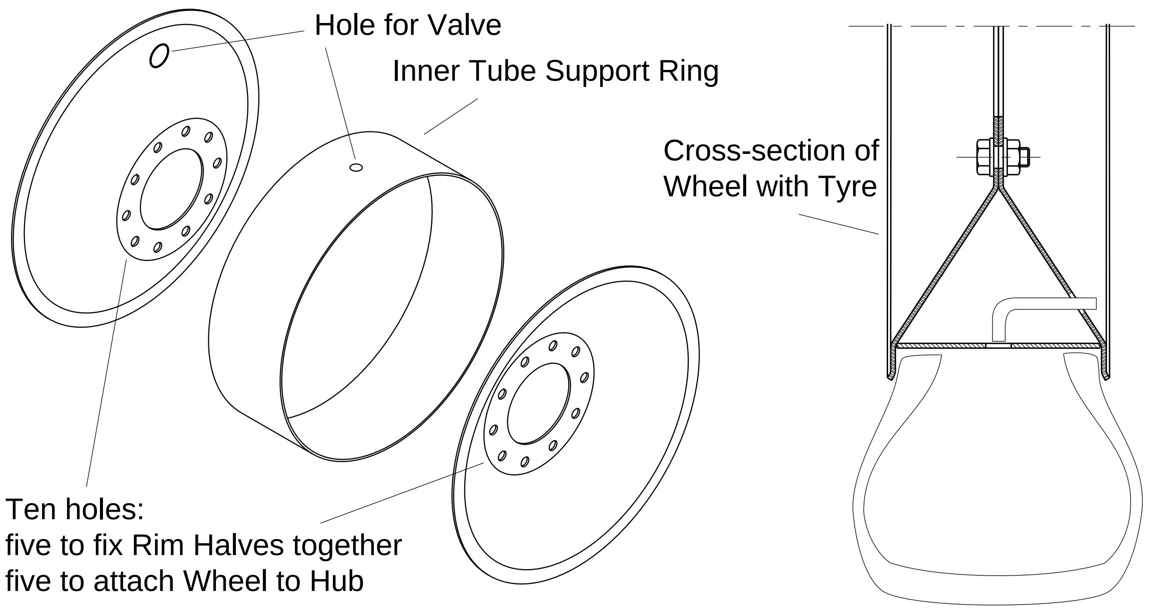


Figure 20: pressed steel wheels.

5.2 Overview of Wheel Manufacture.

The development of pressed steel wheels was centred around the use of 3mm (or 1/8") thick black mild steel sheet. The aim was to produce a wheel (Figure 20) that would be simple to make using a minimum of differing material sizes and manufacturing processes.

The manufacturing process involves pressing two round steel blanks into dish shapes, which are then bolted back to back. A circular inner rim, also made from 3mm steel sheet, is added to support the inner tube. The entire fabrication process consists of about thirty steps and when complete results in a wheel weighing about 12 kg. This may be compared with the earlier Worth and IT Transport designs which weigh about 25 kg, require five different material sections and take up to fifty construction steps to complete.

The hydraulic press is the central piece of equipment, extensively used in the manufacture of this particular design of wheel. The press is used for all the following processes :

- First and secondary wheel-rim pressings.
- Punching a 16mm diameter pilot centre hole in each wheel blank.
- Punching a 113mm diameter wheel centre hole.
- Punching the 16mm diameter valve hole in the wheel rim
- Punching a 30mm diameter valve access hole in one wheel blank.
- Punching the 14mm diameter wheel hub bolt holes and the 11mm diameter wheel rim holes, a total of sixteen per wheel.
- Pressing the bearings into the hubs.

Note, the 113 mm diameter blanks produced when punching the centre of the wheel discs can be reduced using a press tool and then utilized as bearing cover plates at each end of the hubs.

5.3 14" Pressed Wheels: Detailed Manufacturing Procedures.

Two sizes of pressed steel wheel have been developed for use with 14" and 16"tyres, two very popular sizes in Africa. The description below concentrates on the 14" variant and refers to the drawings in Appendix 3. The methodology for making 16" wheels is identical.

Wheel disc blanks are cut from standard 3 mm or 1/8" thick steel sheet and a 4 ft × 8 ft sheet provides enough circular blanks and flat strips for five complete wheels with their inner rims. The remainder of the plate may be used for making cup and cone bearing housings and hub end bearing cover plates, resulting in very economical use of the material.

The manufacturing procedures for the 14" wheels are as follows:

1. Blanks for 14" wheels are marked out on the sheet and cut to the diameter shown in the drawings. Cutting can use oxy-acetylene equipment or a band saw. After cutting, a 16 mm pilot hole is punched in the centre of each blank disc and one disc also requires a 30 mm diameter inner tube valve access hole in the position shown. The blanks must then be annealed, or softened to make them suitable for pressing. This is most readily done by standing the blanks evenly spaced and building a fire around them of scrap wood. The plates need to be heated for about 30 minutes, but they need not reach red heat. The plates should be moved about within the fire for even treatment and left to cool when the fire goes out.

2. Annealing may distort some of the blanks, but this is not a problem provided that each blank will fit into the press tool and the top clamp ring can be bolted in place. Any badly distorted blanks may be straightened by placing them on a flat floor and jumping on them, as in its annealed state the plate is soft and is easily bent back into shape. Eight 16 mm diameter bolts hold the top clamp ring down - it in turn holds the blank in place and forms the outer flat section of the wheel, preventing distortion around the rim during pressing. The blank is centralised in the press tools using the

16mm diameter centring post in the bottom tool.

3. The top tool is now put in place, again using the 16 mm centre post for positioning and the press operated which will force the top tool down into the bottom tool and form the blank into the desired shape. Depth of pressing will be controlled by the plywood spacer in the bottom of the bottom tool: when this is reached ram pressure will rise rapidly. From experience 35 tonf is enough with 40 tonf as a maximum. On inspection the pressed blank should be evenly formed with the central hub mounting area, 200 mm diameter, well defined and flat. After pressing, the rims of two wheel discs placed back to back should be separated by 94 mm internally. In order to maintain this dimension it is important that the pressing force used should not vary from one pressing operation to another.

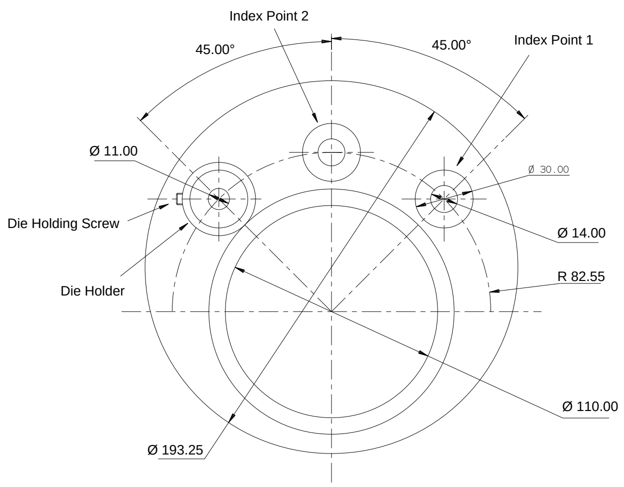


Figure 21: layout of jig used for punching holes in pressed steel wheels.

4. Once all the blanks have been pressed, secondary pressing of the outer rim (which takes the sharp disc edge away from the wall of the tyre) can take place. For this the clamping ring is left in place, but the top tool is turned over. Each disc is then placed, dish side down, on top of the clamping ring, the inverted top tool placed in position, and the press operated to around 30 tonf. The outer rim should be turned up about 30 degrees as a result.

5. A small clearance over the aluminium hub is provided by the 113 mm diameter central hole and this allows for slight deviations in casting size. It is important that the size of the hole is exactly the same in all the wheel discs, as the placement of the four rim bolting holes and four wheel-to-hub fixing holes is determined from the centre hole. This central hole can be produced most quickly and accurately by punching with a simple press tool in the hydraulic press: the force required will be about 38 tonf. In Nigeria a suitable set of tools were made from scrap gears from a lorry gearbox. Blanking forces can be calculated by multiplying metal thickness by length of perimeter of blank, to get the area to be cut, and then multiplying this area by the shear strength of the material.

6. Eight bolt holes are punched in the wheel disc, all on a 165 mm PCD (pitch circle diameter; ie the centres of the holes all lie on a circle of 165 mm diameter). Four of these holes are 14 mm diameter (for fixing the wheel to the hub) and four are 11mm diameter (for bolting the two wheel halves together). A complete wheel has 16 bolt holes (eight in each half) which must not only align when the wheel halves are put together, but the holes must also align with the four studs of the hub. One method of achieving sufficient accuracy is by using a drilling jig, a circular plate which has the holes to be drilled accurately set in it. To avoid rapid wear through repeated use, it is best to fit hardened steel bushes in the jig which can be replaced when worn. A second method is to punch the holes using a press tool that, ideally, produces all eight holes in the correct location, at one time. An alternative punching method, and the one used by the project in Nigeria, produces one hole at a time in conjunction with an indexing system (Figure 21).

7) The inner rim of a complete wheel is a flat strip of 3 mm steel sheet 1094 mm long and 94 mm wide. Before bending the strip into a cylinder the 16 mm diameter valve hole has to be punched in the position shown, and the ends of the strip pre-bent in a vice. It is not necessary to anneal the material as the bend is a gentle one. If a sheet roller is not available the strip can be bent around a wooden former which should be 350 mm diameter. After bending, the ends of the strip can be welded together on the inside of the ring only.

5.4 Wheel Assembly.

On assembly, with the bolts slack, rotate the cylindrical inner rim to be sure that the valve hole in the wheel disc half and the valve hole in the inner rim are aligned. The inner rim should sit comfortably in the pressed area. With the second disc in place and bolts finger tight, the two halves should come nearly together with a gap of about 2 mm. If the gap is greater than this the inner rim will have to be reduced in width and all other inner rims made to the new dimension. With the two halves bolted tightly together the inner rim can be tack welded in two places to the rim disc that DOES NOT have the valve hole in it. After tack welding disassemble the wheel and make four heavy tack welds between the inner rim and disc on the inside of the inner rim. The inner rim is welded to the disc only to prevent movement when the wheel is being taken to pieces or assembled, the strength of the wheel when in use is not dependent upon the welding.

SECTION 6: CAST ALUMINIUM HUB MANUFACTURE.

6.1 Introduction.

In several towns in the north of Nigeria, including Maiduguri, Jos, Kano and Zaria, there is a small but thriving cottage industry of aluminium foundries making cauldron type cooking pots and other small aluminium goods. The Linkage Programme worked with one of these small foundries in Maiduguri. Scrap aluminium is readily available in quantity (by the ton if required) from specialist merchants operating from Kano, Jos, Onitsha, and Lagos, among others. The caster also operates an exchange scheme with his customers, ie new pots for old, as part of his source of scrap. Aluminium, as of May 1994, cost Naira 40/kg (= 0.60 p/kg).

The company (Aluminium Pot Makers and Casting Workshop, Gwange Sabon Line, Post Box 4181, Maiduguri) employs from four to six workers and produces up to 60 cooking pots of 350 mm diameter maximum per day. The owner, Malam Umar Usman is about 35 years old and had been in business for nine years at the time of our involvement. His workshop comprises a floor of about 10 m × 15 m, partially covered with a corrugated iron roof mounted on wooden poles. The floor of the covered area is a thick layer of local sand which is used for the moulds. Aluminium is melted in steel pots (cut from old LPG cylinders) in a ground level furnace, using an blown-air charcoal fire. Air is supplied from two or three metres away, by an underground pipe fed from a centrifugal blower, cast locally in aluminium. It is driven by a rubber belt (made from old inner-tube) from a bicycle wheel turned by a small boy.

Observation of Mal Umar in operation shows that he and his workers are highly skilled from long experience, and have an in-depth knowledge of the process and of solutions to the technical problems involved in aluminium casting (for example old dry cell batteries are added to the melt to assist in de-gassing), and this skill allows the consistent manufacture of high quality products.

6.2 Casting the Mk 4 Aluminium Hub.

The MK 4 cast aluminium hub (Figure 22) represents the final hub development in the present programme. The hub has four studs on a 165 mm PCD and weighs 2.8 kg. Only limited machining of the preformed bearing housings and the drilling and tapping of the four stud holes is required to complete the hub after casting, because a steel mould giving accurate outside dimensions, is used. About four castings per hour can be produced by this method and machining time for the two bearing housings and the four wheel stud holes is twenty to thirty minutes.

Inserts in the steel mould preform bearing housings in both ends of the casting, and a finished-to-length steel core pipe is located in the mould before pouring to provide the central axle space. The mould, totally enclosed, is filled through a simple filler and riser system consisting of two short lengths of 40 mm diameter pipe which stand over holes in the top of the mould (nb they are not fixed to it). The mould and these filler and riser pipes are completely filled during pouring, and as freezing progresses the aluminium in the pipes feeds down into the casting and prevents shrinkage cavities forming. It was observed that the aluminium had to be very hot before pouring into the steel mould in order to flow freely and not freeze too quickly. After about two minutes the filler and riser can be broken off the mould before the aluminium has had time to solidify completely. Extracting the casting from the mould can begin after five to seven minutes (and certainly no later than ten minutes) after pouring, or the aluminium will contract onto the mould bearing inserts, even though they have generous release tapers. Should this occur, allow the mould and casting to cool completely and then suspend the whole issue over a hot charcoal fire for about 15 minutes. The aluminium will expand more rapidly than the steel components and release will be possible.

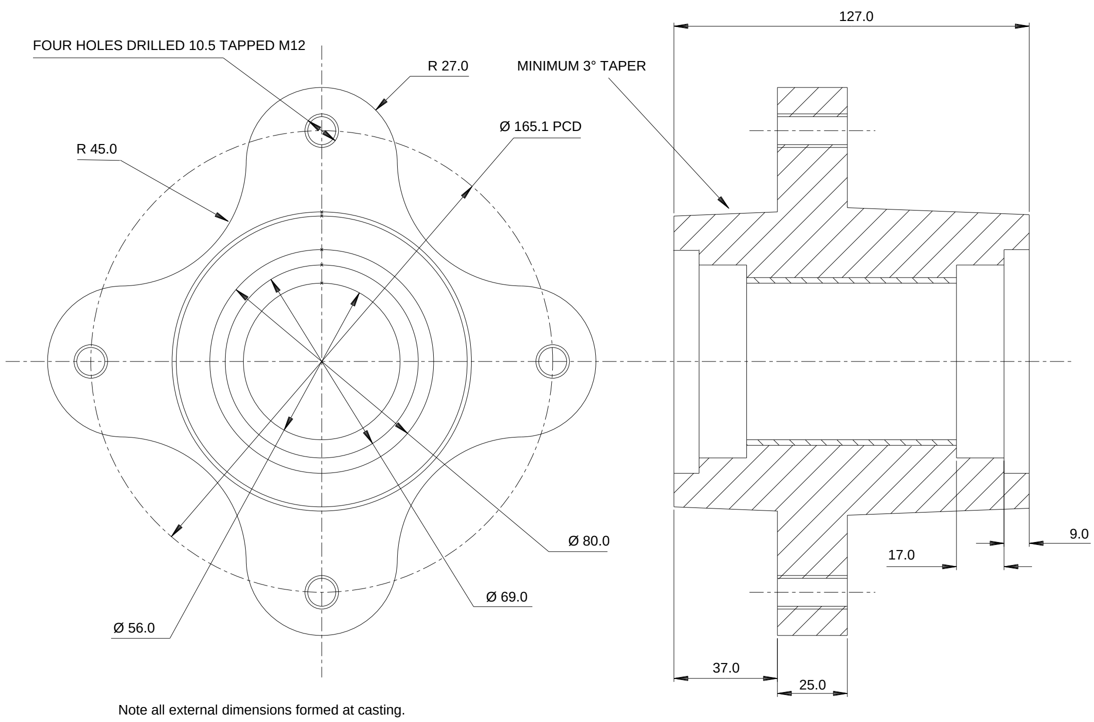


Figure 22: Mk 4 aluminium hub - gravity die cast in steel mould.

It may be argued that foundry facilities or raw materials do not exist in all African countries. This may or may not be the case, but where the materials are available the technology itself is not difficult and could be introduced.

6.3 Further developments.

Figure 23 shows an alternative sand-cast hub, which may be used with commercial tapered-roller bearings which are available at reasonable price in Nigeria. It exploits the bearings' high load capacity and allows machining of both bearing housings in one cut. A snap-ring, made from mild steel wire or similar and let into a groove in the bore, locates the bearing pair in the hub. The wheel studs are fitted into slots in the struts, saving a drilling and tapping operation.

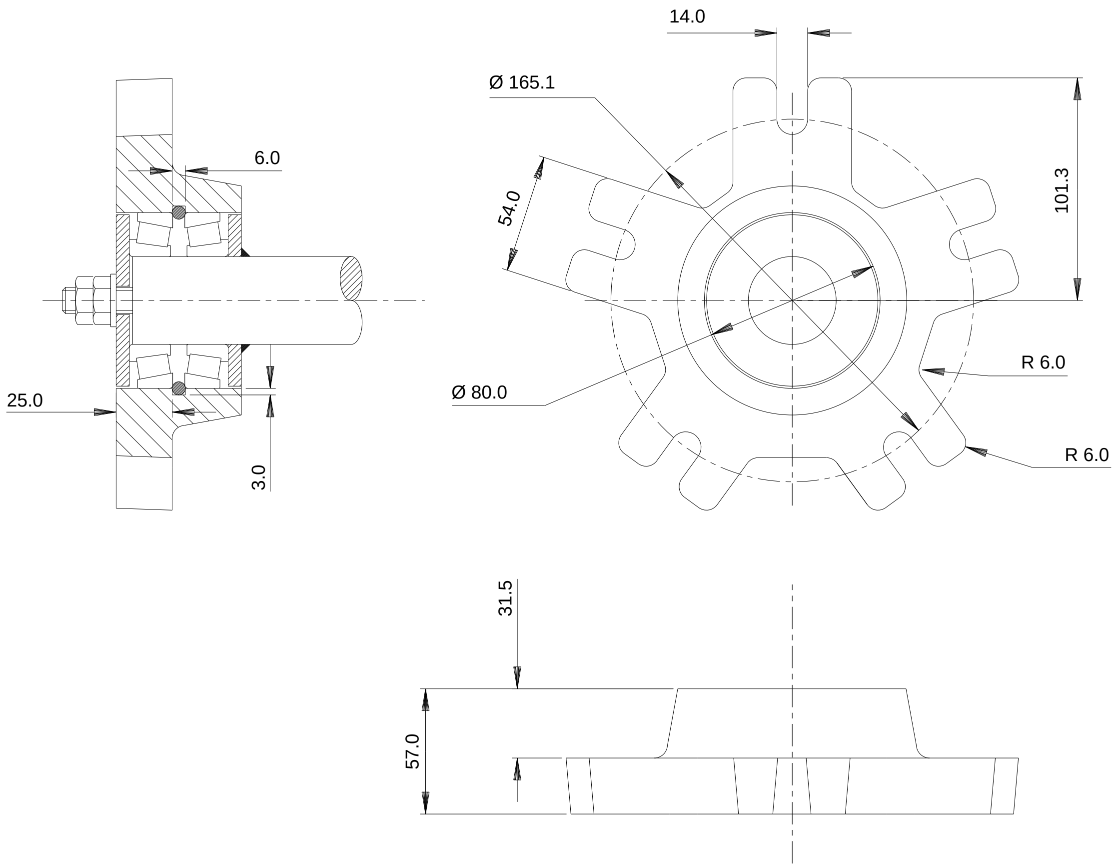


Figure 23: sand-cast aluminium hub - for 6208 tapered roller bearings.

SECTION 7: BEARING MANUFACTURE.

7.1 Introduction.

The current Warwick axle and hub design being constructed at RAMAT Polytechnic utilizes three bearing alternatives, commercial taper roller, fabricated roller (Figure 7) and locally manufactured cup-and-cone ball bearings (Figure 24), the type specifically dealt with here. Cup-and-cone ball bearings, as used in bicycles, are one of the earliest examples of a rolling element bearing. It is a simple system that is adjustable and will carry both radial and axial loads.

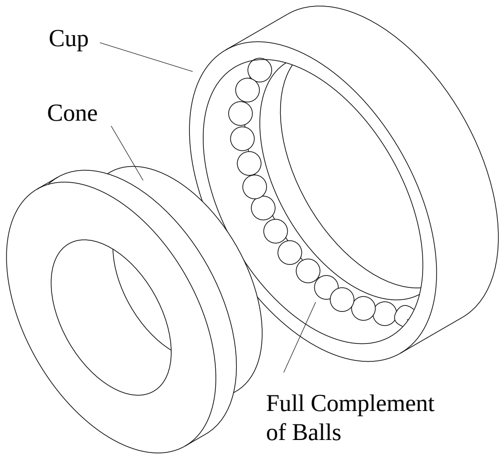


Figure 24: cup and cone ball bearings.

Cup-and-cone bearings are made up of three parts: an outer race (the cup), an inner race (the cone), and the rolling elements (the balls). Commercial bearing balls have a very high finish and are heat treated to be hard enough (55 Rockwell C) to withstand the loads imparted to them when in use, this in turn requires the cup and cone races to be hardened.

Design criteria for the use of these bearings in the axle systems of animal drawn carts were as follows:

1) able to withstand maximum loads of approximately 250 kgf per bearing,

2) easy to fit, adjust and replace,

3) simple and cheap to manufacture.

Earlier versions of cup and cone bearings developed in Tanzania in 1988 were machined from solid bar and did not comply with the third, and perhaps, most important criteria above. Solid bar sections larger than 20 mm dia are hard to come by, and expensive, in most countries in Africa. Also machining these components requires the constant availability of expensive machine tools, a lathe, and the appropriate skilled labour to operate the machine. In order to alleviate this problem development proceeded with a programme for pressing the cups and cones from 3mm thick mild steel plate.

7.2 Production.

Bearing components must be consistent dimensionally in order to operate reasonably well and be interchangeable when worn. Such regularity should not be dependent on constant machine tool availability and a skilled labour input over a long production run of components. Even if facilities and personnel were available, costs would be high, as production rates would be low. A better approach to the problem is to develop a set of specialised tools, the function of which is to replicate bearing components speedily and faithfully, using unskilled labour.

Simple press tools for producing cups and cones can be made from mild steel to save costs and may be case hardened using the same process (described below) as for for the cups and cones. Figure 16 gives the dimensions of the cups and cones to be produced with the press tools. The top tool is made to the internal dimension of the component whilst the bottom tool is made to the outside dimensions. Blanks for the cups and cones are made from the same material as the pressed steel wheels ie 3 mm steel sheet. These are annealed as described for the wheels. Each component blank is located centrally on the bottom tool whilst the top tool is forced down and into the bottom tool - pressing the blank into the tools and taking up their shape. The process takes very little time and requires minimum skill - although skilled input is required in the initial making of the press tools and to some extent in setting them up for use. Tools maintenance is minimal provided that checks are made on the components being produced from time to time, and wear on the tools is monitored.

Bearings easiest assembled with heavy grease to hold the 20 × 8 mm diameter bearing balls in place. Adjust the bearing to run freely once the wheel is fitted.

Bearing components produced from pressing are of mild steel and will require heat treatment using a process called case hardening. The simpler aspects of heat treatment and the case hardening process are explained below.

7.3 Heat Treatment of Steel - General.

Before proceeding it should made clear that steel with less than 0.5% carbon cannot be hardened by the normal process of heating to 850/900°C and quenching in cold water or oil. Such low carbon steels are generally known as "mild steels". If in doubt take a very small sample, heat it to bright red and quench it in cold water. If the treated sample can be cut easily with a file then it is a mild steel, if however the sample is very hard and cannot be filed or cut, then it is almost certainly a high carbon steel.

To render steel soft or malleable, a reverse process of the above is applied - the steel is heated to 700/800°C, a dull red heat, and left to cool slowly - a process known as annealing. The blanks for the pressed cups and cones for the bearings and for pressed steel wheels are treated in this way.

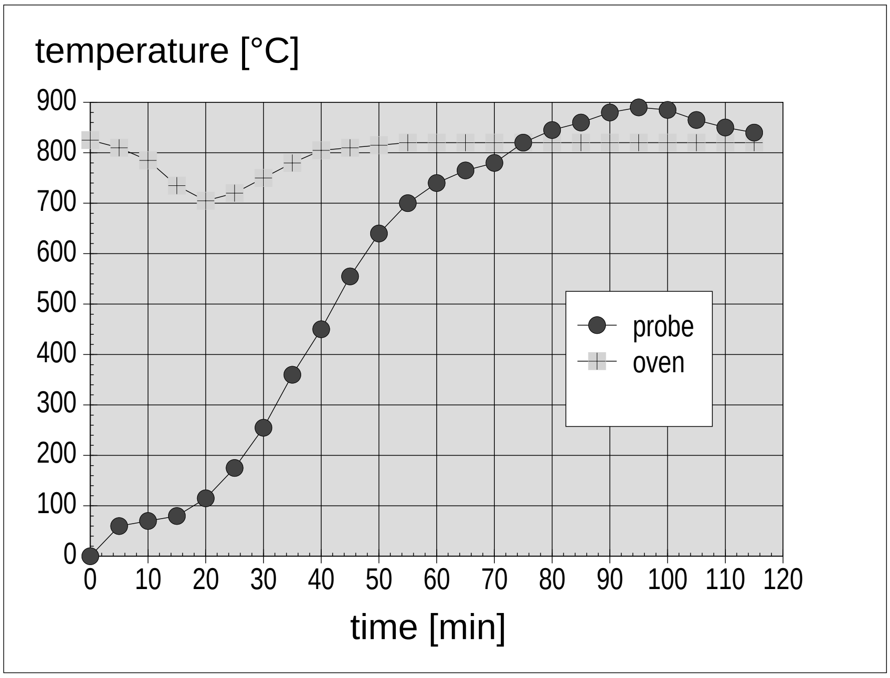


Figure 25: temperatures during carburisation of bearing components.

7.4 Case Hardening - The Process.

This process is applied to articles of mild steel where a hard skin or surface is required (gears in an automotive gearbox for example) as opposed to hardening the components right through. The best results are obtained with steels containing from 0.1% to 0.2% carbon. Articles to be case-hardened are embedded in some form of carbon-rich material - in developing countries the most readily available material is probably wood charcoal, and this should be ground into a fine powder. The process is considerably improved if 1% by weight of sodium carbonate (**nb not 'bicarbonate of soda'**) is mixed with the charcoal as an activator.

Items to be hardened are packed, with the carbon rich mixture, in a steel or cast iron container not less than 4 mm thick. It is important that the container is as airtight as possible because air getting in will oxidise and waste the carbon (clay can be used to seal the lid). Also the container must be big enough to allow a space of about 25 mm between each component and the sides and top and bottom of the box. Carbon diffuses into the components and converts the outer layers into a steel that will harden, having a surface carbon content of about 0.9%. The depth of case depends upon the size of the article being hardened, the carbon rich material used, the length of time the process is continued and the temperature held during the process.

The box containing the articles to be carbonized must be heated to 850/950°C and the temperature maintained for a period of three hours. This can be achieved with the use of an electric furnace or a blacksmith's forge. Using a forge, the steel container must be kept bright red, covered with hot charcoal at all times and it should be rotated in the fire from time to time to ensure an even distribution of heat. Treatment is normally considered to start when the box reaches working temperature (visually a bright red) and at this temperature the depth of case can be expected to increase at about 0.2 mm/hr.

After the allotted time the box should be removed from the furnace or fire and allowed to cool naturally. To harden the components they must be removed from the box and re-heated to 700/800°C (visually a dull red) and quenched in oil or water. Bearing components must be cleaned with a wire brush or emery paper before assembly into hubs. The charcoal mixture is re-usable but must have the volume made up with the addition of new charcoal powder/ sodium carbonate mix. Figure 25 shows the temperature changes in a carburization box (measuring 230 × 100 × 100 mm and constructed from 6 mm mild steel plate) during the heat treatment of cup and cone bearing components.

Bearing cups and cones, heat treated for three hours, will be carburized to a depth of a little over 0.5 mm. Such a depth of case is required to withstand the pressure from the hardened steel balls when the bearing is under load. Charcoal case-hardening prolongs the life of the bearing components considerably and in itself is a *'key process'*, potentially of great value to rural and urban artisans and to small industries in developing countries.

SECTION 8: CART BODY MANUFACTURE.

As discussed above, two cart bodies have been developed for construction in the current project. The body developed by the DTU for use with the DTU Borno Ox-Cart is made from sawn timber with simple fixings which are very easy and cheap to construct. Construction details of the oxcart body are detailed in Technical Release 24 attached as Appendix 9.

Also available is a design for a lightweight donkey cart. Construction of this cart utilises 50 mm × 50 mm light steel box section for the frame, with floor and sides made from wood. Construction details for this body are available in DTU Technical Release 23 included as Appendix 10. This design can easily be modified for use as an ox cart with small increases in material sizes and overall dimensions.

SECTION 9: TOOLS FOR DTU BORNO CART PARTS.

9.1 Introduction.

Earlier in this paper, details of the manufacture of the various components for the Borno ox-cart were described. Production of many of these parts depends upon the use of tools, dies, jigs and fixtures - generally known as special tools. These are used in order to ensure dimensional regularity, interchangeability, speed of production and general high quality of finish. This section of this document will describe how to construct these various special tools and as some of the tools are similar in their function, a general description of the tool will be given before a description of the variants needed for producing different parts. For example, making a hole-punching tool is explained in a way that can be applied to different punches of various sizes. The special tools dealt with are as follows:

- forming or bending tools for wheel-discs and cup-and-cone bearing components,
- punching or blanking tools for holes in the wheel discs, cup-and-cone bearing components and hub end-thrust washers,
- drilling jigs for holes in cast aluminum hubs,
- steel casting moulds for aluminium hubs.

It is recommended that the making of these tools should only be considered if it is expected that a significant number (say 100) carts are to be manufactured each year. For more limited production, traditional methods, such as drilling, cutting and filing can be employed. Of course press tools would still have to be made for the wheels and the cup-and-cone bearing components.

Regarding limited scale manufacture, the DTU is committed to developing other designs which can be made without special tooling and has made significant progress (as exemplified by the axle shown in Figure 7). Further development of this axle is necessary before it can be released for general use, but it is able to give up to 15 000 km in good conditions, and work is in hand to reduce still further the level of tooling required. New designs may even be able to avoid the need for drilling.

9.2 Forming or Bending Tools - General.

It may be helpful to the reader to understand the pressing process and the importance of controlling pressing forces during the manufacture of the sorts of component discussed in the sections above. The following is an attempt to throw a little light on the subject for those not already well informed.

A hefty blow from a large hammer delivered onto a thin piece of steel will bend it or change its shape, usually not for the better, unless the person wielding the hammer happens to be very skilled. It will be even more difficult to bend a second piece of the metal to the same shape as the first. The problem is that the force of the blow from the hammer (and its position and angle) is not well controlled, because it is delivered by hand. Most of the problems of force control are overcome by using a press which delivers the force in a straight line, usually vertically. The problem still remains however, that the material being pressed will have a tendency to form or flow along the least line of resistance, which may or may not be that intended. A simple form like a straight bend across a flat strip is easily controllable, whereas multiple bends close together may cause a problem because the stresses in the material pull in different directions. Such forces in the material can be controlled using press tools, which are designed to hold the material firmly in place and, under the force of the press, make all the bends at once. Figure 26 shows a simple bending tool which might be used to produce a V-bracket.

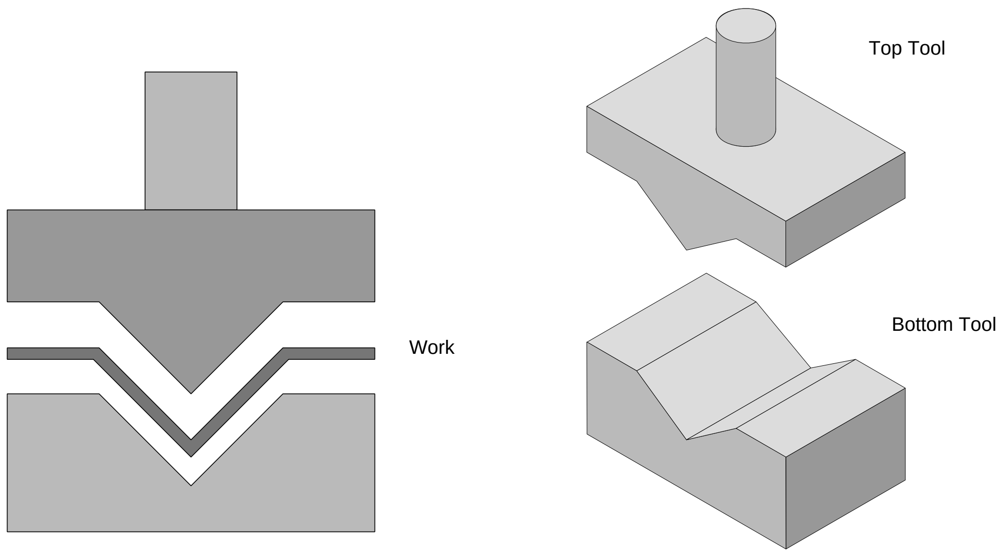


Figure 26: simple press tooling example - here for a V bracket.

Before pressing a blank must be made, but material stretch during pressing will cause dimensional changes which must be allowed for in the size of the original blank. It is possible to calculate the amount of stretch that will take place, but such calculations are not easy and will depend on the original state of the material - for instance if it has been annealed or not. The simplest method for determining the correct blank size is to run a series of pressing tests. By comparing the pressed components with the required design for a series of blanks, a reasonable estimate can be made of the necessary blank size.

The operation of the bending tools for wheels is to change the shape of the material being formed into that required. For this purpose the tools have to be sufficiently robust to withstand the pressing forces whilst the operation is taking place. Details of the simple tools for pressed wheels are shown in Appendix 3. The materials used are the lightest that may be expected to tolerate the pressing forces imposed when pressing wheels, and still give reasonable tool life. If available, heavier materials can be used with an obvious increase in cost and life. The important function is to form the rim and press the centre 200mm diameter portion to the correct depth in relation to the rim. The exact shape of the coned or dished portion of each wheel half is not important, and so its exact shape is not controlled by the press tools. In practice it may take up a slightly rippled shape, but this will not detract from its strength or function.

When making the wheel discs, simply to press the centre portion down without controlling the rim would result in an irregular dish shape with a corrugated and ill defined rim. The very important function of the pressure ring is to hold the wheel disc in place and control the flow of the material. The wheel rim is held flat and in a horizontal plane by the pressure ring whilst the centre portion is pressed down to form the dish. The pressing of cups and cones for bearings is a very similar process to that of the wheels, and basic pressing methodology applies. Details of the press tools required are shown in Appendix 3.

9.3 Punching or Blanking Tools - General.

Correctly set blanking or punching tools produce an accurate hole or blank, (the 'blank' is the piece that falls out of the hole). Other advantages compared with drilling are that the finish is good and that the production time is short. Tools, once made are long lasting, and easy to service. The most important parts of the tool are the punch and die, both of which must be made of a high carbon steel and hardened (heated to 850/900°C and quenched in oil). Figure 27 shows the general arrangement of a simple punching tool. Hole or blank size is determined by the size of the die and not the size of the punch. A small clearance is required between the die and punch, as the material being punched is only cut for about one third of its thickness, the other two thirds being sheared or broken. This clearance is determined by the thickness of the material being punched - a good guide is to subtract 0.10 mm from the dimension of the punch, for every 1mm thickness of material being punched. It is of essential that the relative position of the punch and die be accurately controlled as otherwise damage will occur to both tools.

9.3.1 Pressing Force for Blanking.

The force required to punch holes, or cut out blanks of any size or shape, depends upon the total cross sectional area of metal to be cut, and the shearing strength per mm^2 of the material. It is usually expressed in tonnes force (tonnef) and is calculated by multiplying the total length cut by the thickness to obtain the sectional area in mm^2 (square millimetres), and multiplying the result by the shear strength of the material in N/mm^2. A typical example with mild steel, which has a shear strength of 430 N/mm^2, might be a blank 50 mm × 100 mm in 3 mm plate. This would need a force F of:

length of cut × plate thickness × steel shear strength.

$= 300 \text{ mm} \times 3 \text{ mm} \times 430 \text{ N/mm}^2$

= 387 000 N = 39.5 tonnef, (as 1 tonne force = 10 000 N)

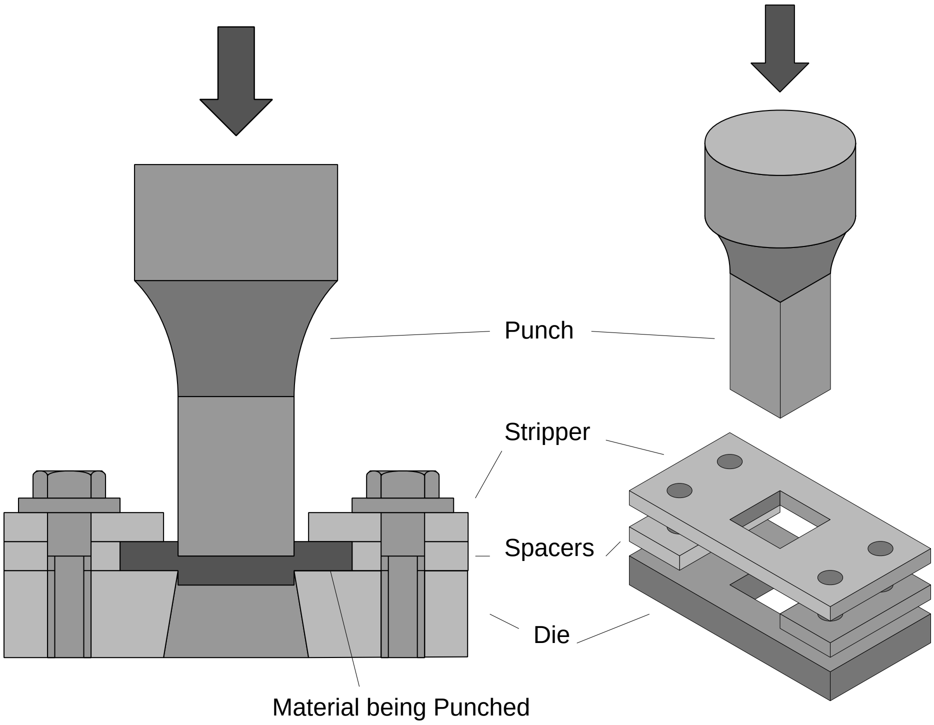


Figure 27: design of a blanking tool to produce a square hole.

A similar example using imperial measurement would be:

rectangular blank 2"× 4" = 12 sq ins (length of cut) × 1/8" (thickness) = 1½" sq ins (area of cut) × 30 tonf/sq in (shear strength of mild steel) = 37.5 tonf.

The capacity of many presses is expressed in tons force (tonf), and materials are often sized in imperial measurement and so it is useful to have the Imperial formula handy. (In any case the reader will notice that there is only a small difference between tonnef and tonf and that 4" is nearly the same as 100 mm.)

9.4 Punching or Blanking Tools for Ox-Cart Manufacture.

In the manufacture of components for the Borno oxcart, punching tools are used for producing the following, (Ø means diameter):

wheel discs:

1. eight (four Ø 14 mm & four Ø 11 mm) wheel hub and bolt holes,
2. one Ø 32 mm valve hole,
3. one Ø 113 mm centre hub hole,

hubs:

4. bearing endplate protectors (these can be made from the discarded Ø 113 mm blanks from the centre hub hole),

bearings:

5. blanks for cups and cones.

Tools for punching the above items are constructed as illustrated in Figure 27. Details of the punching tools for the items 1 and 5 are shown under Appendices 4 and 5 of this report.

9.5 Drilling Jigs - General.

A drilling jig is used for accurate placement of a hole or a series of holes. This is particularly important in production and for the interchangeability of parts. Accurate drilling of holes is a difficult exercise at the best of times. Though drilling appears simple and straightforward, it is full of pitfalls for the unsuspecting - for example accurate marking out, accurate centre punching, adequate holding of work and correct sharpening of the drill are all difficult, and any one, or a combination of the above, will cause inaccurate placement and size of the hole being drilled if badly done.

A drilling jig is made to locate over the component being drilled, and hold the component firmly in place whilst the drill is guided through a well fitting bush. Of course the jig itself has to be made accurately if it is to do the job correctly. The time taken in making a good drilling jig will be more than compensated for in the time saved with accurately drilled components.

When designing a drilling jig some thought has to be given to the method of inserting, holding, and removing the component. The jig has to have an open plan of construction to allow the swarf (cuttings that result from the operation) to clear from the jig and not block it up. An example of a simple drilling jig is shown in Figure 28.

If press tools are employed, only one drilling jig is required in the manufacture of the Borno ox cart. This jig is employed to locate accurately the four stud holes in the lugs of the cast aluminium hubs. Details are shown in Appendix 4 of this report. Ideally drilling bushes are made from high carbon steel and hardened, or they can be made from mild steel and case hardened as described earlier.

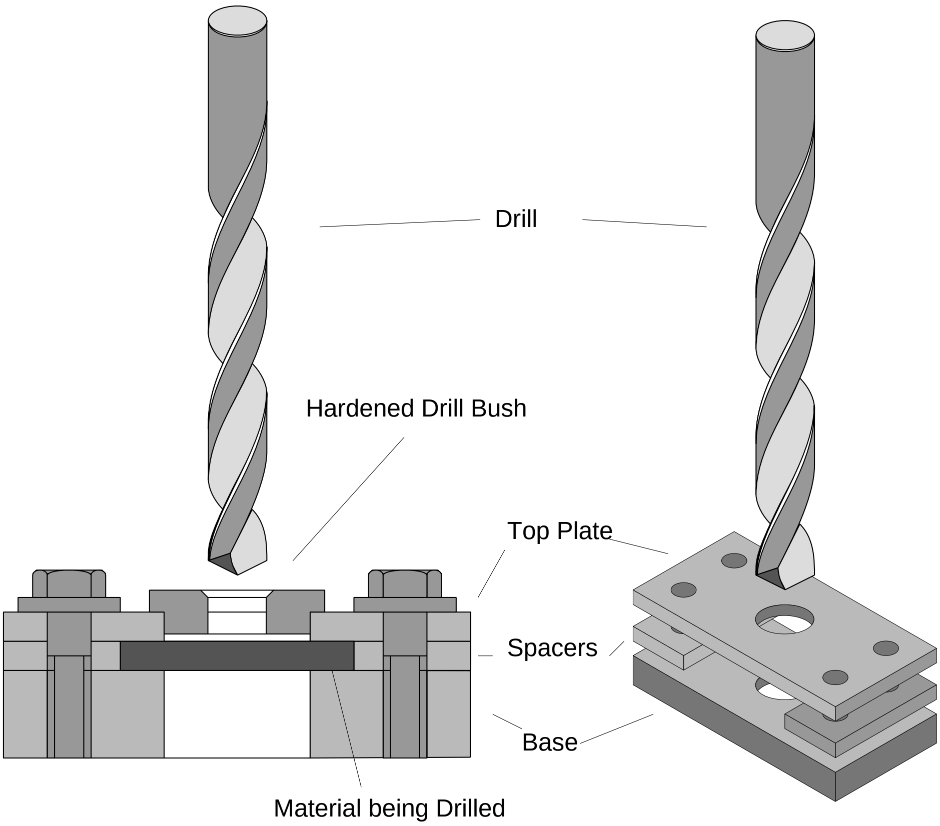


Figure 28: simple drilling jig.

9.6 Steel Casting Mould.

Details of the steel mould for casting aluminium hubs for the Borno Ox-Cart are shown in Appendix 6 of this report.

Appendix 1: Test results for aluminium cast in Nigeria.

Appendix 2: Test results for Chinese Ball Bearings.

Appendix 3: Pressed Steel Wheels: Dimensions and Tooling.

(See also Appendix 4: Wheel Hole Punching Tools.)

Appendix 4: Wheel Hole Punching Tools

Appendix 5: Cup-and-Cone Press Tools.

Appendix 6: Aluminium Hub Casting Tools.

Appendix 7: Drilling Jig for Aluminium Cast Hubs.

Appendix 8: Axle Drawing for Wheel Hubs.

Appendix 9: Wooden Ox-Cart Construction.

Appendix 10: Steel and Wood Donkey Cart Construction.

Appendix 11: Pipe and Roller donkey Cart Axles.

DTU Working Paper No.44 **December 1995**

The Potential of Cement-Stabilised Building Blocks as an Urban Building Material in Developing Countries

by Dr D E Gooding and Dr T H Thomas

Statement

This Working Paper comprises the *Overview* part of a Report (TDR Project No.D141) to the Overseas Development Administration (ODA) of the British Government which wholly funded the study on which the Paper is based. The Contents List also shows the *Appendices* to the Report which for reasons of economy have been omitted from this Working Paper. Appendices A to I contain the raw survey data acquired during visits to 7 African, 1 Asian and 1 Latin American countries. Individual appendices are obtainable on request from the DTU.

The responsibility for opinions expressed in this Paper rests with the authors and are not necessarily endorsed by ODA. Moreover reference to the names of firms, commercial products and processes does not imply their endorsement by ODA, nor does failure to mention a particular firm, commercial product or process indicate any sign of disapproval.

Abstract

This Paper examines the level of technical achievement in production and the level of social acceptance of cement-stabilised building blocks (alias ‘soil-cement’) currently displayed in 9 developing countries surveyed in early 1995. The survey established that these blocks are currently in common use and are likely to be more widely used in the future. Several technical problems or deficiencies were however identified across the whole area visited, as were new developments pertinent to the advancement of this building technology. These deficiencies and developments are analysed and used to define the research, design and training needed to significantly improve the effectiveness of cement-stabilised blocks as a low-cost walling material in urban areas of developing countries.

CONTENTS

APPENDICES

(total 65pp, individually available on request)

EAST AFRICA

A Kenya
B Tanzania

SOUTHERN AFRICA

C Botswana
D Zimbabwe
E South Africa

WESTERN AFRICA

F Ghana

OTHER

G Sri Lanka
H Mexico
I Uganda

J ECONOMIC COMPARISON OF FOUR POSSIBLE PRODUCTION METHODS

SUMMARY

There is a widespread shortage of permanent housing in urban and peri-urban areas of Africa. This shortage is increasing both because of high rural-to-urban migration rates and because of the relatively high cost of permanent urban building materials. The poorest sector of the community is most affected by this housing shortage as it is least able to afford construction materials classified as permanent under prevailing building regulations. This project has focused on building materials for this sector of the population and in particular on cement-stabilised blocks.

Review of objectives

The project leading to this Working Paper falls into ODA’s TDR theme objective U1, "Improving the quality and accessibility of low cost housing and other infrastructure provision in poor urban areas". It forms a preliminary survey to asses the current cost, quality and potential for improvement of cement block making and the viability and value of accelerating the extension of this technology in poor urban and peri-urban areas through the implementation of subsequent programmes of research and design, and of producer training.

It was intended to measure the scale of cement block use, the social acceptability or otherwise of such blocks, whether there is a need for improvement in quality (if so potential interventions were to be identified) and whether it is possible to reduce costs significantly. Using the data gathered it would then be possible to determine whether in combination these factors form sufficient justification for further research to facilitate any potential interventions identified or whether other building materials are more suited to these circumstances and consequently cement block production should not be pursued further. See also Figure 1, "Logical Framework" (overleaf).

Summary of work carried out

Major surveys were carried out in Kenya, Tanzania, Botswana, and Ghana while minor survey were conducted in Sri Lanka Uganda and Mexico. Predominantly urban and peri-urban areas were visited. Discussions were held with governmental and non-governmental organisations involved with building material production, housing construction, planning and building regulation. Poor urban and peri-urban residential locations were visited to assess the current levels of use of cement blocks. Block manufacturers were visited to determine the current quality and cost of the blocks produced and to observe the production methodology in use. Alternative competing building materials were also examined to compare their cost and quality with that for cement blocks. Data was gathered to enable economic predictions to be made to determine the potential block cost reductions which could be made if the process methodology and production equipment were improved. Table 1 summarises this economic modelling overleaf. Potential building material purchasers, private individuals and developers were also visited to determine the acceptability of the available building materials in terms of their cost, quality and social acceptance and the likely acceptability of improved blocks.

Based on observation of the production methodology in use, economic analysis and discussions with local experts, potentially appropriate interventions to improve cement blocks were identified.

Technical results

Cement blocks were found to be a major construction material in both urban and peri-urban areas and are increasingly becoming the basic walling material in these areas. The block quality obtained for a given production cost is much below that which could be achieved. Problems were observed with raw material testing, cement optimisation, mixing, batching, mould filling, compaction and curing. These problems could be reduced if producers were more informed, better skilled, equipped with better production and testing equipment and more diligent in quality control.

It was found that micro-enterprise production of soil-cement can offer cost savings over sandcrete walling. The cost advantage is small (0 to 30%) for built-up walling using current soil-cement block production systems. Soil-cement blocks are usually smaller than sandcrete blocks and consequently are more costly to lay because of the increased laying time and additional mortar required per square meter. Using local cost data for predictions it was found that further savings, in the order of 50%, are potentially possible if impact compaction of larger size soil-cement blocks (equivalent to the size of current sandcrete blocks) were instigated. However it was found that at present soil-cement is disadvantaged by the incorrect perception that it is not a "permanent" building material; it is strongly associated with traditional unstabilised soil construction in the minds of many. It was found that nomenclature was the prime reason for this association and that this may be remedied through the removal of "soil" from the material's name.

The manual equipment used for block production was found to be often poorly designed and its purchasers appeared generally unable to distinguish good design from bad. There was an absence of quality control procedures and in particular of testing equipment to monitor quality.

Implications of results

The implications of these findings for future R&D or training interventions are covered in the last section of this Executive Summary.

This survey found that cement blocks are a major building material for the poor (and the more well off) in the areas of study and that they are increasing in importance as competing materials continue to increase in cost. It is feasible to substantially improve block quality and reduce block costs both for soil-cement and sandcrete. If such improvements were successfully implemented then it is likely that blocks could become both more accessible and more desirable to the urban and peri-urban poor. Consequently it is likely that they could contribute to alleviating the current housing shortage.

Research is required to determine why the present level of process understanding displayed by manufacturers is so low and what improvements could be obtained under current market conditions if operators/owners were better informed and possessed the ancillary equipment to support better quality control.

Current manual compaction machine design is often poor. A wide range of machines are available globally but generally only one machine is available within one country. There is a need to assess the available machines, make public the assessment findings and feed back recommended machine developments to manufacturers and purchasers.

There is sufficient justification for the development, field testing and promotion of the impact moulding process as it offers considerable potential savings in cost and improvements in quality over pressure-moulding.

OVERVIEW

INTRODUCTION

The following report gives details of the result of a four month survey into the use of cement-stabilised[1] building blocks in urban and peri-urban areas of eastern, southern and western Africa, Southern Asia and Central America. The countries of focus for this report were: Kenya, Tanzania, Uganda, Zimbabwe, Botswana, South Africa and Ghana. Sri Lanka and Mexico were also visited to identify practices with potential for future use in Africa. In these countries the building materials used in poorer urban and peri-urban areas were surveyed to determine their price, quality and social acceptance. Particular attention was paid to cement-stabilised building materials and the level of improvement in quality and reduction in cost which could result if a programme to assist block producers were instigated. The data presented in this report was gathered from the field by Dr D.Gooding and Dr T.Thomas[2] between February and May 1995. All prices given are current at this time unless otherwise stated.

Report structure

This report has been arranged so that the central themes and patterns which extend beyond a single country are presented together as an "Overview" which comprises the body of the report. Country specific information is presented in a set of four appendices. In this way repetition of common factors has been minimised.

The Overview has been split into three parts introduction, survey and interventions. The introduction part gives a brief explanation of the differences and similarities between soil-cement and sandcrete blocks, provides the definitions which will be used in this report and outlines the importance of compaction and densification to cement-stabilisation. The second part presents the findings of the survey conducted to establish the current patterns of use, levels of technical production skill and levels of understanding in the 9 countries visited. Also in this part the most commonly observed problems with cement-stabilised building materials are presented. This part finishes with a consideration of the economic value resulting from the use of soil-cement compared to alternative building materials in urban areas. In the third part recent developments in the theory of cement-stabilisation of building blocks are presented and both immediate simple interventions and longer term remedies to the problems observed in part two are suggested. Factors which may affect the advancement of this technology are noted. Research, design and training needs are discussed.

THE TECHNOLOGY

1 THE CURRENT UTILISATION AND QUALITY OF CEMENT-STABILISED BUILDING BLOCKS

1.1 Definition of cement-stabilised building blocks

In this report the term "cement-stabilised building blocks" is used as a generic name to cover a wide range of building materials. A cement-stabilised building block is defined here as one formed from a loose mixture of soil and/or sand and/or aggregate, cement and water (a damp mix), which is compacted to form a dense block before the cement hydrates. After hydration the stabilised block should demonstrate higher compressive strength, dimensional stability on wetting and improved durability compared to a block produced in the same manner but without the addition of cement. This definition includes a range from hand-tamped soil blocks containing only enough cement to enhance their dry strength a little (but not to achieve any long term wet strength) to close-tolerance high-density concrete blocks, mechanically mass produced and suitable for multi-storey construction without a render. The spectrum of cement-stabilised building blocks has been split traditionally into two distinct fractions, sandcrete and soil-cement. Although the terms "soil-cement" and sandcrete/ sandcement/concrete have very different images in

---

1 covering the spectrum of materials from soil-cement to sand-cement or "sandcrete".

2 Both from the Development Technology Unit, a research centre of the University of Warwick.

the public mind in Developing Countries, there is no clear boundary between them. Good soil-cement may be stronger than poor concrete and use "soil" no different in particle size distribution from the so called "sand" used in sandcrete. Provided that the mixtures are "damp" rather than liquid[3] then there is no practical reason to discriminate between soil and sand cement, the production process being the same.

Sandcrete use is widespread and increasing: it has a good popular image. Soil-cement by contrast carries an association or stigma linking it with unstabilised soil and simple adobe construction which has much limited its popularity. However in areas where demonstration production has been undertaken (Arusha, Dar Es Salaam, Nairobi, Taita, Otse, Francistown, Kumasi to mention a few locations) the public has been impressed with soil-cement and the opinion has been repeatedly voiced that it appeared better than the prevailing low quality sandcrete blocks. It is therefore suggested that it is primarily the association of soil-cement with rural adobe building that has restrained its propagation.

It seems appropriate to acknowledge the spectrum of possible cost and quality which cement stabilisation encompasses but to counter the public perception that "sand-cement" blocks are high quality, durable building components, while "soil-cement" blocks are low quality and not as durable. In the country-specific appendices a differentiation has been made between soil-cement and sandcrete because at present, with the exception of South Africa, manufacturers either produce soil-cement or sandcement blocks and identify the materials as separate.

For the purpose of this report soil-cement is defined as a permanent durable material which is produced from a natural or modified soil containing sufficient fines to provide cohesion on densification, sufficient to allow unsupported handling of the freshly moulded block. Good soil-cement blocks may thus be stacked for curing. Quasi-static compaction is usually employed and block depth is typically restricted to 120mm. Using depths greater than this leads to excessive variation in density within the block as a result of high internal friction. Ideally block depth should be 100mm or less.

Sandcrete/sandcement is here defined as a permanent and durable material formed from a washed sand, a natural sandy soil or a modified sandy soil such that cohesion of the freshly moulded block is insufficient to allow unsupported handling or stack curing. Block depth may be greater than 100mm and is typically 230mm. Dynamic compaction is commonly used which produces more uniform compaction resulting in sufficient strength for the block to retain its moulded shape, though not enough for unsupported handling or stack curing.

The key differentiating factors between soil-cement and sandcrete are then cohesion/strength of the freshly demoulded block and the block size. During the course of the survey it was found that block size effectively determined the marketed name, large blocks were sold as sandcrete while smaller blocks were sold as soil-cement. The exceptions to this were in South Africa and Botswana where cement stock bricks are common. However these are typically smaller than soil-cement blocks, 100x225x87.5mm (width x length x depth) compared to 140x290x100mm for soil-cement and 150x460x230mm for sandcrete.

Stabilisation is also possible with alternative cementitious binders such as lime. The following report deals only with ordinary portland cement. At present this is the most widely available and quality-consistent stabiliser and is likely to remain so for at least the next ten years. Even if lime were to become widely available with assured quality, lime stabilisation requires at least twice as long for initial curing. As quick curing has a significant economic value in block production, lime use is likely to remain less common than ordinary portland cement (hereafter cement).

1.2 Methods of forming cement-stabilised building blocks

In all cases blocks are formed by the application of energy to a loose soil/sand-cement-water mix in a mould that determines all but one of the final block's dimensions. The commonest forming processes are

(i) hand-tamping of a moist mix into a wooden mould with no top or bottom, placed on a flat surface; the mould is removed prior to curing the block in situ on that surface. This process was seen in all of the surveyed countries, primarily used for the production of decorative ventilation blocks. Research into the use of this process for labour intensive methods of soil-cement block production has been carried out in South Africa but it is not currently used in the field.

(ii) pressing in a rigid steel mould with a force of up to 10 tonnes (pressure up to 2 MPa) onto the bottom face of the block; the force is obtained by using levers to amplify the pull (say 500N) of an operator; the best

[3] Poured concrete requires compaction to remove air entrained in a viscous liquid which is a different operation to the compression of a damp powder. Moreover such concrete must be contained within a mould until it has hydrated sufficiently to retain its cast shape on removal from the mould.

known machine type operating in this way is the Cinva Ram press. This process was seen in all countries and used exclusively for the production of soil-cement or unstabilised soil blocks.

(iii) pressing with a force greater than 10 tonnes (pressures typically between 2 MPa and 10 MPa) using hydraulic cylinders such as in the Brepack machine. This process has been used in Botswana, Tanzania and Ghana for the production of soil-cement but is no longer in use.

(iv) slamming a hinged and weighty lid onto the exposed top of a mix that has been hand-tamped into a mould. This process was seen in all countries surveyed for the production of sandcrete blocks and also for the production of soil-cement blocks at Camartec in Tanzania.

(v) vibrating a mix in a mould to which some modest pressure or shock impulse is applied towards the end of the process: the vibrator may be powered by electricity or an engine; the blocks may be moulded onto the ground directly by a mobile machine and left to cure there or they may be carried on pallets from a stationary machine to a curing area. This process is used exclusively for sandcrete and was seen in all of the countries surveyed.

1.3 Compaction and Densification

It was shown by Lunt (1980) that higher compaction pressure up to 10 MPa, generated quasi-statically, has beneficial effects on compacted density and cured strength (research conducted on lime-stabilised soil). Subsequently several machines were produced, eg the Brepack, which used a hydraulic circuit to achieve compaction pressures up to 10 MPa. Higher density blocks are easier to handle between moulding and curing, have a higher compressive strength after curing and also an improved surface hardness. The first is important to reduce the incidence of damage during handling and to permit the stacking of green blocks during curing (thereby facilitating good curing and reducing the size of any curing yard). The second is important because standards for building materials are usually expressed in terms of bulk compressive strength. Surface hardness is important as lack of it results in rapid rain or wind erosion or requires a render to be applied to protect the blocks.

By increasing the compacted density of the block, whether soil-cement or sandcrete, the stabiliser content may be reduced for a given strength, thus reducing the cost of the block. However experimental research[4] conducted by the DTU has shown that the cement saving resulting from higher compaction pressures is not enough to offset the increased capital cost of a quasi-static high pressure machine, unless production output is dramatically increased. Additional advantages of high density production were noted by the DTU, namely an increase in the allowable range of particle grading for the material to be stabilised and improved resistance to poor curing as a result of reduced block porosity. These factors do counteract the greater cost of high density compaction but not sufficiently to encourage the use of manual quasi-static high pressure machines.

Block density is not solely determined by the maximum compaction pressure that the forming process could exert. In the case of fixed-volume compaction the amount of soil placed in the mould is highly significant. Too little material and a low density block is produced, while too much material and the machine is over-stressed and liable to jam. Moreover if the material is not compacted at its optimum moisture content, lower density will result. If too little water is present, internal friction is high and densification prematurely ceases. If too much water is added then hydrostatic conditions may be generated where the applied compaction force increases the pressure of the material's pore water but does not result in particle rearrangement and densification. Variable water content causes a further complication as the volume occupied by a damp soil also depends on its moisture content. A dry soil initially expands as water is absorbed, up to a point known as the fluff point, more than this amount of water and the soil volume again decreases.

[4] The relationship between applied compaction pressure, cement content and cured strength was determined empirically in a laboratory setting. This work is described in DTU Working Paper No.40 "Quasi-Static Compression Forming of Stabilised Soil-Cement Building Blocks" (1993).

SURVEY

2 THE TREND TOWARDS INCREASED USE OF CEMENT-STABILISED BUILDING BLOCKS

The traditional building materials common in Developing Countries may be considered to fall into four broad groups; unstabilised soil, fired brick, wood and stone.

Unstabilised soil construction is a widespread construction material in rural areas but is generally seen as undesirable being the bottom rung of the materials ladder. This view is pronounced in South Africa, Kenya and Zimbabwe. Unstabilised soil is not classified as a permanent material under current building regulations which prevents its legal use in urban districts, leaving the home occupier vulnerable to dispossession and the dwelling vulnerable to demolition. None of the survey countries define urban unstabilised soil construction as permanent. Finance organisations are highly unlikely to lend money for the construction of any property built from material not classified as permanent.

Use of unstabilised soil is likely to continue in rural areas where it is freely available (dug on site) and the cost of construction is primarily determined by the cost of labour (which is considered free in a self-build situation). A French organisation, CRATerre has been involved in the promotion of improved architecture to extend the life of unstabilised soil structures, however despite the existence of some admirable demonstration houses unstabilised soil remains firmly fixed in the minds of Developing Countries residents as being second rate. The overwhelming demand in all of the countries surveyed is for "something better than soil". In areas where soil walling is common it is seen as a temporary structure, built because no other alternative material could be afforded. It seems likely that unstabilised soil will remain associated with poor quality and will always be chosen as a last resort by those with limited means. In consequence its use will continue for the foreseeable future in rural areas but not in urban ones.

Fired brick is one of the cheapest building materials where supplies of suitable soil and firewood are present. The quality of burnt bricks was found to be highly variable. In Kampala the soil used has a low clay content and high amount of sand. This is an unsuitable soil for fired brick production, the bricks produced are low quality being porous and even liable to collapse in the rain. These bricks were observed to be highly fissured and bent even before firing. The cost of fuel for a single clamp of bricks in Arusha, Tanzania is 300,000 Tsh (£400). This alone contributes 25% of the final cost of the bricks. Traditionally wood has been the most common source of fuel for brick firing but supplies are rapidly diminishing and have already been exhausted in many areas, in desperation dried cattle dung is now being used by Kenyan artisans south of Nairobi. Work is under way to find alternative sources of fuel; waste wood shavings (Ghana), furnace slag (Botswana and Zimbabwe) and coffee husks (Kenya), all of these are being used with varying degrees of success. In areas where the price of firewood is high, brick production falls in to two categories: high cost bricks produced using adequate quantities of firewood and poor quality bricks using inadequate quantities of firewood with consequent under-burning. It is likely that the use of fired brick will decline in the lower income groups. This trend may be delayed in areas with suitable soil and current reserves of wood but unless the deforestation process is reversed quickly these areas too will see an escalation in cost.

In many areas sawn timber is now one of the most expensive construction materials and consequently one of the least popular, particularly as ongoing preservative treatment to counter the termite threat is expensive. Many squatter settlements are built with waste wood, as seen to some extent in all of the countries surveyed, but this is classified as non-permanent housing and is always vulnerable to demolition by the town authorities. Split bamboo although still widely used in parts of the humid tropics is not an important building material in any of the nine countries visited. Wood will continue to be used for roof support but unless sustainable forest husbandry is successfully promoted its use as a permanent walling material will continue to decline.

Stone is a common building material in areas where soft easily quarried deposits are found. The cost of the material is determined by the labour costs of extraction and dressing and the transport cost of supply to the construction site. Suitable stone is mainly found in Eastern Africa, most notably Nairobi, Kenya where volcanic tuff has been quarried since the 1940s. The early local quarry sites are now becoming worked out and the cost of the material is increasing because the transport costs are escalating as the quarries become more remote. In areas where accessible supplies exist its use is likely to continue, as it is seen as a highly durable material with a low initial purchase price. Stone housing construction by individual home owners has been following an ***extending*** pattern. This pattern of construction has been noted in all of the countries surveyed and is discussed in more detail below in section 2.1.

In all the countries surveyed except Uganda cement-stabilised blocks are becoming the most common urban walling material, despite the increasing cost of cement (a result of internal economic difficulty and structural adjustment programmes). They are steadily displacing the many forms of unstabilised soil, fired brick, wood and stone that have been traditionally used. These blocks owe their popularity to their image of modernity, strength and durability, although at present many of the blocks do not live up to these expectations. They are easy to produce with very little equipment, generally relying more on labour than machinery. Their large unit size compared to stock bricks offsets their higher purchase cost per piece as fewer are required per meter of walling. The large unit size and regularity speeds construction, reducing labour costs and requires less mortar. In all of the countries surveyed current methods for producing soil-cement were found to be capable of producing less expensive block-walling than sandcrete. However soil-cement is currently much less popular than sandcrete because of its stigmatising associations.

At present the majority of these blocks, both soil-cement and sandcrete are not reaching their potential strength or durability, defined by the quantity of cement used in their production. There is great potential for reducing the cost of these materials *and* increasing their quality. Although cement is an expensive industrial product, it is made on a large scale and is widely available, exceptions are remote rural areas such as the Kalahari in Botswana where the price rises dramatically with the distance from supply centres and Ghana where, until recently, supply has been largely restricted to government contracts. With improved production methodology the quantity of cement used in blockmaking and hence the cost of the blocks may be reduced. In contrast, the traditional materials although indigenous are becoming more scarce, particularly firewood, and in consequence more expensive.

There are many possible options for improving the provision of housing, one is to fight against public opinion and promote more traditional building methods, for example improved soil architecture. Another is to promote sustainable forestry to supply wood both for direct use in construction and for fuel for brick firing. A third is to improve the efficiency of cement use in cement-stabilised building blocks. With improved soil/sand selection, more efficient compaction and in particular well controlled curing (see below), the cost of these blocks may be reduced substantially, significantly reducing the quantity of cement consumed per unit of walling, while at the same time maintaining or improving strength and durability.

In the short term at least, it seems likely that cement-stabilised materials will continue to increase their market share. It is generally easier to improve a popular product with an established and expanding market than to revive one with a failing reputation. Moreover it is possible to improve the efficiency of cement-stabilisation through only minor modifications of current production practices. Following the recent rapid increase in cement price in many of the areas visited (the Kenyan price rose from 170 Ksh to 370 Ksh in a matter of months, while the Tanzanian price rose from 1400 Tsh to 4000 Tsh in one year), methods of reducing the cement content per block will be welcomed by the existing manufacturers. It is a more complex proposition to find widespread alternative fuel supplies for brick firing or to implement widespread sustainable forestry practice. As the need for improved affordable housing is current and very pressing it seems preferable to promote methods of reducing the cost of cement-stabilised material now.

2.1 The *Extending* pattern of house construction

In this section the *extending* pattern of construction, common in urban areas of the countries visited, is explained and its influence on materials choice is explored.

Typically a house is built on a grand scale as the owner's "ideal" house. It is not uncommon to let its construction take ten years, divided into many stages of construction and payment, rather than to build an immediately affordable smaller house. Initially a site is obtained and no construction work takes place until the site has been paid for. Then blocks are purchased and stockpiled until enough have been acquired to construct a section of wall whereupon a mason is employed. The stockpiling and block-laying phases are then repeated until the house is ready for roofing. In this way the home owner may use a higher quality material than could otherwise be afforded.

In the particular case of Nairobi where stone is believed to be the highest quality construction material, rough hewn stone is purchased very cheaply and a mason employed to provide the final dressing as and when finance is available. Once enough stone has been purchased and dressed the mason is then employed to lay the blocks and so on. Stone is suited to this pattern of construction as each stage of construction is cheap, even though in total the final cost of the wall may be higher than one constructed from sandcrete blocks.

While the house is under construction the eventual owner is usually renting accommodation elsewhere. The rapidly increasing urban population in many areas is causing the price of rented accommodation to increase rapidly, in Botswana the urban population is increasing at 6.9% per year. Similarly the population of Nairobi is increasing at 7% p.a. In Arusha rent for a 10' by 12' room is currently 7000 Tsh a month, payable for one year in advance (the minimum wage is 15000 Tsh a month). Such high rents are discouraging the *extending* pattern of house building described above. As rent rises so it becomes more attractive to speed up the construction process even if this means a smaller house; the quicker the owner can occupy the house the quicker he/she stops paying rent. Once construction speed becomes an issue then the total cost of the house is considered rather than the cost of its components. For example although undressed stone is cheap to purchase, the cost of subsequent processing and construction is high. Because the stones are irregular, a thick layer of mortar is required and consequently a substantial quantity of cement is used. Similarly a substantial quantity of cement is required for the render. In contrast cement-stabilised building blocks although marginally more expensive to purchase initially, do not require final dressing and are regularly sized using less cement both for mortar and render. As general awareness of the financial drawbacks to *extending* construction increase it is likely that it will become less popular, perhaps becoming replaced by a *core-extension* pattern.

2.2 The *Core-extension* pattern of housing construction

Core-extension housing was observed in the redeveloped squatter settlement of Old Naladie in Gaborone, Botswana. The situation is slightly different in Old Naladie as the future home owner already owns the plot of land and usually occupies a temporary structure on the site. In this case the drive for rapid construction is to allow income to be generated by renting space in the house to third parties. Small twin room houses are initially constructed (using sandcrete), one is occupied by the house owner while the other is rented out. The rental income is then used to provide finance for further extensions to the house which are used either by the home owner or also rented out to generate further income.

Once the advantages of faster construction become more generally appreciated it is likely that both a reduction in the size of the initial construction and an increase in the use of more affordable materials will be seen.

3 CURRENT PRODUCTION OF CEMENT-STABILISED BUILDING BLOCKS: PRACTICES AND PROBLEMS

3.1 Process methodology

3.1.1 Curing procedures

The most detrimental practice seen in all of the countries surveyed was poor curing of the formed cement-stabilised block. Once formed the blocks are frequently left out in the sun to "dry", large areas of sun-dried "curing" blocks were observed in every country surveyed. Cement relies on the presence of water to hydrate, forming an interlocking skeleton of calcium silicate hydrate which gives the material its strength. If the block is allowed to prematurely dry then full hydration of the cement does not occur and consequently only part of the cement used contributes to the strength of the block. Experiments conducted by the DTU at the University of Warwick have shown that the strength lost due to poor curing can easily reduce the final block bulk strength by 20%. As the block surfaces lose water first, strength loss in these regions is still higher. The loss of surface strength reduces both the handleability (edge and corner chipping during transport) and the durability of the blocks. If proper curing were implemented, maintaining the moulding water content for at least seven days, then both strength and durability improvements would be seen. Good curing practice is not followed for one of two reasons. Either the producer is not aware of the need for curing (instances in all countries surveyed) or it is felt that the cost of constructing a suitable curing area is not worth the potential increase in quality. In Kampala where sandcrete blocks are produced by "egg-laying" vibrating machines, producers observed that wet weather, provided heavy rain did not pit the newly moulded blocks, gave better curing. However they were reluctant to use cloths to increase humidity during curing and also indicated that shading the blocks would be unacceptably expensive, given the large area which would have to be covered.

With conventional moulding methods the strength of the green blocks is not sufficient to allow stacking of the fresh blocks and hence a large sheltered curing area is required. Improved compaction produces higher density green blocks which may be stack cured, greatly reducing the area needed. Furthermore a higher density block loses water more slowly as a result of its reduced porosity and consequently is less susceptible to poor curing practice.

3.1.2 Batching

Cement hydration begins as soon as the cement comes into contact with water. In consequence the batch-time, the length of time between addition of water to a cement mix and the use of the last part of a batch, is important. Ingles and Metcalf (1972) suggest that as much as 50% of the final cured strength of cement-stabilised soil may be lost by a delay of two hours before compaction. Experiments conducted by the DTU confirmed a strength loss due to compaction delay but found it to be less pronounced, namely 20% loss after a two hour delay. The significance of batch time was not understood by field producers and consequently batch times of two hours or more were common, in isolated instances batch times up to six hours were found (St Joseph's Mission, Nairobi) which would result in at least a 50% strength loss for blocks produced at the end of a batch. Batch times of 30 minutes are recommended and it is advisable that times do not exceed 1 hour.

3.1.3 Optimum water content

The amount of water added to the cement mix is also important for good compaction during moulding. Moulding at the optimum water content results in the most dense block yielding the greatest strength. If too much or too little water is added the formed block will be less dense. This fact was not fully appreciated by any of the block manufacturers visited and consequently variable moisture contents were used at moulding. This fact also contributes to the argument for shortening batch-times, as water is progressively lost from the mix both in the hydration of cement and also by evaporation.

3.1.4 Raw materials testing

The material, either "soil" or "sand" to be stabilised is not adequately tested and the importance of the fines content is not understood. Thorough soil testing has always been advocated for soil-stabilisation but not for sand stabilisation. It has been found in the field that what is sold as sand, which should contain minimal quantities of material finer than fine sand (0.063mm) frequently contains high quantities of such fines. CSIR in Pretoria, South Africa have found supplies of "building sand" composed of over 50% clay. More commonly contamination is in the order of 25% fines, as observed in Zimbabwe, Kenya, Botswana and Ghana for unwashed pit sand. The proportion of silt and clay (the fines) present in the material to be stabilised plays an important role in determining the amount of cement needed for a given degree of stabilisation. It is the fines, particularly the clay fraction, which expand and contract on wetting and drying and consequently affect the durability of the cured block. Without an understanding of both the effect of the fines and the quantity present it is unlikely that the optimum use of cement will be made.

Although testing is advocated for soil stabilisation it has been found that this rarely happens. In fact in Tanzania the training given by Camartec to purchasers of their block press (Cinva Ram type) does not include how to test the soil. Instead Camartec technicians conduct soil tests on site and report to the producer the quantity of cement required. This is only a very short term solution to the testing problem as the composition of soil is highly variable and unlikely to remain at the tested composition once soil extraction extends from the immediate vicinity of the soil samples taken. Once the composition has changed significantly, the recommended cement content should also change. At present this does not happen and consequently cement use is not optimised.

The soil testing methods reported in the literature have been found to be lacking (see DTU working paper No 38 "Soil Testing for Soil-cement Block Production"), yet these publications usually form the basis for NGO organisations' knowledge. Hence where soil-testing information has been disseminated to block manufacturers by such organisations the information was found to be incomplete, faulty or misunderstood. High quality blocks may be produced with no soil testing whatsoever if adequate trial block production takes place. However without some form of testing and understanding, this process is extremely lengthy and was not observed in any of the countries surveyed.

3.1.5 Quality control

Quality control is usually not appreciated by the block manufacturer, in consequence there is a large degree of variation in quality, both between manufacturers and within the stock of a single manufacturer. Neither testing of the green compact nor testing of the cured blocks was observed. Most NGO projects had tested blocks at the start of production to determine the optimum cement content for the required strength but had not continued testing subsequently. Blocks produced by St Joseph's Mission (Kenya) were recently tested for compressive strength by the Kenyan Standards office and found to be only half of the value expected, 0.7 MPa compared to 1.5MPa. This is not surprising as the production of blocks had begun 12 years earlier, methodology was passed from operator to operator, degenerating over the years, and no quality testing had been implemented to monitor gradual changes.

Testing of the green blocks' density would identify production problems at an early stage, allowing quick remedial action to be taken. Testing of representative sample blocks for cured strength would serve as both an overall check on the production system and a useful marketing tool, namely the adherence of the block to the local building regulations. However although building regulations exist in nearly all urban areas, defining the minimum allowable compressive strengths of walling materials, they are not effectively enforced for low rise construction. Botswanan site inspectors rely on a purely visual assessment of the blocks (source Mr Maititing, Acting Director of the Department of Architecture and Building Services). Moreover the only one of the surveyed countries to have passed a standard relating specifically to soil-cement was Kenya (KS02-1070, 1992), in the other countries either no standard or only an unadopted draft standard exists.

Although the cement-stabilised production process is a simple one, it relies very heavily on tight quality control to achieve good results. The following is a summary of the factors which can cause block defects if not adequately monitored;

- soil/sand composition may vary considerably even if dug from a single pit
- inadequate mixing can produce a highly uneven distribution of cement
- mixing too large a batch of stabilised material at one time can reduce strength due to premature cement hydration

- incorrect moisture content at the time of moulding adversely affects the efficiency of compaction
- variations in the volume of mix placed in the mould for compaction affects the final density of the block and can seriously damage the machine
- inappropriate curing will allow the block, in particular the block surface, to lose the water required for full hydration of the cement, causing low strength blocks with poor surface durability

3.2 Compaction systems

3.2.1 Hand tamping

This method of production was only observed for small, but relatively high value decorative ventilation blocks, carried out exclusively in the informal sector. These are particularly common in Arusha and Dar Es Salaam (Tanzania). It was not observed for larger sized building blocks except in one small Tanzania town and for an experimental investigation conducted by CSIR in South Africa, as the labour cost becomes excessive compared to mechanical compaction.

3.2.2 Low-pressure compaction

The most common compaction equipment used for "soil" stabilisation is based on the Cinva Ram machine, invented in the 1950's. This slowly applies a pressure (usually less than 2 MPa) to the mix. These machines are generally produced in the informal sector although formal production does also occur. Table 3.2.2 details the most common type of machines observed in the African countries surveyed. Machine cost varies significantly with country of manufacture and quality of construction from £63 for a machine produced in Ghana under the supervision of the University of Science and Technology (UST) to £526 for a high quality machine incorporating sealed bearings, the Shelter Press made in Zimbabwe, commissioned by Intermediate Technology (Zimbabwe). The compaction pressure is applied mechanically by transmitting the force exerted by the operator to the contained mix through an over-centre toggle lever arrangement. There are a number of common problems both with the manufacture and use of these machines.

TABLE 3.2.2 Cinva Ram type machines observed in Africa

Organisation	Country	Block size	Compaction Ratio	Novel features	Cost /£
Camartec	Tanzania	140x290 x100mm	1.65:1	none, has poorly aligned mould box	92
Approtec	Kenya	140x290 x115mm	1.7/1.9:1	variable compaction ratio, secondary pivot to ease block ejection, slam-shut lid	321
IT (Zimbabwe)	Zimbabwe	140x295 x120mm	1.6:1	Sealed bearings, sliding mould lid to automate removal and strike off after filling	526
RIIC	Botswana	150x300 x115mm	1.6:1	none, has piston guidance problems	159
UST	Ghana	200x290 x150mm	1.5:1	mould top linked to compaction handle so that removal is automatic	63

The construction quality of these machines was found to be highly variable, some machines are manufactured using jigs to ensure correct alignment of parts (Approtec, IT Zimbabwe and UST) while others are not (RIIC and Camartec). Non-parallel mould boxes and misaligned compaction pistons were the most serious common faults, producing sub-standard blocks and quickly jamming and breaking down, sometimes after only weeks of use (experience of Habitat for Humanity, Botswana). Some of these machines have been modified from the original Cinva Ram design to include useful innovative improvements. A significantly improved machine could be produced by amalgamating the useful features seen individually world wide in many of these machines. However this is unlikely to happen without external assistance as the manufacturers/designers are not aware of machines outside their own locality. Several organisations are currently working separately to improve Cinva Ram type machines, including RIIC, Botswana Technology Centre (BTC) and IT Zimbabwe. If these organisations could effectively liaise then progress would be much faster and duplication of mistakes minimised. Both RIIC and BTC are working independently on interlocking block designs. The organisations are within the same country and less than two hours drive apart but are not currently collaborating.

Most of the machines operate on a fixed compaction ratio (typically 1.6:1), this determines the volume of the mould box at the time of filling. Different soils have different densities at the time of moulding and require different degrees of compaction in terms of the ratio of loose to compacted volume. This necessitates some form of batching to place the correct amount of mix in to the mould; too little and the block is under-compacted and weak, too much and the machine is over-stressed and liable to break. Batch-box filling was not observed in the field, both under-compacted blocks and machine breakdown were common. Overloading of the mould box was by far the most common cause of breakdown yet Approtec were the only organisation to have included the ability to vary the compaction ratio in their machine.

The significance of mould friction is not appreciated by block manufacturers. Research conducted by the DTU has shown that mould wall friction can significantly reduce the effectiveness of any applied pressure. To minimise this the mould should be lubricated with a release oil. This not only improves the compacted density of the block but also improves the block's surface finish and eases ejection. Mould lubrication is not currently common in any of the countries surveyed.

3.2.3 Manual high pressure quasi-static compaction

This type of machine uses the Cinva Ram toggle lever to first provide the majority of the compaction; final compaction to high pressure is then achieved by operating a hydraulic ram which acts on the moving piston. This type of machine is produced only in the formal sector. It is no longer in current use as it is very costly to purchase and has a reduced production speed since the hydraulic ram must be operated in addition to the toggle lever for each cycle. Although savings may be made in the cement used for a given degree of stabilisation the increased capital cost of a high pressure slow-squeeze machine (£ 3000-£4000) outweighs these savings. (See section 5.1 for new developments in the field of alternative high pressure compaction). Powered high-pressure compaction machines are also available but these are much more expensive eg the Ceratec machine seen in Botswana which costs £24,000, excluding ancillary powered mixing and sieving equipment.

3.2.4 Slam-shut low-pressure compaction

The slam-shut compaction machine was found in all of the countries surveyed. The compaction ratio of these machines is very low, a maximum of 1.3:1 if the mix heaped above the top of the mould is included. Consequently the mix is heavily pre-compacted by hand-tamping before repeatedly slamming the mould lid to achieve final compaction. This machine is made cheaply in the informal sector and its origins are unknown. It is less complex than the Cinva Ram and less prone to manufacturing problems. It generally costs slightly more than the cheapest Cinva Ram machine, eg £110 compared to £93 (cost comparison for Tanzania). In Ghana a slam-shut machine costs less than a Cinva style machine, 60,000 C compared to 111,000 C, but 300 wooden pallets are also required at an additional cost of 400 C per piece, increasing the effective cost to 120,000 C. This machine applies a variable compaction energy to the mix, depending on the degree of hand-tamping employed, the amount of mix contained in the mould and the number of blows applied by the lid. Consequently quality and consistency are dependant on the degree of care exercised by the operator. One informed Tanzanian manufacturer, Kunda Co Hardware in Arusha, attempted to ensure quality consistency by closely supervising production, always using the same operators for the same task and ceasing production if one of the team became ill, rather than hire a temporary replacement. Again batch-box mould filling was never observed.

Typically six blows are applied which equates to an applied energy of approximately 1kJ/block[5] (assuming an effective drop height of 30cm, for a mould lid weighing 15kg which is thrown down onto the mix by the operator applying a force of 400N, repeating the operation six times). The energy applied per block falls to 0.5kJ when two blocks are formed at once as is the case for some machines observed in Tanzania. 1kJ is a low level of applied energy. The DTU has found that slow compaction to the low pressure of 2 MPa requires an applied energy of approximately 2kJ (calculated for a block 290x460x100mm).

The low compaction achieved with slam-shut machines results in low density blocks with little green strength. In order to allow transportation from the compaction machine to the curing area the blocks are moulded on a wooden pallet which is then used to carry them. They are too weak to allow stack curing and consequently curing normally follows the "sun dried" pattern. Improving machine design to increase the amount of energy applied

[5] The standard size of block produced by these machines is either 290x460x100mm (4") or 290x460x150mm (6").

would allow the quantity of cement used to be reduced and also increase the green block strength, allowing further savings to be made by facilitating stack curing.

3.2.5 Powered vibro-compaction

In this process machines powered either by electric or internal combustion motors utilise vibration coupled with a very low confinement pressure to compact the mix; some machines finish the compaction cycle with an impact blow of moderate energy. Two types of these machines are common. The moving "egg-laying" type compacts several blocks at once; these are laid directly onto the ground where they are cured without moving (sun dried). The stationary machine produces one or two blocks per cycle which are ejected from the mould on a pallet (a pallet is placed in the bottom of the mould prior to filling) and carried to a separate curing area (sun dried). These machines are produced in the formal sector and are expensive, costing £6000 in Zimbabwe. They require a wetter mix for successful compaction than the impact machines and blocks are prone to slump. The size of the final block is dependant on the wetness and amount of mix placed in the mould and the length of time for which vibration is applied. The mould is filled and scraped off flush. However only volume and not weight of the charge is fixed; block heights were observed to vary as much as 10mm in Harare.

4 ECONOMIC ANALYSIS OF BUILDING MATERIALS COMPETING FOR THE URBAN AND PERI-URBAN MARKETS

Table 4.1 (below) shows the raw data, labour rates, cement costs etc which were gathered in the field and subsequently used to perform the economic analysis contained in the relevant country appendix. It also shows the best case for the costs of built-up walling using respectively, standard-size blocks made by conventional quasi-static compaction of soil-cement to low pressure, large-size blocks made by impact compaction of soil-cement to high pressure, sandcrete blocks, burnt bricks and quarry stone. In the Appendix J four methods of soil-cement production were modelled (using Kenyan costs). The most efficient of these methods, peri-urban micro-enterprise production, was the only method to be used in the subsequent analysis conducted for other countries. The full methodology is detailed in Appendix J and used for the remaining countries examined.

The ratio of labour wage to cement cost, row 14 in table 4.1, was thought to provide a good indicator of the economic viability of soil-stabilisation; however this has proved to be incorrect. Although this labour/cement ratio varies strongly from country to country, the cost ratios of standard block soil-cement walling to sandcrete walling (row 15) and of large block impact formed soil-cement walling to sandcrete walling (row 16) vary little. The results of the economic analyses show that using current low-pressure production, switching from sandcrete to soil-cement will reduce walling costs by between 11 % and 35 %. This is significantly less than the figure of 50% which has been quoted in the past. In previous studies only the cost of the individual walling elements have been compared, whereas in our study the cost of a built-up wall has been used for comparison. Consequently small blocks are disadvantaged. A larger number must be used per square meter of walling (usually considered in earlier analysis), more mortar is required (not normally considered) and they take more time to lay (not normally considered). Although this type of production method is less expensive than sandcrete, a cost advantage of 30% or less is unlikely to encourage the uptake of the technology in areas where social stigma is a factor. It should be noted that for this analysis the two production figures used (200 and 400 blocks per day for low pressure compaction) define the range generally achieved by self help and NGO projects, while the prices quoted for sandcrete are those charged by commercial yards where the production rate is much higher, even if quality is low. For example although 200 and 400 blocks per day was used as the basis for the Action Pack block press output, Approtec believe that with a suitably trained and motivated workforce the daily output could easily reach 800. If such outputs were seen in practice, as should be the case for micro-enterprise rather than self-help production, then this method becomes more attractive.

However with production rates of only 160 and 320 blocks per day, the hypothesised large-size, impact compacted block would cost significantly less than any other available durable walling material. The cost efficiency seen for this type of walling is principally a result of the method of compaction. Impact compaction, resulting in high uniform densification allows a deeper block to be produced which has enough strength to allow hollowing. The hollowing of the block reduces the mass of stabilised material used per square meter of walling. The high block density allows the percentage of cement to be reduced for a given strength. The larger size of the block allows further savings to be made in laying time and the amount of mortar required. Research conducted by the DTU has also shown that impact compaction, by virtue of its superior densification, will also be more tolerant of poor production methodology. It has also shown than a wider range of soils may be economically stabilised and that reduced porosity reduces the rate at which water is lost from the compacted block during curing.

TABLE 1 COSTS FOR LABOUR, CONSTRUCTION MATERIALS AND BUILT UP WALLING IN THE PRINCIPAL COUNTRIES OF FOCUS (figures given are in local currency)

	KENYA	TANZANIA	BOTSWANA	GHANA
1 exchange rate	68.5 (£1)	865 (£1)	4.22 (£1)	1780 (1)
2 unskilled daily labour wage	70 (£1.02)	1000 (£1.16)	5 (£1.18)	1500 (0.84)
3 foreman wage /day	250 (£3.65)	2000 (£2.31)	35 (£8.29)	3500 (1.97)
4 skilled construction labour /day	200 (£2.92)	2500 (£2.89)	35 (£8.29)	3500 (1.97)
5 cement cost /50 kg	370 (£5.40)	4000 (£4.62)	10.95 (£2.59)	6000 (3.37)
6 sand cost /tonne	1000 (£14.60)	4286 (£4.95)	22.6 (£5.35)	7600 (4.27)
7 soil cost /tonne	429 (£6.26)	1714 (£1.98)	1.7 (£0.40)	517 (0.29)
8 machine cost (Cinva Ram type)	22000 (£321)	80000 (£92)	670 (£159)	111625 (63)
9 cost /m^2 for internally rendered wall built with std size soil-cement block	327 (£4.78)	2970 (£3.43)	16.45 (£3.90)	4360 (2.45)
10 cost /m^2 for internally rendered wall built with large size soil-cement block produced by impact	213 (£3.11)	1866 (£2.16)	11.36 (£2.69)	2551 (1.43)
11 cost /m^2 for internally rendered wall built with sandcrete blocks	457 (£6.67)	3341 (£3.86)	25.21 (£5.97)	4880 (2.74)
12 cost /m^2 for internally rendered wall built with burnt bricks	n/a	3454 (£3.99)	19.79 (£4.69)	4796 (2.69)
13 cost /m^2 for internally rendered wall built with quarry stone	418 (£6.10)	n/a	n/a	n/a
14 ratio of labour wage to cement cost	5.29	4.00	2.19	4.00
15 ratio of soil-cement to sandcrete cost std size	0.72	0.89	0.65	0.89
16 ratio of soil-cement to sandcrete cost large size	0.56	0.56	0.45	0.52

INTERVENTIONS

5 RECENT DEVELOPMENTS IN THE THEORY OF CEMENT-STABILISATION OF BUILDING BLOCKS AND PERTINENT INTERVENTIONS

5.1 Compaction, the effects of density on block quality and the superior densification of impact compaction

Research over many years, including that undertaken since 1990 by the DTU, has shown that block quality depends upon three main factors. These are materials selection, densification during moulding and curing. The effectiveness of the first and last of these depends upon the block-maker's skill and the expenditure on cement. Densification depends on the moulding machinery used as well as the moulder's skill. New knowledge in this field relates particularly to the densification process and its consequences: the most recent findings are summarised as follows.

The advantages of increasing the pressure of conventional quasi-static compaction were presented in section 1.3. An alternative method of generating high but transitory forces is through the use of a dynamic lever. A weight lifted slowly through a height gains potential energy. Once the weight is dropped its potential energy is transferred to kinetic energy. When the weight strikes a stationary object all of the stored kinetic energy is released in a very short distance, generating a large force. With quasi-static mechanical machinery the full compaction force must be transmitted through elements which move relative to each other, leading to high bearing wear rates and a short machine life, moreover this force must be transmitted through the body of the machine hence requiring it to be relatively massive. Impact machinery does not suffer these drawbacks nor does it suffer from the complexity associated with the inclusion of the hydraulic circuitry required to achieve compaction pressures above 2 MPa.

Recent research conducted by the DTU has found dynamic methods of compaction, utilising impact blows are also capable of producing high density blocks. Such blocks have a more uniform distribution of density and require lower levels of applied energy than do slowly pressed blocks. For example compaction to a mean density equivalent to that resulting from a quasi-statically applied compaction pressure of about 10 MPa was found to require 43% less energy. Moreover the improved density distribution seen with impact compaction was found to equate to a 24% increase in compressive strength for a given mean density. In combination these factors result in a 50-75% overall saving in the energy required to achieve a given compressive strength. In addition one of the main problems with quasi-static compaction, difficulty of block ejection, is overcome with optimised impact compaction. The dilation and subsequent contraction of the impacted material (which allows the more uniform density distribution to develop) also reduces the final lateral pressure exerted by the mix on the mould walls and consequently greatly eases ejection.

5.2 Possible interventions, immediate

There are thus four prime areas in which improvement may be made to the production of cement-stabilised building blocks; curing practice, production methodology, soil/sand selection and processing/compaction equipment. Crucial to the process of improvement is increased understanding of the pertinent factors both by NGOs engaged in promotional and support activities and the block producers themselves.

The prevalent poor curing practices observed in the field cause a significant under-attainment of compressive strength and durability. This is typically 20% but depends on the degree of adversity experienced by the blocks during curing, the amount of exposure to direct sun, air temperature, relative humidity and wind speed. Improved curing practice to maintain the moulding moisture content for at least seven days is very simple to implement, requiring no additional equipment other than a covering for the blocks. The intervention identified here would be to provide training to block makers, demonstrating the improvement in block quality resulting from good curing practice. Good curing resulting in either a stronger more marketable block or alternatively a cheaper block as a result of the cost saving resulting from cement optimisation if strength was maintained at the current level. Strength is approximately proportional to cement content, so a process-related 20% improvement in strength for a block where cement cost comprises 60% of the total cost could reduce the block cost by 12 % (the cement content may be reduced by 20%).

Similarly immediate improvements, either reductions in cost or increase in quality, will result from improved understanding, used to implement better production practice. Reduced batch times, optimum water content used for moulding, consistent mould filling (batch-box filling) and consistent compaction all increase the quality of the cured blocks.

Correct use of simple soil/sand testing procedures will enable the most appropriate use of the available material to be made, identifying unsuitable soil/sand which should not be used and allowing discrimination between alternative materials. Proper initial use of soil/sand testing can greatly speed the setting up of production while occasional subsequent monitoring can identify changes likely to lead to substandard production.

Improvements to current low-pressure compaction equipment are possible by amalgamating the various refinements seen in individual Cinva Ram type machines. These refinements will increase the production capacity of the individual machines by streamlining their operation and improving compaction consistency. For example a machine with a variable compaction ratio (as is the case for Approtec's Action Pack Block Press, Kenya), once its purpose, adjustment and operation are understood, will result in reduced variation of compacted block density and remove the risk of over-stressing the machine without resorting to batch-box filling.

5.3 Possible interventions, medium term

There are significant advantages in producing blocks of higher density than those obtainable from low-pressure, slam-shut or vibration machines. High-pressure quasi-static compression does not seem to be the answer; as argued earlier it is too slow and the machines are too costly. Impact appears to offer a more economic route to obtaining high and uniform density. Any refined impact machine should find a ready market in competition with the existing slam-shut machines and displace the vibration based machines.

The development of an impact-based machine capable of generating densities equivalent to 10 MPa quasi-static compaction should allow the benefits of high density mentioned above to be achieved in addition to the separate benefits of impact compaction. Any such optimised machine will compact with a much lower energy input than say a hydraulic machine, resulting in less operator fatigue if the machine is manually powered and less fuel consumption if motor driven. The robust simplicity of the slam shut compaction machine and its current popularity with block producers is a good indication that any improvement to this type of machine should find a ready market. A short life and high incidence of mechanical breakdown were the most commonly cited dissatisfactions with the Cinva Ram type machines. The fundamental problems with this type of machine (see section 5.1) are overcome by impact force application. Consequently provided an impact machine's mould box is designed to withstand the fatigue of small repeated shock loading, the machines life and reliability should be superior to the Cinva Ram.

A high-pressure prototype impact machine, the Ranko Block Maker (cost £120), was designed and successfully used by Agas Groth in Botswana in 1985 to produce blocks which were used in the construction of several houses. These houses have now been standing for ten years with no maintenance and are still in excellent condition. They should be compared with neighbouring houses recently built with high density blocks produced using mechanically sieved and mixed soil, compacted with the Belgium Ceratec and South African Hydraform machines which cost £24000 and £14000 respectively. These houses are already suffering from erosion.

6. FACTORS LIKELY TO AFFECT THE UPTAKE OF THE IDENTIFIED INTERVENTIONS

Sandcrete is an established material. The only barrier to extension of its use is its cost. The cost may be reduced through implementing the interventions mentioned. Operators are generally keen to reduce production cost and consequently the uptake of the interventions is likely to be high in this field.

If soil-cement is specifically promoted then there are factors likely to hinder advancement, those mentioned below are the dominant ones which will have to be contended with;

- Soil-cement has frequently been promoted as a low-cost walling material, which it is. However this ignores the social status associated with permanent housing. Namely the owners are prepared to spend ten years building a house rather than use "low-cost" materials because of the social stigma (see below).
- Standards dealing with soil-cement are not yet widespread, consequently soil-cement is not officially recognised as a permanent building material. Therefore planning permission cannot be given for dwellings built from it. Moreover the passing of national standards frequently requires local ratification. Kenya is the only surveyed country where a national standard relating to soil-cement has been adopted (KS02-1070, 1992). Local Kenyan building by-laws are changing but only slowly as these are modified on a district by district basis, frequently hampered by "personal conflict of interest". It should be noted that although standards do exist for sandcrete blocks these are infrequently enforced.
- The need for quality is often not appreciated in the informal manufacturing sector (neither in the soil-cement nor the sandcrete fields) where the majority of block production has taken place. It is one thing to demonstrate a cost reduction resulting from an improvement to a manufacturer's existing production method, which can be appreciated and another to generate interest in training from new manufacturers who believe the process to be simple. Recently a five day course run by Approtec (Kenya) on the proper use of its Action Pack block press machine has had to be shortened to two days in an attempt to increase participation by machine purchasers. The cost of the training course is included in the purchase cost of the

machine so that participants incur no additional fees. The non-attendance of purchasers must therefore be attributed to a lack of appreciation of its importance.

- The literature dealing with soil-cement production methodology is not adequate. Soil-cement is often presented as a simple process while in fact it relies on a significant degree of understanding coupled with a rigorous pre-production testing programme. While the labour force will have been trained in the mechanical operation of the machines which is a simple task, other aspects, such as detailed soil-testing and determining the optimum moisture content have been less rigorously taught and less well understood.
- The field training of existing block producers (both soil-cement and sandcrete) has frequently been conducted by technicians whose knowledge has been gained from the available published literature. This literature is not adequate and consequently the training given to block producers is frequently inadequate. In particular the coverage of methods of soil testing and final block quality are generally not adequate and not sufficiently emphasised.

The soil-cement block must be treated like any other commercial product and subjected to a coordinated marketing strategy. The prime aim of this strategy should be to de-stigmatise the product. Pronounced anti soil-cement stigma was observed in Kenya, Zimbabwe and South Africa. Stigma was also observed in the other countries surveyed but was found to be less pronounced. It is stressed here that at the moment a number of the criticisms which are sustaining the stigma associated with the technology are deserved. The soil-cement blocks currently produced frequently do display poor quality and vary greatly within batches. Although these deficiencies are also commonly shared with informal sandcrete block production it is the blocks marketed as soil-cement which are identified by both the general public and builders as poor.

If soil-cement can be demonstrated to existing sandcrete manufacturers in suitable locations and offered to them as a diversification product to be marketed as a cement-stabilised block, then the uptake of the technology is likely to be sustained (providing that the block quality is maintained). Soil stigma will be averted and the cost of building materials will be reduced. It is recommended that future promotion of both "soil-cement" and sandcrete be combined under the title "cement-stabilised building blocks".

7 TRAINING AND RESEARCH & DESIGN NEEDS IDENTIFIED

Further research is required to determine the most successful method of implementing training and the usefulness of additional ancillary equipment to promote quality control. It is envisaged that three types of training are required, one to provide a proper grounding for employees of NGOs and city councils involved in promoting low-cost housing and two for end users. Of the end user courses one should focus on the practical detail of correct soil preparation and machine operation for foremen/fundis and the other should be for machine owners interested in quality control, cost reduction and marketing.

This training should be fully supported with permanent reference material. The available literature has been reviewed by the DTU and it has been concluded that at present good reference material specifically concerned with cement stabilisation is not available, either to NGO technicians or more importantly to end users. Such literature as there is oversimplifies cement stabilisation: the production process is mechanically a simple one but it requires skill if it is to be cost effective. The soil/sand testing procedures reported in the literature have been found to be misleading and on occasion incorrect. No adequate explanation of the mechanisms underlying stabilisation have been found and in particular the field remains firmly split into separate soil-stabilisation and sand-stabilisation parts.

At present the only NGO/city council orientated training course is run by CRATerre in Grenoble France. This course places an emphasis on CRATerre's own field of focus namely unstabilised soil and improved soil architecture. There is a need for courses focused specifically on cement-stabilisation to be conducted within the developing countries. It is envisaged that a small number of such courses would be well subscribed and have a significant impact, provided that the information presented can be taken away in a permanent reference format.

At present there are almost no end user training courses and those which are available are under attended. The reasons given for the poor attendances were the length of the course and the financial impact to the participant of not working for the course duration. Such courses also have to contend with the additional difficulty of a highly mixed audiences. In Approtec's case (Kenyan NGO) although most of the participants were end users, some of these were foremen/fundis whose prime interest was the machine operation, others were owners whose interest was in optimisation of cost and quality and marketing, while still others were interested in construction methods.

Manual equipment for block production is often poorly designed and purchasers are unable to distinguish good design from bad. To further improve the production process the existing compaction equipment may be improved. The many various designs of machine available worldwide need to be quantitatively assessed. Bad machines must be improved or publicised as being deficient. Generally there is only one common design of machine in each country even if it is produced by several organisations. A competent external agency with no vested interest should be responsible for any such assessment, one outcome of this would be recommendations on simple improvements to individual machines and also if suitable a new amalgamated machine combining the useful features noted on individual machines. The design of any new machine should be circulated to all machine manufacturers.

This survey has indicated that dynamic compaction (impact) can potentially reduce block walling costs by 50%. Hence it appears worthwhile to undertake field testing and promotion of this moulding process. Research is required to develop a commercial "high-pressure" impact machine. Such a design would be based on the experimental research already conducted by the DTU into optimisation of impact compaction and the practical experience gained by Agas Groth with the Ranko Block Maker in Botswana.

REFERENCES

1 ApproTEC.: A Sub Sectoral Analysis of Walling and Roofing Materials in Nairobi and the Environs. (ApproTEC, P.O.Box 10973, Nairobi. December 1993).

2 Botswana Technology Centre (BTC).: BTC Experimental Staff Housing Project Design and Construction Assessment Report. (Botswana Technology Centre, Architectural and Civil Engineering Units, Botswana, April 1995).

3 British Standards Institution: Methods of Testing Soils for Civil Engineering Purposes. doc. BS1377 (London, BSI, 1991).

4 British Standard Institution: Methods of Test for Stabilised Soil. doc. BS1924 (London, BSI, 1975).

5 Building Research Establishment.: Building in Hot Climates, A Selection of Overseas Building Notes. (London, H.M.S.O., 1980).

6 CRATerre.: Earth Building Materials and Techniques, Select Bibliography. (Eschborn, Federal Republic of Germany, German Appropriate Technology Exchange, 1991).

7 Doat, P. et al.: Construire en Terre. (Paris, CRATerre, 1979).

8 Enyatseng, G.R.: Research into the Application of the Ranko Block Machine. (Botswana Technology Centre, Gaborone, Botswana, Sep 1987).

9 Fitzmaurice, R.: Manual on Stabilised Soil Construction for Housing. (New York, United Nations, 1958).

10 Gooding, D.E.M.:Soil Testing for Soil-Cement Block Production - Working Paper No.38. (Coventry, Development Technology Unit, University of Warwick, 1992).

11 Gooding, D.E.M.:Quasi-Static Compression Forming of Stabilised Soil-Cement Building Blocks - Working Paper No.40. (Coventry, Development Technology Unit, University of Warwick, 1993).

12 Groth, A.: The Use of Impact Blows to Compact Soil-Cement Blocks. (Third-year undergraduate engineering project conducted at the University of Warwick, 1984).

13 Groth, A.: The Ranko Soil-cement Block Machine, Design, Manufacture and Testing of Prototype. (Motheo (Pty) Ltd, P.O.Box 2224, Gaborone, Botswana, 1984).

14 Groth, A.: CORDE Shelter Study, Final Report. (CORDE, Co-operation. Development and Education, P.O.Box 1895, Gaborone, Botswana, March 1994).

15 Gurusamy-Naidu, K.: Housing in Botswana, A Case for Soil Block Construction. (Botswana Technology Centre, Gaborone, Botswana, 1993).

16 Handbook For Building Homes of Earth. Department of Housing and Urban Development, Office of International Affairs, Washington, D.C. 20410 (undated).

17 Hawkins, A.M.: Soil Stabilised with Cement. (Third-year undergraduate engineering project conducted at the University of Warwick, 1988).

18 Haavaldsen, T.: Plaster for Soil and Soil-cement Walls. Basic requirements, materials and test results. (Building Research Unit, Ministry of Lands, Housing and Urban Development, Dar Es Salaam, May 1981).

19 Head, K.H.: Manual of Soil Laboratory Testing. (London, Pentech Press, 1980).

20 Ingles, O.G. & Metcalf, J.B.: Soil Stabilization Principles and Practice. (London, Butterworths, 1972).

21 Hoek-Smit, M.C.: Housing Demand and Preferences Study for Urban Areas. (Ministry of Local Government and Lands, Botswana, 1989).

22 International Labour Office.: Small-scale Manufacture of Stabilised Soil Blocks. (Geneva, Switzerland, Technology Series, Technical Memorandum No.12, ILO Publications 1987).

23 Intermediate Technology Development Group (Zimbabwe).: How to Make Stabilised Soil Blocks - Technical Advice Notes Number 2. (Harare, ITDG, undated).

24 Lawson, W.: Materials for Low-Cost Building in North East Nigeria - Working Paper No.35. (Coventry, Development Technology Unit, University of Warwick, 1991).
25 Lunt, M.G.: Stabilised Soil Blocks for Building in Building Research Establishment.: Building in hot climates, A Selection of Overseas Building Notes. (London, H.M.S.O., 1980).
26 Mason, K.: Sedimentation Jar and Linear Shrinkage Mould Tests - Suffix to Interim Report. (Report outlining results of Dr Peter Walker at the University of Zimbabwe, ITDG Zimbabwe, 1992).
27 Moriarty, J.P. & Therkildsen O.: Lateritic Soil-cement as a Building Material. (National Housing and Building Research Unit, Housing Division, Ministry of Lands and Urban Development, Dar Es Salaam. 1973).
28 Moriarty, J.P.: Earth Wall Construction in Tanzania. (Building Research Unit, Ministry of Lands, Housing and Urban Development, Dar Es Salaam, March 1976).
29 Mukerji, K.:Soil Block Presses. (Eschborn, Federal Republic of Germany, German Appropriate Technology Exchange, 1988).
30 Mukerji, K & Worner, H.: Soil Preparation Equipment. (Eschborn, Federal Republic of Germany, German Appropriate Technology Exchange, 1991).
31 Mwafongo, F.G.: Alternative Building Materials, Iringa Demonstration House. (Building Research Unit, Prime Ministers Office, P.O.Box 1964, Dar Es Salaam, 1984).
32 NBRI.: Low-cost Housing. (NBRI, National Building Research Institute, Council for Scientific and Industrial Research, Pretoria, South Africa, 1987).
33 Norton, J.: Building With Earth: A Handbook. (London, Intermediate Technology Publications Ltd, 1986).
34 Ola, S.A. & Mbata, A.: Durability of Soil-cement for Building Purposes - Rain Erosion Resistance Test. (Butterworth-Heinemann Ltd, Construction & Building Materials Vol. 4 No.4, Dec 1990).
35 Ophoven, P.: Compressive Strength of Walls built up of Mudblocks and Soil-cement Blocks. (Building Research Unit, Ministry of Lands, Housing and Urban Development, Dar Es Salaam, Jan 1977).
36 Parsons, A.W.: Compaction of Soils and Granular Materials: A Review of the Research Performed at the Transport Research Laboratory. (London, HMSO, 1992).
37 Perera, A.: Landcrete Blocks (Cement Stabilized Laterite Soil Blocks). (Department of Civil Engineering, University of Moratuwa, Sri Lanka, 1993).
38 Perera, A.: Low Cost House Using Cement Stabilised Blocks and Timber Poles. (Department of Civil Engineering, University of Moratuwa, Sri Lanka, 1993).
39 Petersen, J.: Experiments with Sample Testing Procedures for Stabilised Soil Blocks. (Rural Industries Innovation Centre, Kanye, Botswana, 1991).
40 Petersen, J.: Material Options & Cost Computations for Building Elements in Botswana. (Rural Industries Innovation Centre, Kanye, Botswana, 1992).
41 Spence, R.J.S. & Cook,D.J.: Building Materials in Developing Countries. (Chichester, Wiley, 1983).
42 Spence, R.J.S.: Predicting the Performance of Soil-cement as a Building Material in Tropical Countries. (Building Science, **10**, 1975)
43 Stulz, R.: Appropriate Building Materials. (London, U.K / SKAT. St.Gallen, Switzerland, Intermediate Technology Publications Ltd, 1983).
44 Sutcliff, J.: Quality Assurance of Stabilised Soil-Cement Blocks. (Third-year undergraduate engineering project conducted at the University of Warwick, 1994).
45 United Nations.: Soil-Cement its Use in Building. (United Nations Publication, Sales No.: 64.IV.6, 1964).
46 United Nations Centre for Human Settlements (UNCHS, Habitat).: Development of National Technological Capacity for Environmentally Sound Construction. (UNCHS, Habitat, Nairobi, Kenya, 1993).
47 United Nations Centre for Human Settlements (UNCHS, Habitat).: Report of the Workshop on the Network of African Countries on Local Building Materials and Technologies. (UNCHS, Habitat, Nairobi, Kenya, 1994).
48 Volunteers in Technical Assistance: Making Building Blocks with the Cinva-Ram Block Press. (Mt. Rainier, Maryland, VITA, 1977).
49 Walker, P.: STABILISED SOIL BLOCKS, The influence of Constituent Materials on Physical Properties. (Department of Civil Engineering, University of Zimbabwe, Mt Pleasant, Harare, Zimbabwe. Draft report September 1994)
50 Webb, D.J.T.: Stabilised Soil Building Blocks. (University of Newcastle-upon-Tyne, Ph.D. Thesis, 1988).
51 Williams, F.H.P. & Maclean, D.J.: The Compaction of Soil. A Study of the Performance of Plant. Transport & Road Research Technical Paper No. 17. (London, H.M.S.O., 1950).

DEVELOPMENT
TECHNOLOGY
UNIT

D
T
U

Working Paper No. 45

Preliminary Study of Rainwater Harvesting in Mid-West Uganda

November 1995

T.H. Thomas

Development Technology Unit
Department of Engineering University of Warwick
Coventry CV4 7AL UK
Tel: +44 (0) 203 523122 Fax: (0)203 418922
email: dtu@eng.warwick.ac.uk

PRELIMINARY STUDY OF RAINWATER HARVESTING IN MID-WEST UGANDA

Results of a study undertaken by Dr Terry Thomas in collaboration with Uganda Rural Development Training Programme (URDT) in November 1995.

Introduction

Domestic rainwater harvesting is the collecting of run-off during rains from impermeable surfaces on houses or close to houses, its storage in waterproof vessels and its subsequent use as the water supply of the inhabitants of these houses. The use may be "temporary" (for example during the 24 hours following a rainstorm), "seasonal" (throughout the rainy season) or "permanent" (throughout the year except perhaps in years of exceptionally low rainfall). These three levels of use require in Western Uganda, storage capacities per household of around 80 litres, 1200 litres and 10000 litres respectively. In some places there are opportunities for collecting water from institutional roofs and the roofs of commercial buildings. As commercial buildings in the Target Areas are mostly small shops that also serve as dwellings, they may be regarded as included within 'domestic' rainwater sources.

The most suitable run-off surface in Western Uganda is the corrugated iron roof. Grass roofs are mostly of such loose construction and small size that neither the quality nor quantity of run-off from them is suitable. Grass roofs are also difficult to gutter. Tile roofs are rare in the Area; asbestos roofs are absent. Bare rock surfaces are rarely close to habitations, other ground surfaces give very contaminated and silt-laden run-off.

At present, over the whole area, the majority of the population is using water from contaminated and inconvenient sources, most commonly from waterholes in valleys below their houses. "Temporary" rainwater harvesting, as defined above, is widely practised: most households put out plastic or pottery containers under their roof during rains. However it is rare to find more than 50 litres of such collecting capacity per household. A few households have gutters and storage adequate for "seasonal" water supply, the storage is most commonly a cement jar of 1 to 2 cubic meters capacity. Larger tanks sized for "permanent" supply are rare; there are a few in Kabarole District for example next to dwellings on tea estates, and a few in Fort Portal and Hoima towns. These large tanks are commonly made of galvanised iron.

Roofs and Gutters

Because of the high rainfall, iron roofs are common in Western Uganda and their use is increasing following the national development in recent years. The Uganda National Integrated Household survey of 1992-3 indicated only 31% of households with iron roofs, however it is believed the current figure for Western Uganda is considerably higher.

A survey was undertaken in November 1995 within the 3 Districts. It is clear that the occurrence of iron roofs is strongly influence by type of settlement.

In the two towns of Fort Portal and Hoima over 90% of buildings have iron roofs (auxiliary buildings like kitchens have not been counted).

Along roads of sufficient important to carry taxi/minibus services over 90% of buildings in trading posts have iron roofs and about 70% of isolated buildings close to those roads have them.

Scattered homesteads away from roads have a lower incidence of iron roofs. A number of villages were surveyed, the villages were selected as being at least 2km from significant roads. Very isolated villages were not surveyed but are believed to have very few iron roofs.

Table **Survey of Roofing in Mid West Uganda**

District	Location	Total homes	Iron roofs		% part gutters
			No	%	
Kabarole	Village near Bigodi	40	20	50	0
	Farms in Hakibale county	110	76	70	0
	4 villages (S of Kyenjojo, Mwenge Co.)	454	181	49	30
Kibaale	3 villages near Kagadi (Buyaga Co.)	195	117	60	10
	4 villages in Bwikara sub-county (Buyaga Co.)	394	236	60	50
	2 villages near Rabona (Buyaga Co.)	55	20	45	0
	Migrant settlement near Narweyo	80	5	<10	0
Hoima	3 villages E. of Hoima	220	145	66	6
Total		1548	800	52	

From this preliminary survey and assuming 25% of the population lives in towns or trading posts, the likely availability of iron roofs is 60% (computed as 0.25 x 90% + 0.75 x 50%).

Full gutters are very rare and even partial guttering is uncommon in the Target Area; gutters are only installed for rainwater harvesting purposes. The normal roof overhang is intended to reduce soil erosion or splash damage due to unguttered run-off but this is often ineffective. Trading posts in particular suffer from significant erosion due to roofwater run-off.

Gutters, where present, are usually of corrugated or folded galvanised iron suspended from the roof sheets themselves or from rafter ends. As eaves are low, guttering is often stabilised by props from the ground, for example 2 meter wooden poles. Plastic guttering supported by special brackets from barge boards (as is the European norm) is absent in Western Uganda. This may change with the current expansion of PVC product manufacture in Uganda.

Rainfall, Water Consumption and Storage Requirement

With the exception of a narrow rain shadow (generally under 2 km wide) along the shores of Lake Albert, the whole Target Area has a bimodal pattern of precipitation generally exceeding 1000 mm a year. Over 1200 mm a year reaches much of the Area and in places annual precipitation exceeds 2000 mm.

Taking 40 sq.m. as a minimum roof size, full guttering, 1000 mm rainfall, 85% rainwater capture and 8 persons per household gives an average daily capture of 12 litres per person. This is half the WHO recommended figure of 24 litres per person per day but about equal to current daily consumption of 5 jerrycans per large family.

Using the more typical 1200 mm rainfall and 50 sq.m. gives 18 liters per person per day. This is a considerable increase on current norms and on the levels recorded in East Africa long ago by Bradley and White for communities without piped water.

A recent study by Rugumayo of the Directorate of Water Development (Rugumayo AI, "Rainwater Harvesting in Uganda", a paper presented to the conference on *Sustainability of Water & Sanitation Systems*, Kampala, September 1995, proceedings to be published by WEDC, Loughborough, U.K.) showed a storage requirement of about 22% of annual consumption in bimodal rainfall areas of Uganda. This figure would be slightly reduced if the roof area were oversized for the consumption or if dry season consumption were lower than the annual average. The figure also varies with the level of drought-year security adopted; however it is generally uneconomic to provide sufficient storage to carry water over from wet years to dry ones.

On Rugumayo's evidence, storage for a family of 8 consuming 18 liters per capita per day would need to be around

$$8 \times 18 \times 365 \times .22 = 11{,}500 \text{ liters} = 11.5 \text{ cubic meters.}$$

It is expected that more detailed studies will indicate that 8 to 10 cubic meters is sufficient for most households.

The present cost of 10 cubic meters storage is too high for most households to afford. 1994 prices, collected by RUWASA for Eastern Uganda, indicated the cost of 10 cubic meters storage + fittings to be about

$ 550	for concrete block tanks (with no cover?)
$ 400	for corrugated iron tanks
$2000	for HDPE plastic tanks.

Ferrocement tanks, cheaper than concrete block tanks in Kenya, are probably of similar cost to concrete block tanks in Uganda due to the very high price of steel mesh here. Concrete jars, as currently made in Western Uganda, would cost about $900 to make up 10 cubic meters.

For comparison iron sheets for a typical roof cost $200 to $300.

It is an informed assumption of this proposal that there is scope for substantially reducing water storage costs in Western Uganda by taking advantage of local factors such as stable soil, low water table, cheap bricks etc. Storage costs dominate rainwater harvesting costs and are therefore critically affect economic viability. An 8 cubic meter domed underground tank has just (December 1995) been built at the URDT Institute, Kagadi and a second is being excavated. Their materials cost is under $100 each: their performance will be tested,

Water Supply Alternatives in Mid-Western Uganda

Excluding the two towns with pumped piped water (although the Fort Portal system is currently barely operational) the viable forms of supply in the Area are, in order of current usage:

(i) waterholes, swamps and streams, not protected from contamination,
(ii) shallow wells in valley bottoms made hygienic with the aid of handpumps,
(iii) protected springs,
(iv) man-made reservoirs, with or without in-situ sand filtering of water,
(v) boreholes,
(vi) rainwater harvesting.

(i) Unprotected surface sources give low quality water very cheaply at locations often far from homesteads and nearly always below them: water is carried uphill from them. There are possibilities of partially improving quality (fencing, filtering, pollution control) and convenience (carrying aids).

(ii) Hygienic shallow wells give good quality water at modest cost (e.g. $100 per household) are therefore the basis of most current water improvement activities in the Target Area. Near trading posts the water extraction rate is often inadequate (due to low pump or aquifer flowrates) and long queues form. Location is rarely convenient and effort in water carriage is typically 100 minutes per household per day in addition to queuing time.

(iii) The high permeability of soil in the region results in generally low water-tables and low-level emergence of springs. Well defined spring "eyes" are relatively rare. Springs are normally lower than dwellings, thereby precluding gravity-fed reticulation. Spring protection is currently undertaken in Kibaale and Hoima districts, but good springs are not common enough to offer most communities water within 500 meters (a distance corresponding to the 100 minutes per day carriage time cited above).

(iv) Man-made tanks (i.e. reservoirs behind low dams) are applicable in the drier North East of the Target Area. They may be regarded as a variant on shallow wells, in that a shallow well with pump alongside a tank is the preferred method of hygienic water extraction. Their cost, coupled with the difficulty of minimising reservoir leakage in permeable ground, limits their application to areas where other methods do not work.

(v) The most difficult parts of the Target Area for water supply are those without the topology that permits shallow well construction. A number of boreholes have been expensively drilled in the Target Area. Most have proved difficult to maintain by their user communities and therefore failed during the collapse of centralised maintenance services in the 1970s. Modern designs and procedures (eg. VLOM) solve some of the problems and a few boreholes are being rehabilitated. For scattered settlements it will always be costly to achieve convenient (i.e. very local) supply via boreholes. Moreover deep ground water in the Target Area is usually sufficiently high in iron content that treatment to remove it is desirable.

(vi) Rainwater harvesting is therefore to be compared with the five alternatives above. Its performance is not yet fully known. Storm intensity (that affects gutter sizing), variability of precipitation and dry season length (which affect storage and roof area requirements) and other factors need more detailed local measurement.

Rainwater harvesting has the following advantages in the Area:

- gives very convenient supply, no walking required;
- is largely independent of outside organisation for construction and maintenance;
- gives fairly high water quality which may be further increased by simple means;
- reduces the area of roofing iron required, because of less overhang;
- reduces run-off erosion, especially in townships and urban areas.

It has the disadvantages of:

- rather high cost per household unless cheaper storage methods can be found;
- discrimination against the poor who have grass roofs and insufficient funds;
- storage structures being vulnerable to earth tremors to which the Area is subject;
- vulnerability to drought years.

Further Development of Rainwater Harvesting

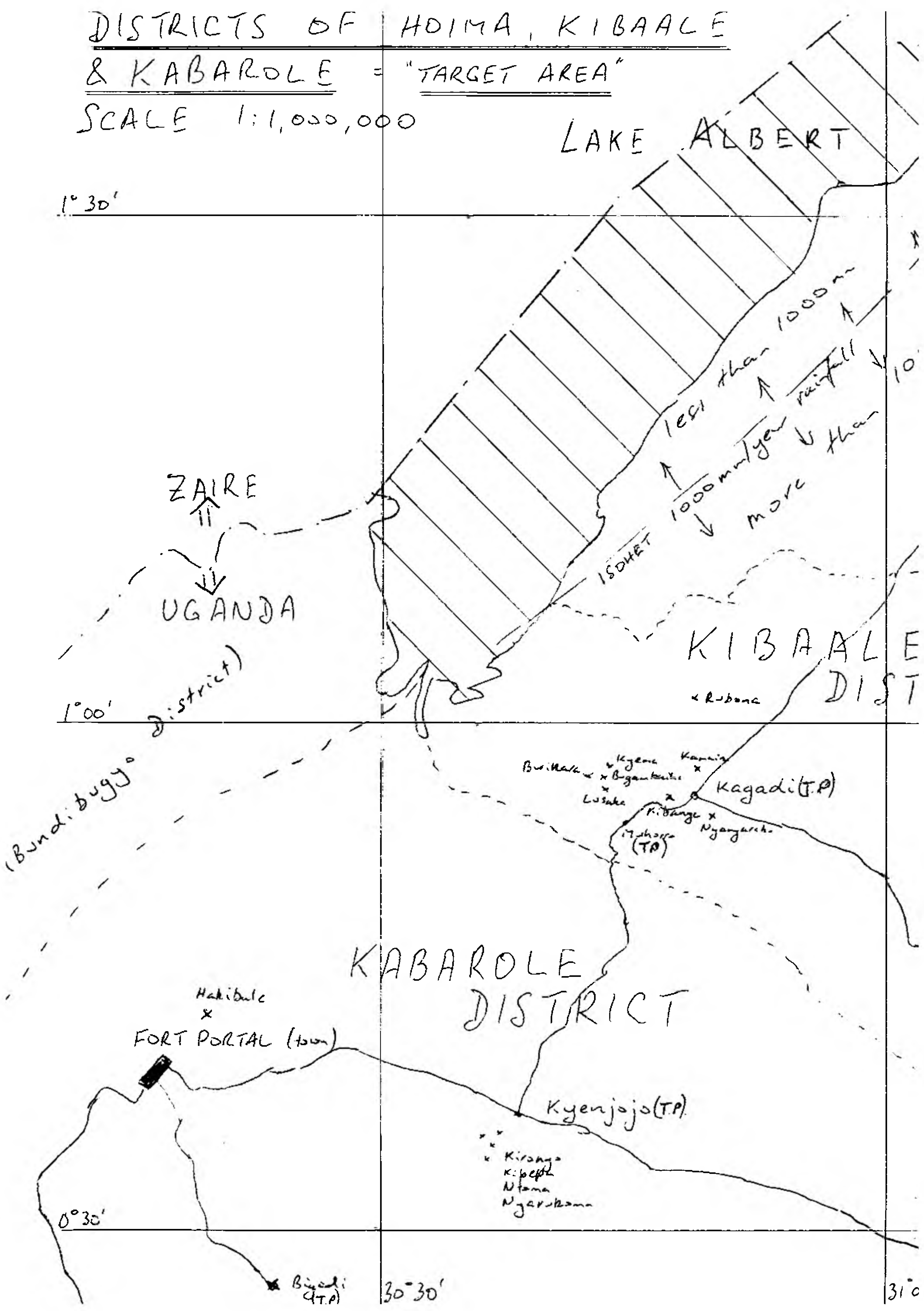
Because a strong *prima facie case* can be made for rainwater harvesting in the Target Area yet current usage of the technique is very slight, there appears to be scope for

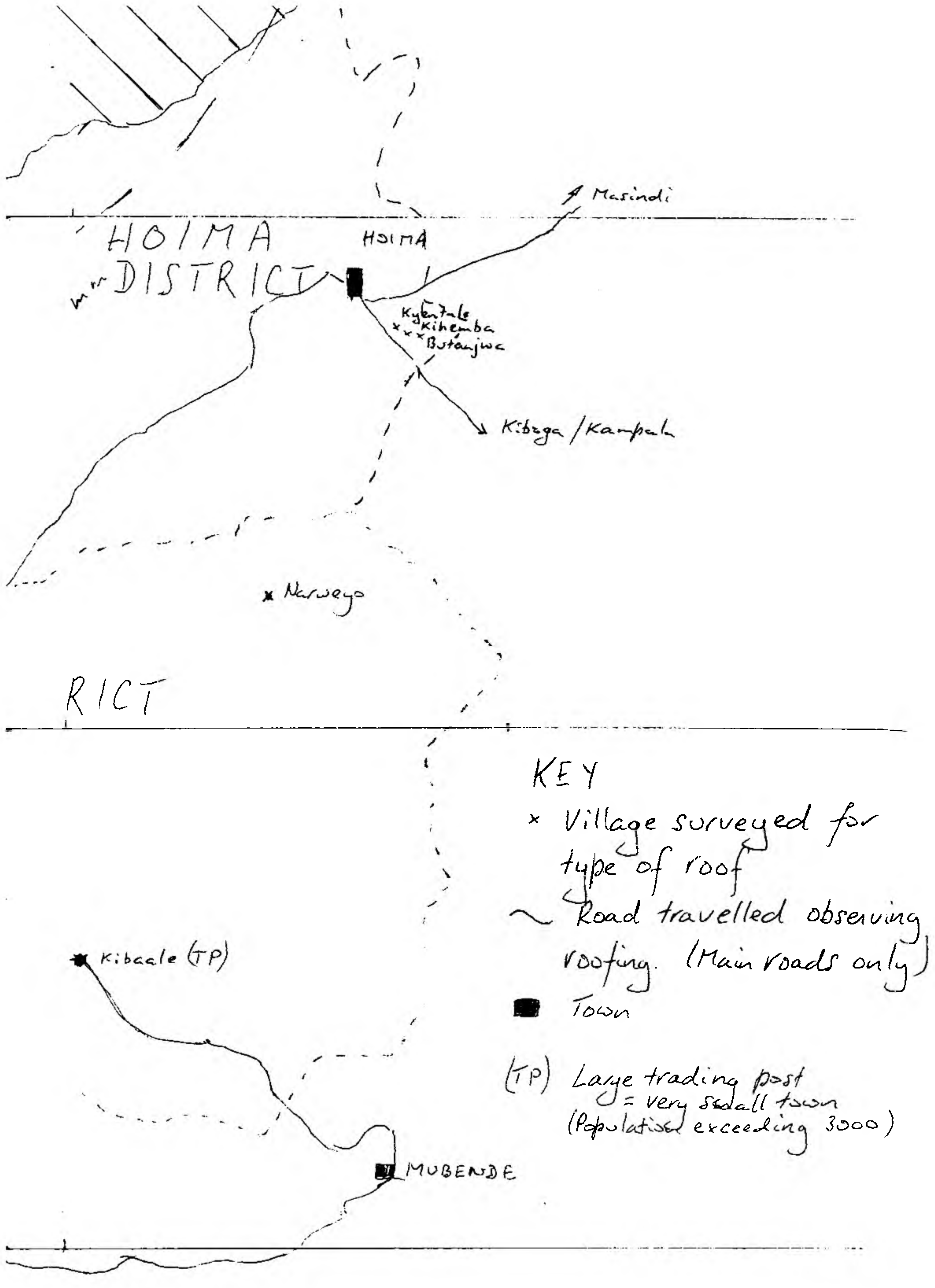
(i) a programme of design development to reduce the cost of components of rainwater systems - in particular water storage tanks - and to optimize their sizing.

(ii) field studies to identify why there is such an apparent mismatch between potential and actual usage of the technique.

Map

The map overleaf shows, Mid Western Uganda, which is made up of three districts: Hoima, Kibale and Kabarole and the location of the villages and roads surveyed for the presence of iron roofing. The total Target Area is about 15,000 sq'km. However other districts of Uganda, besides the three studied here, have rainfall and soil conditions that seem to favour rainwater harvesting. Much of Kiboga, Mubende, Mpigi, Masaka, Bushenyi, Kasese, Rukungiri, Kisoro and Kabale Districts appear suitable, as do parts of Mbarara, Masindi, Luweso, Ntungamo and Mukono Districts. Outside Uganda, large areas of Eastern Zaire and parts of Kagera Province in Tanzania show similar characteristics.


DISTRICTS OF HOIMA, KIBAALE
& KABAROLE = "TARGET AREA"
SCALE 1:1,000,000
LAKE ALBERT
1° 30'
less than 1000mm
1000mm/year rainfall
more than 10
ISOHET
ZAIRE
UGANDA
(Bundibugyo District)
KIBAALE DIST
Rubona
1°00'
Buikalu
Kyema
Bugambuku
Kamain
Lusaka
Kagadi(T.P)
Kibange
Nyamyarcho
Muhorro (TP)
KABAROLE DISTRICT
Hakibale
FORT PORTAL (town)
Kyenjojo(T.P)
Kirongo
Kipepo
Ntoma
Nyarukoma
0°30'
Bigodi (T.P)
30°30'
31°


Masindi
HOIMA
DISTRICT
HOIMA
Kyentale
Kihemba
Butanjwa
Kibaga / Kampala
Narweyo
RICT
KEY
× Village surveyed for type of roof
~ Road travelled observing roofing. (Main roads only)
Town
(TP) Large trading post = very small town (Population exceeding 3000)
Kibaale (TP)
MUBENDE

DEVELOPMENT
TECHNOLOGY
UNIT

D
T
U

Working Paper No. 46

Operating Strategies for Hydropower Systems Using Unregulated Turbines

January 1996

C.Alatorre Frenk & T.H. Thomas

Development Technology Unit
Department of Engineering University of Warwick
Coventry CV4 7AL UK
Tel: +44 (0) 203 523122 Fax: (0)203 418922
email: dtu@eng.warwick.ac.uk

Operating Strategies for Hydropower Systems Using Unregulated Turbines

Dr. Claudio Alatorre Frenk and Dr. Terry Thomas***

1. Introduction

For micro hydropower systems, say under 100 kW, there has been a growing interest in using turbines having no hydraulic controls. The financial savings from omitting control gear is substantial and further savings are possible if 'pumps', mass-produced in country, are used instead of individually designed turbines, often imported. There is now a considerable literature on the use and selection of pumps-as-turbines.

A common configuration of micro-hydro plant is for there to be very little water storage and therefore for the system gross head to be nearly constant. The speed of the turbine-generator set is held constant by electrical means. Thus the fixed geometry turbines run at constant flow. Unfortunately the river flow is never constant. When it exceeds the total turbine flow there is no problem, but when it is less than rated turbine flow there is a mismatch. The absence of storage (which is too costly) removes the possibility of filtering out river flow fluctuations.

There are three system/operation designs we might use in this context. The simplest way of operating (SO) is to employ a single turbine that runs only when river flow exceeds turbine rated flow. A second option is to employ several small turbines operating in parallel (PO): the number in use is varied to match the variation in river flow. A third option is to intermittently operate (IO) a single turbine fed from a small reservoir (*e.g.* holding only 15 minutes flow). It is the purpose of this paper to compare these three alternatives and to show that the third (IO) has apparent economic advantages over the others. All three alternatives can give higher economic returns in many semi-industrialised countries where, power for power, turbines cost over four times more than pumps.

In making economic comparisons there are a great many system variables and cost factors we might accommodate. To reduce the complexity of the analysis we will restrict ourselves to modelling hydro systems connected to a 'large' electrical grid. This allows us to reasonably assume that all the electrical energy produced will be purchased and that each unit will command the same (daily average) price. Even with this simplification, however, it has been necessary to develop a more flexible economic methodology than is normally used for evaluating hydropower. This methodology, described in section 5, is we hope of value in its own right independent of its specific application here.

The ensuing analysis is a simplified form of that developed in a PhD thesis from Warwick University, U.K. (Ref. 1).

* Consultant engineer. Ap. Post 259, 91001 Xalapa, Ver. MEXICO. Fax. (52 28) 145134. E-mail: caf@speedy.coacade.uv.mx

** Director, Development Technology Unit, University of Warwick, Coventry CV4 7AL, UK. Fax (44 1203) 418922. E-mail: estht@eagle.warwick.ac.uk

2. Option SO: Simple Operation of a Single Turbine

As observed above, at fixed head and speed an unregulated turbine will draw a constant rated flow, Q_R. Should this flow not be available ($Q_A < Q_R$), there are two operating alternatives. One is to shut down the turbine. The other is to let the water level in the penstock draw down until flow equilibrium is reached ($Q_A = Q_T$, $Q_T < Q_R$): reducing the effective gross head in this way will reduce the turbine flow. Although the second alternative is inefficient, since some head is being wasted and the turbine is being operated away from its best efficiency point (BEP), some power is better than none. Figure 1 shows the advantage of leaving the turbine running at low flow: typically an extra 10% energy can be obtained per year by doing so. In the subsequent analysis we assume alternative (ii) is followed. In practice, to avoid certain operational problems it is sometimes desirable to turn off the turbine flow when Q_A falls below say 60% of Q_R (which corresponds to very little power).

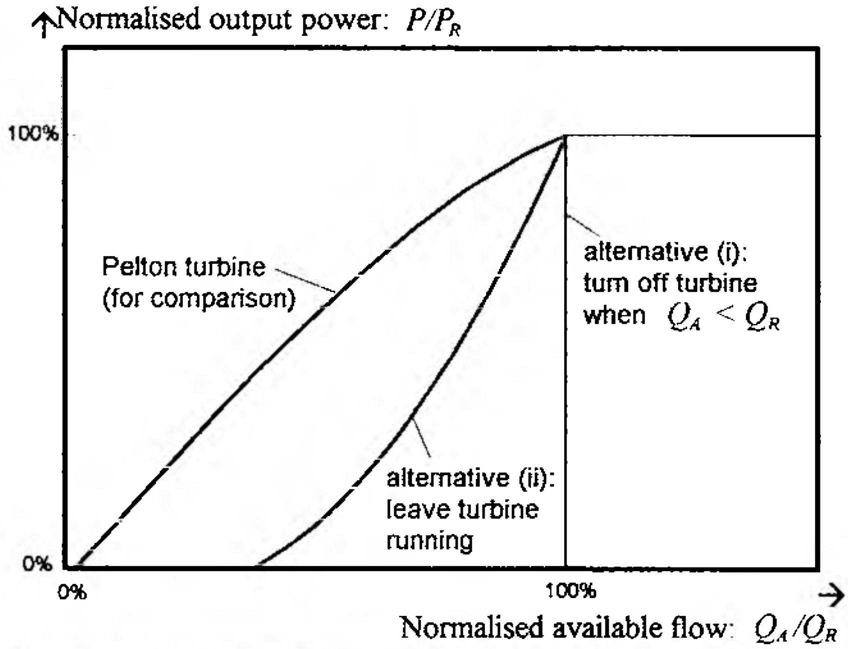


Figure 1: Advantages of operating a PAT with part-full penstock

In the design of this option the key variable is the size of the turbine and hence its rated flow Q_R. Increasing Q_R will increase the capital cost of the system. Up to some limit it will also increase the energy output. Using the economic model described later, by trial and error an optimum Q_R can be identified. It is such an optimised SO system we will later compare with optimised PO and IO systems.

3. Option PO: Several Turbines Operated in Parallel

If two or more PATs are operated in parallel, they can be switched on and off according to the available flow. Parallel operation (PO) of up to 7 machines has proven to be more cost-effective than a single conventional hydraulic turbine of comparable capacity, according to the literature (Ref. 2).

The machines can be different or identical to each other (for example, in a two-PAT scheme, the turbines can handle either ⅓ and ⅔ of the full flow, or one half each). The first option increases the energy generation, as it enables more combinations (in this case ⅓, ⅔ and ³⁄₃); the latter restricts the combinations (½ and ²⁄₂), but makes maintenance easier, as the same set of spares can be used in all machines.

Figure 2 shows the power output of a 5-turbine system. The solid line indicates some of the available flow is being spilled; the dashed line indicates where the penstock is not full; the numbers show how many machines are actively connected to the penstock. The shared penstock and equality of turbine sizes are common features of PO. The optimum number of machines is not however always 5 but depends, *inter alia*, on the variability of the river flow. In optimising a PO system we therefore need to find best values for two parameters: turbine number (n) and total rated flow Q_R.

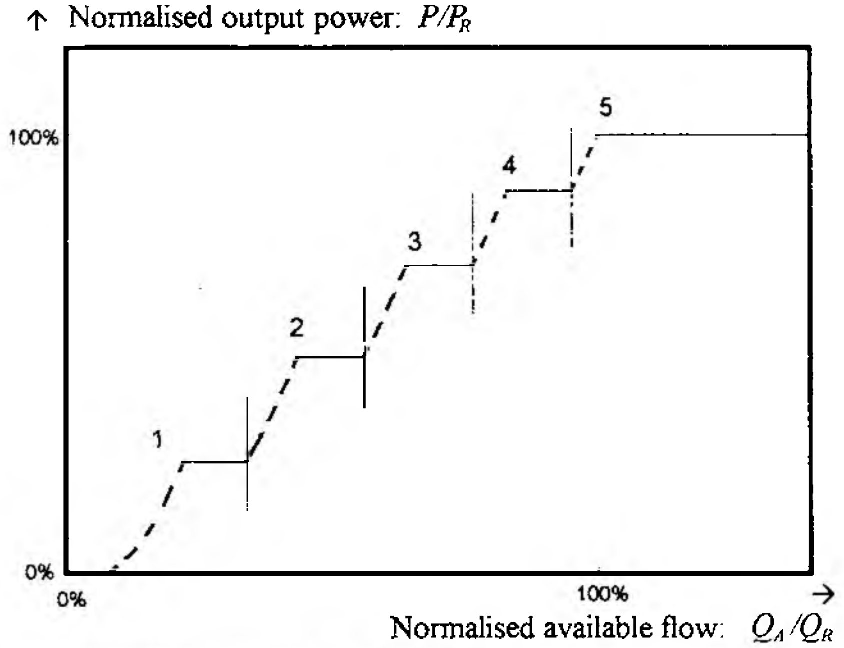


Figure 2. Parallel operation of five equal machines with common penstock.

4. Option IO: Intermittent Operation of One Turbine

When the river flow Q_A is less than the flow Q_R drawn by the turbine in operation, we might operate the turbine intermittently for a fraction Q_A/Q_R of the time. We will need a reservoir (in practice an enlarged forebay tank) whose level falls while the turbine is running and rises when the turbine is shut off. Figure 3 shows the cyclic operation. Although in theory the penstock could be opened and closed using a valve, in practice a quick-priming double siphon (see Fig. 4) would be used. Such siphons have no moving parts and can operate very swiftly.

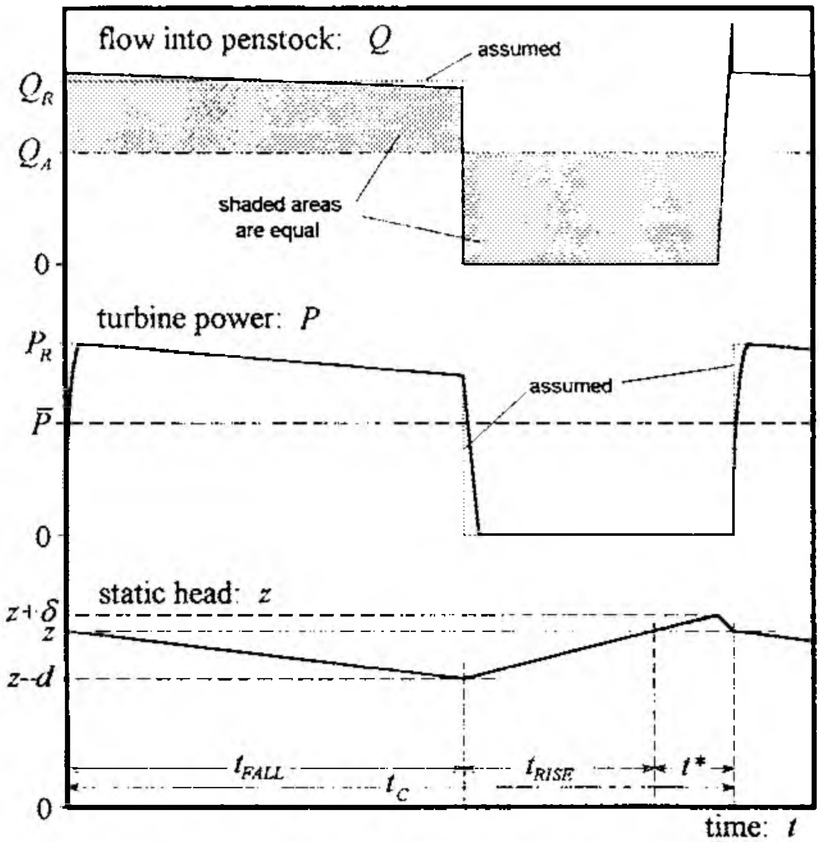


Figure 3. Schematic representation of the intermittent operation using a siphon (with d, δ, etc. exaggerated).
(t^* is the time to surcharge the reservoir with the water needed to refill the penstock.)

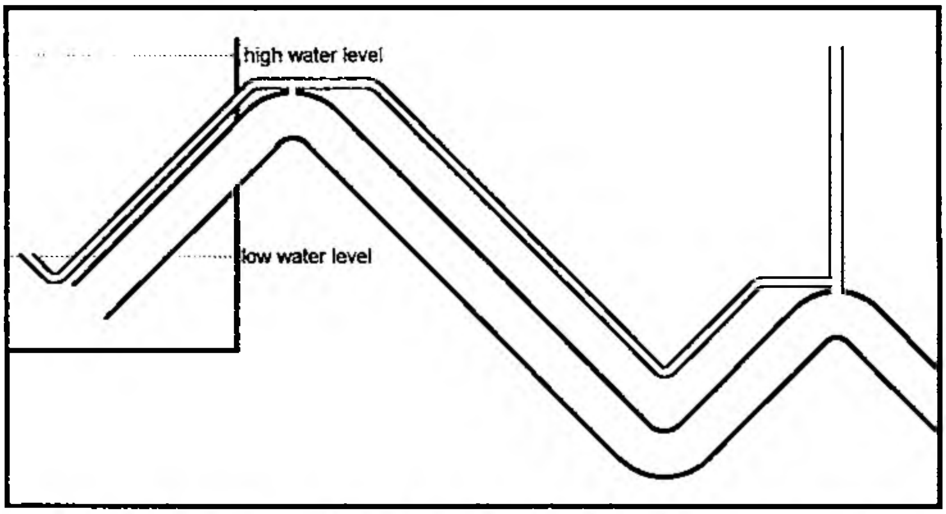


Figure 4. Schematic layout of a double-siphon, developed from a design described in Ref. 3. Performance and design procedure analysed in Ref. 1.

Technically therefore intermittent operation requires us to design an appropriate siphon and a penstock that will tolerate the sudden and frequent changes in flow. Using economic models we need to optimise three variables, namely the turbine size (Q_R), the drawdown (d) of the forebay tank and its effective surface area (A).

There are three regimes under which the system can operate. Where Q_A is greater than Q_R (regime ①) the turbine will run continuously at full power P_R and the tank will be both full and overflowing. Where Q_A is much less than Q_R (regime ②) the siphon will operate cyclically, the power output will be intermittent and the water level in the tank will fluctuate between height $z+\delta$ (rated gross head plus the small surcharge height δ) and height $z-d$ (see Fig. 3). If however Q_A is only slightly less than Q_R, we may observe regime ③ in which the turbine runs continuously but at a reduced head and flow: the water level will be steady somewhere between heights z and $z-d$.

This third regime is unfortunate as it may give less output power than when the flow is slightly lower (regime ②). Fortunately, with typical small values for drawdown ($d < .05 \cdot z$), the system spends very little time operating in regime ③ and to simplify the discussion we can neglect it. Figure 5 shows the power output of a typical system.

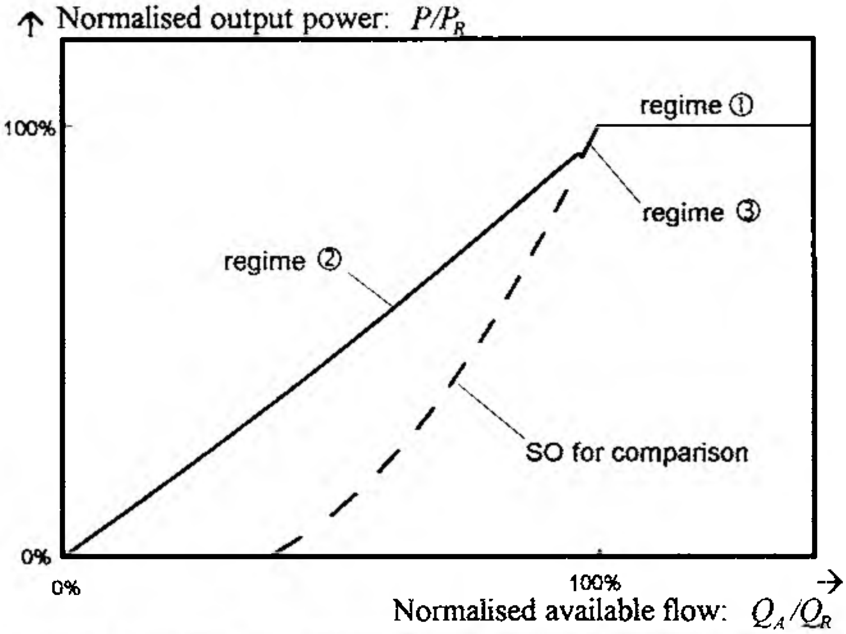


Figure 5. Power *versus* flow for intermittent operation.

The drawdown height (d) and the corresponding drawdown volume ($A \cdot d$) affect the power output during regime ②. A full analysis is given in Ref. 1 and is complex. Fortunately the optimum value of d is such a small fraction of gross head that we can use an approximate analysis in which

- Q is assumed to remain constant at Q_R as the tank level draws down
- the mean gross head during turbine operation is not z but $z-d/2 = z(1-d/2z)$
- the ideal cycle time $t_C = t_{FALL}+t_{RISE}$ is extended an extra period t^* during which the tank overfills by volume V^* which is effectively lost each cycle. As an approximation we can equate V^* to the penstock volume.

The effect of the last assumption is that the cycle time t_C is increased by factor X and the mean power correspondingly decreased.

$$X = \frac{t_{FALL} + t_{RISE} + t^*}{t_{FALL} + t_{RISE}} = \frac{\dfrac{dA}{Q_R - Q_A} + \dfrac{dA}{Q_R} + \dfrac{V^*}{Q_R}}{\dfrac{dA}{Q_R - Q_A} + \dfrac{dA}{Q_R}}$$

$$X = 1 + \frac{V^*}{dA}\left(1 - \frac{Q_A}{Q_R}\right) \qquad [1]$$

So mean power is

$$\overline{P} = \frac{P_R\left(1 - \dfrac{d}{2z}\right)}{1 + \dfrac{V^*}{dA}\left(1 - \dfrac{Q_A}{Q_R}\right)} = P_R \frac{1 - \alpha d}{1 + \dfrac{\beta}{d}} \qquad [2]$$

We may decide to choose d to simply maximise power regardless of its influence on cost. If so, differentiating the function of d in Eq. [2] and setting to zero gives

$$d_{\text{MAX.POW}} = \sqrt{\beta^2 + \frac{\beta}{\alpha}} - \beta$$

$$\frac{d_{\text{MAX.POW}}}{z} = 2\sqrt{\alpha^2\beta^2 + \alpha\beta} - 2\alpha\beta$$

then, since $\alpha \cdot \beta \ll 1$ typically,

$$\frac{d_{\text{MAX.POW}}}{z} \approx 2\sqrt{\alpha\beta}$$

$$d_{\text{MAX.POW}} \approx \sqrt{\frac{2zV^*}{A}\left(1 - \frac{Q_A}{Q_R}\right)} \qquad [3]$$

This power-maximising drawdown varies with flow from 0 at ($Q_A = Q_R$) to $\sqrt{\dfrac{2zV^*}{A}}$ at no flow. Although a variable drawdown siphon is feasible, it will normally suffice to use a fixed drawdown for all flows, choosing a flow of *e.g.* $Q_A = 0.8 \cdot Q_R$ to get close to the flow-averaged optimum. Alternatively we can employ a 'hill-climbing' search, using a spreadsheet version of the full economic model, to optimise all three variables d, A and Q_R.

5. Economic Modelling

There are many economic measures we might use for comparing alternative hydro system designs for a particular site. They all combine the initial cost, any running costs and the income earned through the life of the system into a single measure. The most usual measures are internal rate of return (IRR), benefit-cost ratio (BCR), payback time (PB) and net present value (NPV). IRR, BCR and PB reflect the financial return per unit of money invested, whereas NPV reflects the return "per site"; a system sized to maximise the latter will be larger than one sized to maximise the former three. As hydropower systems are capital intensive and microhydro schemes are particularly vulnerable to capital shortage,. we choose the return-per-unit-of-money-invested approach, *i.e.* the former three measures. Moreover, as we are dealing with systems whose expected incomes per year remain constant over their lives, and whose lives are long, it can be shown that the economic ranking of alternative designs will be the same whichever of the three measures (IRR, BCR or PB) we choose. As the calculation of BCR and PV (and NPV) requires the prior choice of a discount rate, we prefer the IRR as our measure and the ratio of their respective IRRs as our criterion for comparing two alternatives. IRR is the solution of:

$$\frac{\mathrm{IRR}(1+\mathrm{IRR})^{N}}{(1+\mathrm{IRR})^{N}-1}=\frac{\mathrm{NAI}}{\mathrm{CAP}} \qquad [4]$$

where N is project life, NAI is annual income net of running costs and CAP is capital invested. As we are normally interested in IRR values greater than 15% and system lives of over 25 years, we can approximate Eq. [4] within 3% error to:

$$\mathrm{IRR}\approx\frac{\mathrm{NAI}}{\mathrm{CAP}} \qquad [5]$$

The net annual income is equal to gross annual income minus the annual operating and maintenance costs. These O&M costs are usually a function of capital costs. However, for the sake of simplicity, we will assume that they are a fixed multiple of the gross annual income. The error incurred will be negligible since O&M costs in microhydro are minor. This leaves the task of estimating gross annual income (GAI) which is affected by river flow. The flow can be described by a hydrological probability function

$$F_{Q_A}=\mathrm{Prob}(Q<Q_A) \qquad [6]$$

The relationship between output power and flow varies from design to design and is expressed by the technical function of each

$$P=\mathrm{T}(Q_A) \qquad [7]$$

(See Figs. 1, 2 and 5 as examples.) At a given output power, an economic function determines the rate of earning money

$$R=\mathrm{E}(P)=\text{typically } K_E P^{\Gamma} \qquad [8]$$

The parameter Γ can be taken as 1 for grid connected systems so K_E becomes the economic value of 1 Joule and gross annual income is

$$\mathrm{GAI}=\int_0^{\infty} K_E'\,\mathrm{T}(Q_A)\,dF_{Q_A} \qquad [9]$$

(Where $K'_E = K_E \times 3.16 \times 10^7$ s.) The following hydrological probability function (proposed in Ref. 1) fits very well the typical flow-duration curves in the range of interest for micro-hydro (*i.e.* the range of flows smaller than the average annual flow):

$$F_{Q_A} = e^{\lambda Q_A^{-\mu}} \qquad [10]$$

The numerical integration of Eq. [9] using a distribution such as [10] gives us income. We now need a flexible expression for capital cost.

Let

$$\text{CAP} = C_T + C_P + C_S + C_R \qquad [11]$$

where the cost components represent respectively turbomachines, penstock, storage and the rest of the system.

Noting the economies of scale in machinery manufacture, we get

$$C_T = k_T q_R^\theta = k_T q_R^\theta n^{1-\theta} \quad \text{for } n \text{ machines} \qquad [12]$$

(Note the need to change to a normalised flow $q_R = Q_R / \overline{Q}_A$ to keep the coefficient k_T dimensionally simple.) The elasticity θ is typically 0.55 to 0.7.

Penstock size will depend upon flow and the penstock efficiency η_P we choose. Penstock cost can be shown to have the form

$$C_P = k_P (1 - \eta_P)^{0.4} q_R^{0.8} \qquad [13]$$

Storage costs only arise in the case of intermittent operation. Assuming that storage volume ($d{\cdot}A$) is chosen as a large fixed multiple of penstock volume we can use

$$C_S = k_S C_P \qquad [14]$$

Finally we have civil and electrical costs dependent respectively on flow Q_R and power P_R. As power is proportional to flow and treating all equipment as having the same cost-scale elasticity

$$C_R = k_R q_R^\theta \qquad [15]$$

We have in equations [11]-[15] the means of evaluating the changing cost of a system as parameters such as Q_R are varied.

6. Comparison of Options SO, PO and IO

Having optimised the IRR of each of the options by finding the best values for Q_R and other parameters, the three options can be fairly compared. A reference scenario was defined as follows:

$\mu = 0.8$, $\theta = 0.6$, $k_T/k_R = 0.3$, $k_P/k_R = 0.3$, $k_S/k_R = 0.001$

Figure 6 shows the rated flow Q_R, the gross annual income GAI, the capital invested CAP and the relative IRR for PO and IO, taking SO as reference. It can be seen that both PO and IO lead to larger schemes whose internal rates of return are higher than for SO. In the case of intermittent operation IO, the economic advantage is sufficient (10%) to justify considering this alternative. Similar advantage was observed across a representative range of scenarios.

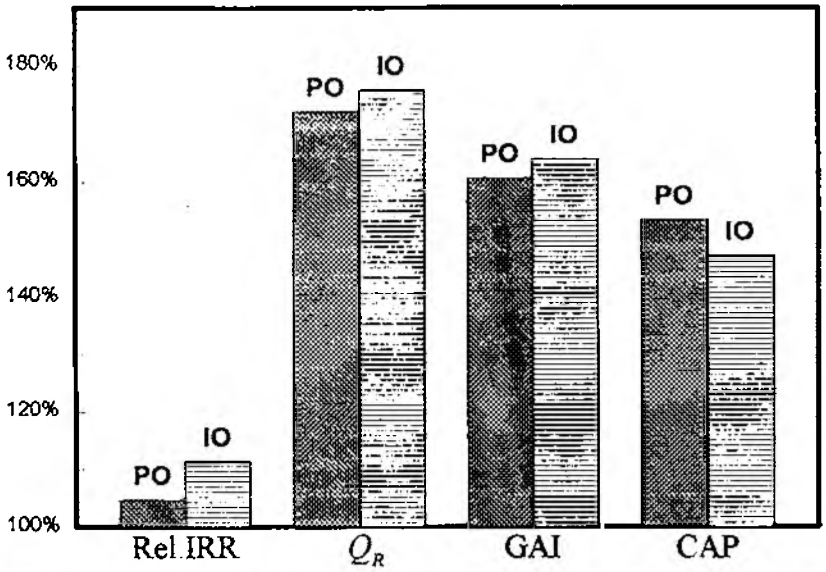


Figure 6. Comparison between IO, PO and SO.

Finally, the effect of varying the different economic and hydrologic parameters is analysed in Ref. 1. One example, relating to the hydrological parameter μ, is shown in Figure 7, confirming that IO and PO are especially attractive for small systems.

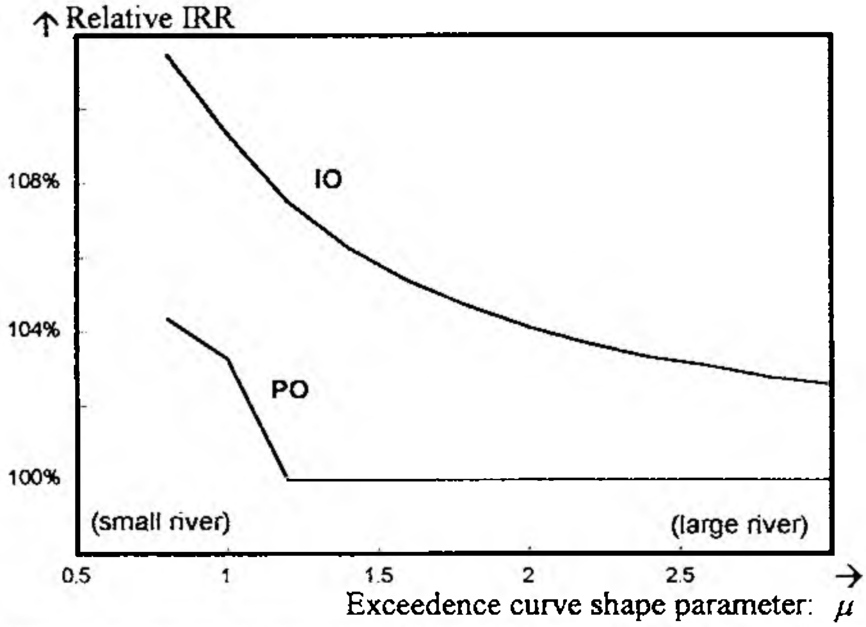


Figure 7. Relative IRR of PO and IO (SO as reference) when varying μ.

References

1. ALATORRE-FRENK, CLAUDIO. *Cost Minimisation in Micro-hydro Systems Using Pumps-as-turbines*, PhD thesis, University of Warwick, U.K, 1994.
2. NICHOLAS, WILLIAM G. "Using Pumps as Turbines. Selection and Application", *Hydro 88. Third International Conference on Small Hydro*, Cancún, Mexico, 1988.
3. WILLIAMS, H.C. "Gisborne Waterworks Intake", *New Zealand Engineering*, Vol. 22, No. 12, 1967, pp. 502-5.

DEVELOPMENT
TECHNOLOGY
UNIT

D
T
U

Working Paper No. 52

Destructive Effects of Moisture on the Long-term Durability of Stabilised Soil Blocks

January 2000

A.G.Kerali

Development Technology Unit
School of Engineering, University of Warwick
Coventry, CV4 7AL, UK
Tel: +44 (0) 2476 523122 Fax: 1203 41892
email: dtu@eng.warwick.ac.uk
http://www.eng.warwick.ac.uk/DTU/

DTU Working Paper 52 **February 2000**

DESTRUCTIVE EFFECTS OF MOISTURE ON THE LONG TERM DURABILITY OF STABILISED SOIL BLOCKS

by A G Kerali

1. INTRODUCTION

This working paper describes the summary of the research conducted at the University of Warwick during the 1998/99 academic year. The purpose of the research was to investigate the main mechanisms responsible for the deterioration of compressed stabilised earth blocks, hereinafter referred to as blocks, and to propose remedial measures for their improvement. The scope of the working paper at this stage is limited to the study of the influence of moisture on the durability of blocks.

A working paper describing the influence of moisture on the durability of blocks has been long overdue. A compressed stabilised earth building block may be defined as one formed from a loose mixture of soil and/or fine aggregate, a stabiliser and water in a damp mix which is compressed to form a dense block before the stabiliser hardens (Gooding and Thomas, 1995). After hardening the block should be able to demonstrate higher compressive strengths, better dimensional stability even on wetting and improved durability, than a similar block produced in the same manner but without the addition of the stabiliser. However, since the blocks are produced from soil as the bulk constituent, and given the poor resistance of soil to water, the long term behaviour of the blocks are dependent on the environment and on time (Fitzmaurice, 1958). For this reason, since the introduction of blocks a few decades ago, interest has been growing in understanding the critical factors governing the long-term durability of the material. The durability of blocks is likely to remain a major concern in the foreseeable future. Blocks may be popular and widely used in developing countries, but aggressive environmental conditions such as cyclic changes of moisture content, temperature and pressure over prolonged periods have tended to result in the unexpected shortening of their working life. This has been facilitated either through production shortcomings, or through irreversible changes in the microstructure of the material leading to failure (Tibbets, 1982). This working paper recognises the fact that although in practice several causes of deterioration my occur simultaneously, with synergy and a cumulative effect, it is nevertheless important to identify the most critical mechanisms, understand the processes involved, and propose remedial measures. This paper suggests that moisture, resulting from driving rains, rising damp, or the condensation of vapour is the most serious factor influencing the deterioration of blocks. If blocks are to become more widely accepted in future as alternative durable building material, then they must be seen to overcome the major drawback of their inability to resist prolonged exposure to moisture (Fitzmaurice, 1958; Agarwal, 1981; Spence and Cook, 1983; Odul, 1984)

The paper examines the experimental work done so far to explain the processes of moisture-linked deterioration. According to literature examined so far, it would appear both that no previous extensive research has been done in this area and that there is a substantial gap in our understanding of the problem. It is hoped that this working paper will contribute towards the orderly progression required to provide a satisfactory theory to assist in understanding the exact nature of the problem, and thereby pave the way for the generation of possible solutions.

The working paper is presented under five main sections namely; introduction, deterioration of blocks, tests and experiments, results and discussions, and conclusion. A list of references used in the preparation of the working paper is presented at the end of the report. Comments, suggestions, and contributions from other researchers, academics, practitioners and users are particularly invited and will be welcome and gratefully acknowledged.

2. DETERIORATION OF BLOCKS

2.1 **Basic Wall Characteristics**

In order to understand better the deterioration process, it is necessary first of all to appreciate the most important basic wall characteristics. For any new material, it is important to identify, evaluate and understand the basic standard criteria that will be used to analyse the ability of the material to maintain its essential initial qualities (Jennings, Kropp and Scrivener et al, 1994). It is necessary to begin with an evaluation process that will require positive answer to a set of given basic standard criteria. The four key and likely variables to which basic criteria of assessment could be applied include: relevant material properties, easily measurable properties, initial performance standards for use, and any set durability thresholds during the lifetime of the material (BS CP3 1950; Baker et al, 1994). We therefore seek to answer four questions.

Firstly, what are the primary and secondary properties of the material in relation to the particular application to which it is being designed for? In the case of wall construction for which blocks are produced, the required basic characteristics include strength, dimensional stability, and durability (Spence and Cook, 1983; Caroll, 1992). Important secondary characteristics include weather-proofness, fire resistance, thermal insulation, acoustical insulation and satisfactory appearance (Rigassi, 1995). These wall properties are to a large extent governed by the characteristics of the blocks from which the walls are built (Atkinson, 1970). Good initial block properties are therefore an important consideration in the study of the deterioration of blocks. Secondly, can the relevant properties be measured? Measurement of relevant properties should ideally be possible as and when required. It should be possible to take measurements through immediate tests (surrogate tests) in the case of compliance testing, through medium term tests (accelerated tests) in the case of laboratory experimentation, and through long term performance testing (exposure field tests) in the case of long term performance testing. What numerical limits and thresholds should be attached to the results? Thirdly, do the initial properties of the block fulfil the basic requirements at all in terms of specifications? Fourthly, will the blocks fulfil the requirements during the lifetime of the structure? It is worth noting here that the durability of blocks is closely related to the block properties but are not constant during the lifetime of the block.

It has been suggested in the literature on stabilised blocks that the durability of blocks is closely related to the block properties which in turn are not constant during the lifetime of the block (BS CP 3, 1950; Fitzmaurice, 1958; Atkinson, 1970; Ingles and Metcalf, 1972; Spence, 1975). Despite this potential problem, however, blocks have to be initially strong and remain with adequate strength to carry vertical or lateral loads for the service duration of the wall structure. The blocks have to be durable and weatherproof to exclude any undesirable influences of the environment such as rain, winds, rising damp or other severe weather conditions of exposure. Dimensional stability is necessary to avoid or contain undue volume changes due to expansion or shrinkage. Volume changes, especially under conditions of restraint such as within a wall structure, often induce undesirable cracks and fissures in walls. These have to be avoided if the service life of the block is to be prolonged.

2.2 Defining Durability and Deterioration.

The terms durability and deterioration are perhaps the two most commonly used words in the field of construction materials. A discussion of what these two words really mean and how they can be assessed in the context of compressed stabilised soil building blocks is necessary for a deeper understanding of the subject.

The underlying assumption driving this research is that durability is probably the most important quality for any construction material since adequate performance over a long time is expected, and implied. According to British Standards, durability is a measure, albeit in an inverse sense, of the rate of deterioration of a material or component (BSI, CP3 1950). For the purposes of this research, durability will be defined as the ‘measure of the ability of the block to endure or sustain its distinctive characteristics of strength, dimensional stability and resistance to weathering under conditions of use for the duration of the service lifetime of the structure’ after Baker et al, 1990. This definition would seem to suggest that the block must be able to maintain satisfactory performance throughout its service lifetime even under the adverse influence of the natural environment. It further suggests that although blocks may initially have suitable and desirable properties, the continued influence of adverse natural environment may alter these properties for the worse. When such alterations lead to substantial failure of the block ahead of, or within its planned lifetime, then the durability of the material can be brought to serious question. Fortunately however, although most construction materials will alter in one way or the other with time, their durability is not always brought to question. This appears to emphasise the fact that durability is not only related to time, but also to the expected function of the material, and conditions of exposure. Conditions of exposure greatly influence the degree of alteration that the block will witness over time.

The alteration process leading to failure may be termed deterioration. The term deterioration will be defined here as ‘the time related loss of quality of the block under the direct or indirect influence of the environment' (Atkinson, 1970). The term as defined may be used to distinguish failure due to the inability of block to fulfil its basic engineering functions from the alterations and possible eventual failure during the lifetime of the block. This term as defined suggests that durability may be taken to be the ability of the block to resist deterioration. It seems very likely, however, that the durability of the block may not be constant. This suggests that due to deterioration, the durability of the block may change considerably. For example, the water absorption and permeability properties of the block may be directly related to the structure and density of the block. However, the structure and density of the block may alter appreciably due to weathering (deterioration). This alteration may considerably increase the water absorption and permeability levels of the block thus accelerating degradation by softening the loose uncemented matrix of the block, and thereby inducing other degradation mechanisms. In short, deterioration and durability influence each other mutually but negatively (Jennings et al, 1994).

The premature degradation of blocks, and the rate at which this may occur, remain of major concern to most researchers (Hammond, 1973; Uzomaka, 1978). A certain length of time is normally required before deterioration starts. This period may be referred to as the initiation stage, which is later followed by the propagation stage when deterioration is progressing. This suggests that the service life of a block structure could be considered to have an initiation phase and a propagation phase which eventually lead to an unacceptable level of deterioration. The service life of the block is therefore the sum total of the duration of the initiation phase and the propagation phase (Baker et al, 1990).

It should however be pointed out that it is very difficult to define precisely a criterion for unacceptable deterioration. This is because such a level of deterioration may depend on a number of factors such as the nature of the use of the structure, the design and the consequences of failure. In moderate climates, The propagation phase is believed to be short in comparison to with initiation phase and hence the service life can be approximated to the duration of the initiation phase. After the initiation of loss of material, the erosion process might proceed at a rate depending on the degree of exposure of the block surface, the bulk resistance of the block, and the stabiliser type used in the production of the block. There is unfortunately still very little literature and data on the subject since no single long-term studies have been conducted. Several methods may exist for the prediction of the service life of construction materials. These include estimates based on experience, deductions from the performance of similar materials, accelerated testing, mathematical modelling based on the chemistry and physics of degradation processes, application of reliability and stochastic concepts, and through the use of neural networks (Baker et al, 1990; Jennings et al, 1994; Sjostrom et al, 1996). Despite the existence of all these methods it is still important to note that not all deterioration mechanisms are well understood.

2.3 **Mechanisms of Deterioration**.

The durability of blocks is affected by both surface and block factors. It is therefore important to understand in each case the source of the action, nature of the action/ reaction, the propagation processes, and the measuring techniques available to quantify their effect on the block over time. The most common influencing mechanisms in the performance of blocks are:

- ♦ Moisture related deterioration- in areas of seasonal, cyclic or continuous alternate wetting and drying which lead to the block retaining sufficiently high amounts of moisture to have deleterious effects. The softening and abrasive action of moisture leads to erosion of exposed surfaces (Fitzmaurice, 1958; Wischmeier 1958; Ingles and Metcalfe, 1973; Lunt, 1980; Stulz and Mukerji, 1988; Ola and Mbata, 1990; Gooding and Thomas, 1995).

- ♦ Temperature changes- in areas of high ambient temperatures causing dimensional changes, which when resisted by restraints of the wall results into splitting, warping, crazing, and cracking. Temperature variations may also induce reversible physical properties like strength, hardness, and rigidity. The retention of moisture and loss through evaporation are dependent on the surrounding temperature. A weakened block then becomes more susceptible to further damage from internal stresses since the maximum load it can carry reduces (Atkinson, 1970; Lunt, 1980; Spence and Cook, 1983)

- ♦ Freezing and thawing-in regions where the environmental conditions allow frost action on exposed blocks under restraints of the wall structure can cause deterioration. The moisture content of the block may near or reach saturation from thawing water or other sources. A block subjected to repeated cycles of freezing and thawing, coupled with the effect of the ice growth in the voids, and the associated development of both hydraulic and osmotic pressures, result in the build up of expansive forces within the pore system. This leads to disruption, loosening and breakdown of particles (Ingles and Metcalfe, 1973; Houben and Guillaud, 1994)

- Chemical agents- where present in the soil like sulphates, calcium hydroxide, soluble salts and acids could potentially react or instigate reactions with the cement bonding component in the block. Sulphate attack is common in clayey soils in terrains of high ground water levels containing sulphates. Exposed blocks, which absorb or permit the passage of water or moisture, enable the calcium hydroxide present in the hardened cement mix to be removed by dissolution, a process also known as leaching. Soluble salts such as sulphates and chlorides entering the pores within the blocks and crystallising within the pores may induce internal stresses leading to cracking. Acid attack on the cement element of an exposed block surface leads to direct disintegration of the hydrated cement mix. Chemical attacks may be prevalent where the blocks are used in foundation level applications, but similar attacks have been rare in the case of above ground use of blocks. The presence of moisture in the block facilitates and speeds up most of the chemical reactions (Neville,1986; Houben and Guillaud, 1996; PCA, 1971; PCI, 1970; Spence, 1975)

- Physical action –mostly occur through adhesive, abrasive, and erosive wear of the block surface, especially the corners and edges. Adhesive wear occurs when two solid surfaces slide against one another under high pressure leading to the removal of material from the surface of the block. Abrasive wear may occur when material is removed from the block surface by contact with, and cutting action of other hard particles. Erosive wear may occur through the impingement and softening action of fast moving liquids leading to loss of material. A block saturated with high levels of moisture over prolonged periods is more susceptible to further damage by physical action (Houben, Rigassi and Garnier, 1996; Lunt, 1980)

- Volume changes- may take place within the block due to the drying shrinkage, variations in temperature, and moisture variations. When the change in volume is resisted by internal or external restraints, cracking results. The block is inherently sensitive to moisture variations due to the presence of clay as its constituent component. The determination of the amount and type of clay present in the soil for block production is of paramount importance. The presence of moisture does have the potential to disperse the clay wafers (Scot, 1963; Van Olphen, 1977; Lunt, 1980; Torraca, 1988).

From the above summary list of possible deterioration mechanisms, the direct or indirect presence and action of moisture is a common denominator in influencing the degradation processes. The potential damaging effect of moisture on the long-term performance of blocks could now be recognised as the most critical processes affecting the durability of block structures (Ola and Mbata, 1990; Spence, 1975; Ingles and Metcalf, 1972; ILO, 1987; Houben and Guillaud, 1994). This research paper therefore focuses on attempting to understand the nature of the action of moisture that would for example lead to loss of material from the block. Mass loss is chosen as the performance criteria because it is more amenable to measurement than for instance strength loss, swelling or expansion, and cracking. The theory likely to explain the phenomena satisfactorily will be presented, and experiments that simulate the damaging processes conducted.

2.4 **Critical Moisture Action**.

As previously noted, changes in the properties of blocks leading to unacceptable levels of deterioration may result from both surface and bulk factors. Weathering through the abrasive and erosive action of moisture often leads to severe loss of material for exposed surfaces of blocks. It is therefore important to identify the sources of moisture, nature of its action, the transport mechanics and the measurable parameters. The inputs that could affect the performance of the block are wide ranging. The most significant of these however include the exposure level to environmental elements (in this case rain) and the manufacturing processes (stabiliser type and content, compaction pressure, clay content and mineralogy, moisture content, curing time, mixing, vibration etc). Both of this result into time varying product properties (density, porosity, permeability, water absorption and retention, etc). The research effort in this case is focused on the examination and quantification of the loss in mass from the block due to the action of moisture from external sources.

Driving rains, rising damp and condensation are the main sources of moisture that could potentially be detrimental to blocks (Torraca, 1988; Agarwal, 1981; Lunt, 1980; Houben and Guillaud, 1994; Norton, 1997). The cyclic wetting and drying of blocks often ensures that there is sufficient moisture to have a decrepitating influence. The durability of the block appears to depend to a great extent on the water absorption capacity, permeability and porosity properties of the block on the one hand, and on the nature of the moisture action, and the capacity of the block to resist disruptive forces on the other. Permeability and porosity appear to play a major role in the entry and retention of pore water, and its mobility inside the block. The actual destructive action of moisture once the block has been penetrated could largely be through adsorption resulting in surface energy changes, dissolution and softening of loose particles, disruption of loose bonds or through pore pressure generation resulting in disruptive internal stresses (Fitzmaurice, 1958; Wischmeier, 1958; Franklin and Chandra, 1972; Ingles, 1972; Darve, 1990). The capacity of the block to resist the disruptive action of moisture will determine the rate and extent to which weakening, softening, swelling, shrinkage, or complete disintegration and loss of material will occur. This analysis would appear to imply that a block which is either impermeable, has a high intergranular strength, and is non-reactive to softening, would be very durable. The question to ask is; given the present fragmented knowledge, can such a block be made? Impermeability and high intergranular strength are directly related to the microstructure of the block, an area rarely covered in stabilised soil literature to date but through which major advances have of recent been made in the improvement of comparable building materials.

According to the literature (Ingles, 1962; Herzog and Mitchel, 1963; Lea, 1976; Houben and Guillaud, 1994), in a compressed cement-stabilised block, the cement binder undergoes a three-phase reaction with the clay component of the soil. This three-phase reaction results into a three matrix mix comprising; an inert sandy matrix bound with cement, a matrix of stabilised clay, and a matrix of unstabilised soil. Assuming that the matrix of unstabilised soil represents the greatest constituent of the block, and yet is also the most vulnerable of three matrices to softening and erosive loss, one would be inclined to predict that the continued exposure of the block to moisture would lead to irreplaceable loss of material from the block over time largely due to the unstabilised matrix.

Other possible mechanisms that may arise from the action of moisture could include ion exchange in the clay minerals, capillary effects, and stress relief (Franklin and Chandra, 1972). Clay bearing blocks are especially susceptible to moisture weakening. Among the several mechanisms that may account for this behaviour, ion exchange appears to be dominant. Clay minerals in the block are surrounded by an atmosphere of adsorbed cations, usually hydroxyl ions that are still loosely bound (Torraca, 1988). The particles can be dispersed in water solution containing negative ions, or can be flocculated by positive ions. The ionic dissociation of the dispersing solution as measured by its dielectric constant controls the efficiency of the dispersion, but the type of clay used also matters. Sodium clays are the easiest to disperse then potassium, magnesium and barium clays. Some clays, for example illite and montmorillonite, contain inter-layer potassium ions that favour hydration. In this instance, the swelling of the crystal lattice may well assist in the dispersion process. Capillary effects, due to the presence of pores in the block microstructure, are likely to play an important part in moisture degradation. Water menisci in the block increase their radius of curvature as the block becomes saturated, so that capillary tensions at grain contacts, and at the tips of cracks are reduced. Also water that is drawn into the block by the action of strong capillary forces may compress entrapped air in its path, resulting in the disruption of the block. Stress relief is probably also an important mechanism, since compressed clay bearing soils subjected to diagenetic forces, are likely to store elastic strains that will be released if intergranular bonds are weakened by the repeated action of water.

In order to quantify the loss of material as a result of the action of moisture, it is necessary to establish a good scientific yardstick of measurement and comparison. There are at present a number of durability tests in use, but with scant application. These include the wet-dry-cycling test, the abrasion test, the rain erosion test, the freeze thaw test, and the immersion test (BSI 4315, 1970; Lunt, 1980; Spence and Cook, 1983; Houben and Guillaud, 1994; Rigassi, 1995). These tests attempt to measure weight loss but cannot be said to measure the resistance of the block to weathering, which is what would be useful. A test is therefore required that would measure the resistance of the block sample to weakening and disintegration resulting from an accelerated abrasion, wetting and drying cycle. The test could be fairly simple, practical, accurate and replicable in application. An index test that would not only measure loss of material but also provide an indicatiobn of the comparative degree of alterablity of the block would be more useful. This working paper proposes such a test in Section 3.

3. TESTS AND EXPERIMENTS

3.1 Need For Tests

Stabilised blocks used for external walling have been widely reported in the literature as being prone to deterioration through weathering i.e., softening and erosion leading to loss of mass. The blocks have also been reported to experience lower wet compressive strength and larger dimensional changes than walls made of other comparable masonry materials such as concrete blocks and fired or burnt clay bricks exposed under similar conditions of service (Fitzmaurice, 1958; Lunt, 1980; Spence and Cook, 1983; RILEM, 1987; ILO, 1987; Stulz and Mukerji, 1988; Houben and Guillaud, 1994). The personal experience of the author who built several dwelling units using the same material in East Africa would also add testimony to this premise (Kerali, 1996). The problems have tended to be more serious in tropical upland climates and humid wet climates which experience more seasonal and very intense rainfall than in the more arid hot dry climates (Eaton, 1981; Spence and Cook, 1983; Ola and Mbata, 1990). Prolonged periods of wetness due to continued exposure from double or triple rainy seasons could be of particular concern

Tests and experiments on blocks on blocks are necessary firstly to measure the block properties upon which durability is dependent like strength, water absorption, micro- and macro-structural changes, and secondly to monitor the blocks performance in conditions which simulate the cause of the deterioration. The tests will provide experimental results and data from which general and localised trends could be identified, and from which comparissons can be made with theoretical predictions or other available data. The tests also would provide an opportunity with which the validity of currently held beliefs could be tested, and any agreements or departures from the norm spotted. The literature on stabilised soils still has scanty and/or little information and data on the influence of moisture on the durability of blocks (Gooding, 1995; Spence, 1975). Translating the experimental data into information to facilitate the potential improvement of the durability of blocks would be the broad objective.

3.2 Selection of Tests to be performed

Four separate tests and experiments, all of which have direct bearing with the investigation of the effect of moisture on the durability of blocks, were selected and conducted. The tests include the wet compressive strength test, the water absorption test, the slake durability test, and microstructural analysis of block samples. Although the wet compressive strength test and the water absorption tests are both now standard performance tests widely described and used for stabilised soils, they were originally developed for concrete blocks and fired bricks. The wet compressive strength of blocks has to be known because it is on the value of the wet compressive strength that structural designs are based. The compressive strength values also give an overall picture of the quality of the block and are an indication of the hardness of the hydrated cement paste that binds the various particles together. In most of the climates where the wall will be wet most of the time, it is critical to know the wet compressive strength values for design and compliance purposes. Moisture, it has been stated in several parts of this report, is responsible for most of the damage in blocks. Through the water absorption test, it should be possible to determine the ability and extent to which blocks can absorb moisture. Knowledge of the water absorption levels of blocks could serve as useful criteria for setting limits and for investigating possible ways of reducing the same in order to improve on the durability of blocks. The slake durability test may not yet be a standard test for stabilised soils, but it remains a standard durability test for measuring the resistance of clay bearing rocks to alteration from the accelerated action of moisture (Franklin and Chandra, 1972). The test was adopted for use with blocks in this research because of its several attractions over existing methods such as the wet-dry-cycling test, the abrasion test, the rain erosion resistance test, the freeze thaw test and the immersion test. All existing methods are non-standardised, fragmented and limited in approach, and rely heavily on the operator for results. A test was sought that would be simple, controllable, reproducible, practical, accurate, and reliable. The slake durability test was able to measure not only the loss in mass but also provided some information by way of an index to show the degree of alterability of the block when subjected to controlled cyclic wetting and drying, erosion and abrasion. The test was also easily applied to test concrete block samples and fired brick samples, and the results compared to those obtained from blocks. Interest in examining the microstructural characteristics of blocks, an area not covered anywhere in stabilised block literature, emanated directly from the results obtained from the standard and adopted tests in an attempt to understand the nature of the distinct phases and the degree of connectivity of the block constituents. The test procedures and operations are presented in the section 3.4.

There are several manufacturing variables that could affect the performance of blocks. These include soil type, cement content, compaction pressure, moisture content, and curing regime (Rigassi, 1995). In the experiments conducted it was decided that of these several variables, only the cement content be varied while all the other parameters would remain fixed. The reason for this decision and approach was based on the fact that it was the stabiliser content which, according to the literature on stabilised soils, was significantly responsible for the improvement in strength, dimensional stability and durability of blocks. Although compaction pressure could contribute towards increasing the densification and thereby reducing voids, it is the stabiliser content that is responsible for binding, sealing, reinforcing and imparting flexibility to the block (Stulz and Mukerji, 1983). By binding the soil particles tightly together, the stabiliser the stabiliser may increase the compressive strength and impact resistance of the block, as well as reducing its tendency to swell and shrink; by sealing all voids and pores and providing a waterproofing film, the stabiliser may prevent or reduce water absorption; by imparting a degree of flexibility which allows the soil to expand and contract within limits, the stabiliser may help to reduce cracking; Conversely, by reinforcing the soil, the stabiliser may reduce excessive expansion and contracting. The effect of stabilisation is greatly increased when the soil is compacted (Ingles and Metcalfe, 1972; Stulz and Mukerji, 1983; Spence and Cook, 1983; Thomas and Gooding, 1995). In the experiments conducted all blocks were compacted prior to curing to a compaction pressure 10 MPa, a value considered to be high enough to produce the best possible quality blocks. In practise however, compaction pressure values much less than 10 MPa are used. In subsequent experiments to follow, both the compaction energy and the cement content will be varied. The stabiliser used in these experiments was ordinary Portland cement conforming to BS 12: 1972. This is because of all known stabilisers currently on the market, it is only ordinary Portland cement which is widely available and used in most parts of the world. It is likely to remain the stabiliser of choice due to its well-established reliability, availability and quality record.

3.3 Specimen Preparation

The preparation of specimens was considered to be one of the most important stages in the execution of the experiments. Extra care had to be taken with the soil, cement mix, moisture content, compression, curing, and sizing of the samples. The high levels of accuracy, reliability and consistency demanded by the experiments be maintained throughout the testing regimes, and for all the different types of tests conducted. Specimen preparation describes the raw materials used, the mix proportions, addition of moisture, the compression method used, the curing regime, and the dimensions of the samples.

For the raw material, an artificial soil for laboratory testing purposes was prepared by mixing ordinary builders sand and potters powder clay of the kaolinite type (grade E). Kaolinite clay was selected for its known non-expansive nature (Craig, 1987; Das, 1983; ILO, 1987; Webb and Lockwood, 1987; Lunt, 1980). Preparing the soil raw material in this way was found to be necessary in order to ensure that the soil mix was a fixed variable for all the different kinds of experiments envisaged. This would also facilitate consistency and repeatability in the research. For purposes of this study, the soil will henceforth be described as 'Soil S'. The proportioning of the ordinary builders sand and the clay powder followed the recommendations obtained from the literature to the effect that an ideal soil would have an optimum raw materials composition of: sand 75%, fines (silt and clay) 25% (Fitzmaurice, 1958; United Nations, 1964). Of the fines, at least not less than 10% has to be clay. The actual mix then used consisted of;

Sand 75%
Silt 13.5%
Clay 11.5%
Total 100.0%

A shrinkage test and a simplified sedimentation test were used to confirm the limits for the different constituents (Houben and Guillaud, 1994). The sand and clay were thoroughly mixed together using the Hobart Machine mixer until a uniform colour was achieved. Proportioning the mix of the soil raw material with the cement stabiliser was done in varying quantities, by percent weight of cement from 3% by weight in 2% increments up to 11% by weight of the soil as follows: 3%, 5%, 7%, 9% and 11%. A total of ten blocks of average dimension 290x140x100mm were subsequently made in this manner. An extra block, the eleventh, was also produced but without cement stabilisation. For each of the experimental blocks, a fresh mix of the soil S was prepared.

The optimum moisture content was determined, or more precisely, estimated by trial and error (ILO, 1987; Stulz and Mukerji, 1988). To the already prepared soil S mix with cement was added approximately 4% by weight distilled water. The entire mix was left overnight for 24 hours in order to homogenise and get absorbed evenly. After the twenty-four hours, a further 4% to 5% more distilled water was added. To approximate that the optimum moisture content had been near achieved, a drop test was used to ascertain the same (Smith and Webb, ILO/UNIDO Technical Memorandum No. 12, 1987). The constituent parts of the mixed soil preparations were separately weighed using an accurate and sensitive electronic weighing machine to plus or minus 0.05g. To improve on the degree of mix, a mechanical mixer, the Hobart machine, had to be used.

To produce the blocks, a pre-installed BREPAK machine designed on the quasi-static compression principal was used for all the samples. Before filling the mould for each compression, the mould lining was lightly oiled with used engine oil. The soil was carefully poured into the mould in three equal amounts, all pre-weighed, packed and sealed in light transparent plastic bags. After each pouring, the soil was levelled in the mould. The use of the BREPAK machine was based on the operational manual of the machine (Webb and Lockwood, 1987).

The blocks were compressed by the pumping action of the side pump up to 10MPa (517Psi). The machine is equipped with a pressure gauge, which was found useful in confirming the pressure excerted. Each block took approximately 50-60 strokes of the hand pump, as opposed to the recommended 40 in the manual. The last 2-3 strokes of the hand pump however required some excertion. The hydraulic pressure was released using the flow valve screw causing the handpump to become slack. The mould cover was then rotated sideways to expose the green block, which was, then demoulded. The green blocks were then carefully removed using two flat 20mm plywood base plates, and immediately placed in plastic bags and left to cure in the laboratory. The dimensions of the green blocks were recorded and so were the combined weight of the blocks and plywood base plates.

Curing of the blocks consisted of two distinct phases described herein as primary and secondary phases (Webb and Lockwood, 1987). The curing time, temperature, duration, and moisture conditions were of particular interest to the experiment. Primary curing, whose purpose is to ensure that moisture is retained in the block, and not lost rapidly, was done for a period of five days. Laboratory dry conditions were used with curing temperatures of 22-24 C. After the five days, the blocks were noticeably lighter in colour than when demoulded. Each of the blocks were marked using permanent ink markers in each case to clearly show the percentage cement content, moulding pressure, date and time of production, and an identification number (BRE OBN, 1980). This decision to mark individual blocks was to be found very useful later on. In order to enable the blocks to further achieve strength, secondary curing was allowed to continue for a further twenty three days. The clearly marked blocks were placed side by side and covered with a large polythene sheet. This was done to slow down evaporation and to protect the blocks from external interference. The blocks were then left to dry in this manner under laboratory dry conditions.

After a total of twenty-eight days, the blocks were carefully weighed, and demarcated for cutting into smaller 100x100x100mm cubes. The dimensions of the left overs, together with their individual weights, were also taken using callipers and an electronic weighing device. All this information was marked onto each sample individually. The samples were then ready for further testing.

3.4 Tests and Procedures

3.4.1 Compressive Strength Test

The compressive strength of blocks is perhaps their most important property. The main aim of the compressive strength tests was to determine the wet compressive strength values of the blocks. It is the wet compressive strength value, which is normally lower than the dry compressive strength, that is used in the structural design of buildings. The compressive strength test done is a standard test based on BS 6073 Part 1, 1981 and on BS 3921, 1974.

After the 28 days curing period, the blocks of average dimension 290x140x100mm were cut using the concrete lathe machine to reduced cube sizes of approximate dimensions 100x100x100mm. The main compression equipment used was the Denison Concrete Testing Machine 7229/T91081 with a maximum load of 100KN. The machine was certified to grade one calibration for the test duration. Attached to the machine is an automatic electronic input and output recording device. Figure 1 shows a photographic record of the compressive strength test taken during the experiment.

Figure 1. Compressive strength testing of block samples.

Two blocks in each category of varying cement content from 3% in increments of 2% up to 11% were tested for wet compressive strength. Each block sample of dimension 100x100x100mm was soaked for 24 hours or overnight in ordinary tap water. They were then removed and kept aside for 30 minutes to let the extra surface water to drip off, then capped with two 100x100x20mm thick plywood pieces. The capped samples were then carefully placed within the set marking pins of the compression-testing machine. The crushing load was then continuously applied without shock to the sample at a rate of 15 KN/min till failure, and in this way the maximum crushing load was obtained for each sample. The computerised device of the testing machine then generated all the input and out information namely, reference, sample type, date, time of test, loading rate, sample dimensions and maximum crushing load for each individual test.

The wet compressive strength was then calculated in each case from the ratio of the maximum load and the cross sectional area of the block in N/mm2.

3.4.2 Water Absorption Test

The aim of the water absorption test was to determine the percentage moisture absorption capacity of the block samples. Cut block samples were weighed when laboratory dry, immersed in water for 24 hours, removed and weighed again. Samples of concrete blocs and fired bricks were also treated in the same manner and under the same conditions for future comparison purposes. An accurate electronic weighing machine was used in case, to an accuracy of 0.05g. The percentage moisture absorption by weight was calculated from the formula:

$$Mc = \frac{Ww - Wd}{Wd} \times 100 \ (\%)$$

Mc = percentage moisture absorption (%)
Ww= mass of wetted sample (g)
Wd= mass of dry sample (g)

The apparatus consisted of an accurate weighing balance, a stop clock and a water trough with a capacity to hold up to 5 fully immersed blocks (Rigassi, 1995). The entire test took two days to complete mainly due to the overnight soaking of the block samples in water.

3.4.3 Slake Durability Test

The aim of the slake durability test was to provide an index that could help show the degree of alterability of the block samples when subjected to continuos wetting and loss of material through softening and erosion. This is a pioneering test on blocks modified after a similar type of testing regime previously used for clay bearing rocks (Franklin and Chandra, 1972). Samples of concrete blocks and fired bricks were also tested. Figure2 shows the diagrammatic illustration of the slake durability testing equipment.

Figure 2. Slake durability testing equipment.

The equipment mainly consists of a drum 140mm in diameter and 100mm long with 2mm-sieve mesh forming the cylindrical walls. Four block sample pieces with a combined total weight of between 450-500g were loaded into the drum. The drum was then rotated at 20 revolutions per minute for 10 minutes through a container or more accurately a bath filled with distilled water. After 10 minutes of the slow rotation, some percentage of the block samples were retained inside the drum, while others were dissolved or suspended in the distilled water filled bath. The block samples were removed, dried overnight in the oven, and then weighed. The slake durability index was then calculated using the formula:

$$SDI = \frac{Mf}{Mi} \times 100$$

SDI= slake durability index (%)
Mf = final mass (g)
Mi = initial mass (g)

After drying, the moisture absorption test was done as before. The test was then repeated for a further 10 minutes cycle, and the slake durability index determined again. The water absorption was also determined again as before for each sample tested.

3.4.4 **Micro-structural Examination.**

Specimen of the block samples made from varying cement content as before and compacted at 10 MPa were examined using the Optical Microscope and screened at magnifications ranging from x10 to x63 (Brandon and Kaplan, 1999). Samples of fired bricks and concrete blocks were also examined in the same manner. The samples which were representative of the whole in terms of features were first observed while in a dry state using the attached binoculars and then their images recorded using the photographic high speed and high resolution cameras also attached to the equipment. The exercise was repeated for the same samples after overnight soaking in water till saturation.

The ultimate good resolution of the recorded pictures was a result of careful sample preparation, imaging in the optical microscope, and individual recording. The optical microscope was selected for the examination of the microstructure because the visual impact of the magnified image would be immediate, and the interpretation of the spatial relationship of the phases would be familiar even to the casual observer. Of major interest to were the inhomogeneity i.e., spatial variation in features and their distribution and the morphological anisotropy of the samples i.e., orientation variations. Since the literature on stabilised soils does not mention any previous work done in investigating the microstructure of blocks by any of the researchers whose works have been assessed so far, it would be reasonable to state that this test has been a pioneering and pace setting contribution to knowledge. The results are presented in Appendix I, and discussed in Section 4 of this report.

4 RESULTS AND DISCUSSION

4.3 Wet Compressive Strength

The results of the 28-day wet compressive strength values obtained are presented in a tabulated form in Appendix A and in a graphical form in Appendix D. From these results general and localised trends can be discerned.

According to the tabulated results in Appendix A, it would be reasonable to predict that for a given constant compaction pressure, an increase in absolute wet compressive strength can be achieved by increasing the cement content. From the graphical presentation of the results shown in Appendix D, the rate of increase in strength can be approximated. The graph reveals that the absolute increase in wet compressive strength appears to remain constant but then increases less at the lower cement contents but more at the higher cement contents. For instance, when the cement content is doubled from 3% to 6% at constant compaction pressure, a wet compressive strength increase of 130% is achieved; further doubling of the cement content from 6% to 12% would produce a projected increase in wet compressive strength of up to 145%.

The results also show that all the blocks produced at the varying cement contents from 3% in increments of 2% up to 11% but at constant compressive pressure of 10MPa, have 28-day wet compressive strength values well above most of the recommended minimum values for use in structural work. According to the literature, several different values of 28-day wet compressive strength, all above 1.0 MPa are proposed; 1.2Mpa (Houben, Rigassi and Garnier, 1996), 1.4 MPa (Lunt, 1980), 1.0MPa to 2.8 Mpa (ILO, 1987). According to the

1987 ILO Report, the 1.0 MPa 28-days minimum wet compressive strength values are recommended for dry arid zones while the 2.8 MPa minimum 28-day wet compressive strength values are recommended for the wet rainy zones. The high wet compressive strength values obtained in the tests can be explained by the use of a high compressive pressure of 10MPa, which was used to produce the blocks. In practice, however, compressive pressures of less than 10 MPa are adequate.

The behaviour of the block samples tested for wet compressive strength showed that although blocks may be considered as hard and rigid materials, they remain brittle and fragile under the action of extreme loading. The most notable visual observation made during the tests was the fact that damage tended to start from the block edge and corners, and extend to the centre and inner parts. It would also appear from the tests that if the load were removed, the samples would not fully recover their initial shape and configuration. This appears to suggest that blocks are not elastic but could be plastic from their behaviour on removal of the tress. The explanation for this could stem from the fact that blocks are heterogeneous materials composed of many different crystals and particles, held together through contacts and joints with variable degrees of strength. Some of the joints or particles start to start to break up well before others, causing irreversible deformations that remain even after the removal of the stress.

4.4 Water Absorption

The experimental results of the water absorption test are tabulated in Appendix B, and shown in a graphical form in Appendix E and Appendix F. Appendix E shows the effect of cement content increase on the water absorption capacity of the block. Appendix F shows the effect of bulk density increase on the water absorption property of blocks.

According to the tabulated results in Appendix B, the 28 day mean water absorption values for the various samples tested range from 5.3% for the 11% cement content samples to 8.1% for the 3% cement content samples. The recommended maximum water absorption value is 15% (ILO, 1987). Houben, Riggassi, and Garnier, 1996 proposed the maximum value of 20% water absorption for blocks. During the same test, samples of fired bricks and concrete blocks were also examined for purposes of comparison. The results, also shown tabulated in

Appendix B, reveal mean water absorption values of 9.1% and 3.4% for the fired brick samples and concrete block samples respectively. These results appear to suggest that blocks compacted at 10MPa but with varying cement contents from 3% to 11% have water absorption values which fall within the recommended limits, and which compare well with water absorption values of similar materials like concrete bricks and fired bricks. The fired brick samples actually absorb more water than any of the block samples tested. Does this mean fired bricks are more porous than blocks? Do fired bricks have more voids than blocks? Are the voids more interconnected? Are the values obtained significant in any way for blocks? Why do the fired bricks absorb more water yet are considered to be more rigid and durable? An investigation of the microstructure of blocks and comparable materials is called for. The issues to be determined include the interconnectivity of the pores, the pore size distribution and the total porosity. The results do show that in ambient conditions, the blocks do have a potential to retain significant amounts of moisture.

According to Appendix E, an increase in cement content has the effect of reducing the water absorption value of the blocks produced at constant compaction pressure. A doubling of the cement content from 3% to 6% results into a reduction in mean water absorption of 22%. A further doubling of cement content from 6% to 12% is projected to reduce the mean water absorption by 15%. This shows that the increase in cement content results into a reduction in water absorption, and the reduction is more at the lower cement contents but less at the higher cement contents, eventually becoming constant above the 9% cement content level.

According to Appendix F, an increase in bulk density from 2055Kg/m3 (3% cement content) to 2082Kg/m3 (11% cement content) results into a reduction of water absorption of up to 35%. Above the 2079Kg/m3 density value (9% cement content), any further increase in cement content does not correspond to any further reduction in water absorption. Below the 5% cement content level, an increase in density from 2055Kg/m3 (3% cement content) to 2060 Kg/m3 (4% cement content) results into a reduction in water absorption of 20%. Above the 5% cement content level, an increase in density from 2065Kg/m3 (6% cement content) to 2067Kg/m3 (7% cement content) results into a reduction in water absorption of 2%. Further incremental increase in cement content would result into no further reduction in water absorption. This appears to suggest that a further increase in density would not necessarily eliminate water absorption by the block. The block would still be able to absorb significant amounts of moisture.

In practice, water can gain access to the block either in liquid phase in the case of rainwater infiltration or suction from a wet surface, or in the vapour phase in the case of condensation or adsorption, but leaves the block almost exclusively in the vapour phase through evaporation. Therefore the water content of the wall should be determined not only by its contact to water sources but also with its water vapour balance i.e., evaporation minus condensation and adsorption. Given that the block undergoes s seasonal cycle with maximum water content in the rainy season and minimum water content in the dry season, such cycles constitute an added complexity in analysing the moisture balance and therefore any remedial steps that could be taken. Would plastering the wall, which in effect hinders evaporation, be useful or harmful? It would be recommended that a test of moisture conditions should follow the entire wet- dry cycle, and be repeated annually. There is currently no known test in stabilised soil literature that corresponds to this. Further studies will need to be conducted in this area as well as in respect of water penetration, mobility, retention and solubility of constituents.

4.5 Slake Durability Test Results

The results of the slake durability test are tabulated in Appendix C, and shown in graphical form in Appendix G and Appendix H.

According to the tabulated results in Appendix C, the mass loss in blocks stabilised with varying cement contents from 3% cement content to 11% cement content ranged from 37% to 7% respectively. Fired brick samples and concrete block samples were similarly tested with results showing mean mass losses of 1% and 4% respectively. This shows that despite the significant water absorption capacity of the fired brick and the concrete block, the integrity of the two materials when subjected to alternate wetting and abrasion, were upheld. One of the three fired brick samples showed integrity of 100% against wetting and abrasion. Does the answer to integrity lie in strong interparticle bonding? Blocks stabilised at 11% and 9% cement content experienced mass losses of less than 10%. Blocks stabilised at 7% and 5%

cement content experienced mass losses of below 20%. Blocks stabilised below the 5% cement content level experienced mass losses of between 21% and 37%. Recommendations encountered in the literature for acceptable limits of mass loss set the maximum at 10%(Spence and Cook, 1983; Fitzmaurice, 1958). Apart from these two references there appears to be complete silence to the possibility and degree of mass losses in blocks. The most significant aspect of this experiment, which attempted to simulate the combined effect of accelerated wetting, softening and abrasion of blocks, was to confirm that blocks do indeed lose mass faster than comparable materials. The answer to redressing this potential problem would probably lie in understanding the degree of bonding, sealing, flexibility and reinforcement capacity of the binder and the contribution of compaction pressure. Why is the integrity of the fired brick, which absorbs more water than the blocks, almost total? The microstructure of the two materials will need close examination if the shortcoming is to be solved. Therein could lie the key to the strengthening of the inter-particle bonds in blocks while continuing to expose them to moist conditions and thereby increase their desirability in the developing countries.

The general trend shown by the results plotted in Appendix G is that an increase in cement content would result into a reduction of mass loss. A doubling of the cement content from 3% to 6% results into a mean reduction in mass loss of 62%. Further doubling of the cement content from 6% to 12 % results into reduction in mass loss of 57%, i.e. only 5% improvement. By extrapolation, would higher cement content of above 15% approximate a zero mass loss? Would further cycles of the slake durability test result into further disintegration of blocks? After how many cycles would the mass loss become constant, or would a weakened block disintegrate?

As with the case of wet compressive strength, it is possible to relate mass loss variation to density variation, with the latter depending mostly on the cement content variation. According to Appendix H, the general trend that emerged showed that an increase in density from 2055Kg/m3 (cement content 3%) to 2063Kg/m3 resulted into reduction in mass loss of 54%. A further increase in density from 2063 Kg/m3 (cement content 5%) to 2067 Kg/m3 (7% cement content) resulted into a lesser reduction in mass loss of 30%. Does this suggest that the higher the density i.e., the lesser the voids, the lesser the mass loss? At a density of above 2085Kg/m3, approaching the zero voids line, would the mass loss be nil? Would it be possible to achieve this through better processing methods like more vibration before compaction? This aspect, together with the microstructural nature of blocks, will need further investigation.

From the results of these experiments, the slake durability test which combines wetting and abrasion of blocks has the advantage of allowing fast comparisons to be made between blocks made from the same batch, different batches and even different materials to be compared efficiently and reliably. Unlike other durability tests that rely heavily on the actions and senses of the operator, the slake durability test was found to be simple, practical, reproducible, controllable and repeatable.

Among the various factors that could influence the results, the following could be of importance; the apparatus (sieve, drum size, speed of rotation), the sample (size, shape, weight, microstructure, and storage, drying), duration of slaking, nature of the slaking liquid (chemistry and temperature). All these factors are common to most standard tests but could be simplified and and adopted. The need to develop a classification of block methods that are quantitative and independent of block dimensions is long overdue. No previous durability tests done in the past have been able to achieve the degree of reliability and controllability made possible through the slake durability test.

4.6 Microstructural Characterisation

The photographic records of the microstructural investigation of blocks and samples of fired bricks and concrete blocks are presented in Appendix I and Appendix J.

According to the images in Appendix I which show the microstructure of a wet block sample and that of a dry block sample both stabilised with 5% cement content, the volume fraction of voids appear quite prominent. In the wet block sample, films of water, which have filled the pores, are evident. The interparticle bond appears to be basically light contact bonds possibly with weak connectivity. It is difficult at this stage, without the benefit of further tests, to discern the distinct different constituents of the block microstructure, such as clay and cement. The larger particles that are visibly prominent due to relative size could be sand particles. What this apparent weakness in bonding and connectivity have on strength may not be possible to discern at the moment but their implications on softening, dissolution and disintegration could be serious. This is a pioneering investigation in as far as research on stabilised soils is concerned. This investigation also has the potential of revealing contamination or unwanted inclusions in block samples due to inadequate processing techniques. The fact that the wet samples reveal complete saturation with absorbed moisture could have a bearing on the future performance of blocks. Would it be possible to device processing methods that can reduce the level of the presence of moisture in blocks? Would it be possible to establish relative quantities of the three mixed constituents comprising the block namely, the inert sandy matrix bound with cement, the matrix of stabilised clay and the matrix of unstabilised soil? Which of these three matrices is more prone to softening, dissolution, and erosion? Does stabilisation affect all the sand particles? Further research is required in all these areas. The literature on stabilised soils does not cover these pertinent and critical questions.

The fired brick samples and the concrete block samples appear to have exceptionally high degree of connectivity, almost approximating strong covalent bonding. In the fired brick sample, the clay crystals appear to have melted into a continuos mullite and quartz structure. A hard and non-crystal porous mass appears to have been formed. It would be reasonable to suggest that the higher the firing temperature, the stronger the brick formed. The recommended wet compressive strength of fired bricks is in the order of 5.2 MPa (BS 3921, 1974). The concrete block sample reveals a hard porous but brittle material. The degree of connectivity of the constituent particles also appears to be much stronger than in comparable stabilised blocks. The recommended wet compressive strength of concrete blocks is 2.8 MPa (BS 2028, 1968).

The limitations of this investigation could arise from the fact that the samples used were not as flat and coated as would have been required. No chemical enchants solvents or differential imaging agents were used. The focusing of the images also relied on the use of the eye before imaging. Nevertheless, reasonably good images discernible to the casual observer were obtained. Future research into the differentiation and separation of the different phases that could offer a quantitative approximation of the different constituents should be done.

5 CONCLUSION

The conclusions that could be made from the results of these exploratory tests and experiments are wide ranging, and include;

- The wet compressive strength of a block is a valuable indicator of its quality, and by implication, its durability. Increase in cement content results in an increase in the wet compressive strength value of blocks made at the same constant compaction pressure. The increase in absolute strength remains constant but increasing less at the lower cement content levels but more at the higher cement content levels. Increase in cement content could be a more effective method of increasing wet compressive strength values, and by implication the durability of blocks than an increase in compaction pressure. The final wet strength reached by a block is much more sensitive to variations in the cement content than to densification.

- The moisture absorption capacity of the block could be significantly correlated to its durability. Increase in the cement content of block results into a reduction of its water absorption capacity. Further progressive increase in cement content beyond about 9% however only results into a reduced but constant water absorption by the block. The water absorption value of a block reduces with an increase in density, but a further increase in density results into a constant water absorption value. The ability of the block to expel moisture should also be studied in trying to arrive at the moisture balance of the block, and in analysing its potential damaging effects.

- Blocks made with the highest possible standards of manufacture can still be capable of absorbing water and losing mass when subjected to continued wetting and drying, and therefore softening and erosion. Current durability tests are fragmented, non-controllable and hardly reproducible. A standardised test that can approximate the action of wetting, drying and abrasion is required. An increase in cement content results in a reduction in mass loss, but the least depreciation of blocks would require cement contents approaching 15%. This would be highly uneconomical, so more stringent durability thresholds are needed for lower cement contents especially in wet areas of use. A quick predictive durability test has not yet been developed for blocks.

- The microstructural characterisation of block samples revealed the presence of a large volume fraction of voids, which could be detrimental to the durability of blocks. The characterisation of blocks in this manner could be of tremendous potential in essaying to understand the nature and degree of bonding of the particles inside the block. Its use could be further extended to analyse the strength of interparticle bonding and/or inter phase connectivity in blocks.

Deliberate further research will still be needed to test refine and validate the findings contained in this report with a view to developing a reliable long-term durability model. Improvements to the durability of blocks can only become possible when most of the currently unanswered questions are settled. The most probable likely answer will lie in ways to achieve higher interparticle bonding and the exclusion of the damaging effects of moisture. Comments and contributions to any aspect and findings of the research report are invited.

DEVELOPMENT
TECHNOLOGY
UNIT

Working Paper No. 53

Dynamic Compaction of Soil for Low-cost Building Blocks

January 2000

David E Montgomery and Terry Thomas

Development Technology Unit
School of Engineering, University of Warwick
Coventry, CV4 7AL, UK
Tel: +44 (0) 2476 523122 Fax: 2476 41892
email: dtu@eng.warwick.ac.uk
http://www.eng.warwick.ac.uk/DTU/

DTU Working Paper No.53 **January 2000**

Dynamic Compaction of Soil for Low-cost Building Blocks

by David E Montgomery and Terry Thomas

Table of Contents

Abstract

This paper reports the results of experiments carried out on the process of dynamic compaction of stabilised soil blocks. The interest in this area has been fuelled by the previous research that has shown the dynamic technique of compaction has significant advantages over quasi-static compaction. During the experiments emphasis was placed on determining the wet compressive strength obtained after curing of the formed material. The results lead to a greater understanding of the different factors that affect this strength and suggest a means of predicting the strength without applying a destructive test. The “green” density of the newly formed material was found to be a good surrogate for its subsequent wet compressive strength.

The discovery that density was a good indicator for strength led to further investigation into the factors affecting the achieved density. It was noted that the moisture content of the soil mix was an important variable. The concrete and the soil literatures both give very inappropriate guidelines for a suitable moisture content; around 6% water by mass was found to be optimum for the production of stabilised soil samples. The energy used to compact the material is another key factor in generating a high density. The same energy applied via dynamic and quasi-static compaction was found to achieve similar densities, a disappointing result as larger scale tests indicated dynamic methods uses less energy for compaction than quasi-static methods. The lower efficiencies could however be a result of non-optimum dynamic compaction as the variables within the method of energy transfer, were not specifically optimised for the small (200g) samples used.

Finally, the different findings have been interpreted into a possible machine specification for the dynamic compaction of stabilised soil blocks. The most notable advantage that dynamic compaction has over quasi-static is its potentially lower machine cost. The impulsive blow to compact a soil sample does not exert massive forces for sustained periods of time during the compaction process. Consequently dynamic compaction has shown to be possible with thinner mould walls and using low-tech mechanisms than hydraulically assisted high-pressure quasi-static compression, yet to achieve similar levels of densification (and subsequent strength). It is therefore envisaged that a machine capable of producing high-strength building blocks can be made at a tolerable cost whilst also requiring a tolerable input of human energy into the production process.

Nomenclature

Aggregate: Pieces of crushed stone, gravel, etc. used in making concrete.

Brick: An object (usually of fired clay) used in construction, usually of rectangular shape, whose largest dimension does not exceed 300mm.

Block: A larger type of brick not necessarily made of fired clay, but stabilised in some way, sometimes with central cores removed to reduce the weight.

Bulk Density: Density calculated including any moisture present in the material.

Cement: Ordinary Portland Cement (OPC).

Clay: The finest of the particles found in soil, usually of less than 0.002mm in size and possesses significant cohesive properties.

Concrete: The finished form of a mixture of cement, sand, aggregate and water.

Dynamic Compaction: A process that densifies soil by applying a series of impact blows to it.

Fines: General category of silts and clays.

Gravel: A mixture of rock particles ranging from 2mm to 60 mm in diameter.

Green: Describing the state of material containing cement and water before it reaches the critical time, after which further plastic deformation hinders the final set strength.

Green Density: The density calculated immediately after ejection prior to any curing, drying or soaking.

Inferred Dry Density: The calculated density at ejection assuming no moisture is present in the formed sample, only solid matter.

Permeability: Describing a material that permits a liquid or gaseous substance to travel through the material.

Porosity: A measure of the void volume as a percentage of the total material volume.

Sand: A mixture of rock particles ranging from 0.06mm to 2 mm in diameter.

Silt: Moderately fine particles of rock from 0.002mm to 0.06mm in size.

Soil: Material found on the surface of the earth not bigger than 20mm in size, not including rocks and boulders and predominantly non-organic. If soil is to be used for building material it must not contain any organic material and it can be a natural selection of particles or a mixture of different soils to attain a more suitable particle distribution.

SSB: Stabilised Soil Block

Stabilised soil*:* Soil which has been stabilised (treated to improve structural characteristics) by using one or more of the following stabilisation techniques: mechanical, chemical and physical.

Religious dedication

Sometimes at the beginning of a publication one finds a dedication to a certain person or member of the family who has been an influence in the author's life either in general or specifically in generating the work in question. There is one person in my life that immediately springs to mind who is worthy of such a dedication. Furthermore, my experience with this person is not unique as millions of others have found him to be a great inspiration, comfort, guide and friend. "What's his name?" you may be asking yourself and, "Why haven't I heard of this incredibly influential person". The sad thing is that you probably have, but you have never accepted him as such or welcomed him into your heart and life. Well, now you have an opportunity to do just that. Please read on.

The man's name is Jesus and although he was born nearly 2000 years ago his testimony still remains and his power to save is just as great. "Save from what?" you may ask, sin and the consequences thereof, or more specifically, your sins and the consequences you face when you die. As humans we demand justice to be done, and justice will be done, but on a perfect scale and to a perfect standard. That leaves us all falling short and without hope when we come face to face with a holy God. But, God in his great love towards us send his only begotten Son into the world that the world through him might be saved. Jesus Christ died for you so that you would not have to be punished for what you have done wrong. You can be spared eternal punishment in hell and enjoy love and peace in the presence of God forever. Today the choice is yours. Reject God's free gift of love at your peril, accept it and who knows you too may have the joy of writing a dedication such as this someday. Please ponder the verses below and make your choice carefully, it will be the most important decision you ever make.

David Montgomery

"*For by grace are ye saved through faith; and that not of yourselves: it is the gift of God: not of works, lest any man should boast.*" Ephesians 2:8,9.

"*For God so loved the world, that he gave his only begotten Son, that whosoever believeth in him should not perish, but have everlasting life.*" John 3:16.

"*For whosoever shall call upon the name of the Lord shall be saved.*" Romans 10:13

"*He that believeth on him is not condemned: be he that believeth not is condemned already, because he hath not believed in the name of the only begotten Son of God.*" John 3:18.

"*Jesus saith unto him, I am the way, the truth, and the life: no man cometh unto the Father, but by me.*" John 14:6.

1. INTRODUCTION

This report describes the experiments carried out that investigate the characteristics of soil samples stabilised by different methods of compaction. The effects of such variables as water content, compaction energy, mixing delays and method of compaction are examined. Particular emphasis is given to the dynamic compaction method of soil stabilisation.

1.1 Motivation for this work

It is well documented that there is a massive and growing shortage of low-cost housing for the urban and peri-urban poor in developing countries. Several technological solutions that use local soil as the basic building ingredient have been proposed to help alleviate this problem. Currently two devices are widely available for the manufacture of stabilised soil blocks (SSB), namely high and low pressure block-making presses. The high-pressure (e.g. 10 MPa) press is capable of achieving sufficient densification to allow the quantity of stabiliser (cement) to be reduced to a low level (<6%) while still achieving adequate block properties. Low-pressure (<2 MPa) machines do not achieve such high densification and consequently the quantity of stabiliser in each block needs to be increased to a higher level (8-12%). However, the saving in cement when the high-pressure machine is used does not outweigh the significantly higher machine cost. What the market requires is a machine that achieves the same level of densification as the high pressure machine whist costing little more than the low-pressure one. *Dynamic* compaction of soil samples has in the past been shown capable of achieving high levels of densification and promises to provide a basis for designing the required machine.

1.2 Dynamic compaction

The basic principle behind dynamic compaction is the simple one of using a fast impact to transfer the energy of a falling mass into the object being hit. One can cite the parallel of the superiority of impact over pushing when driving a nail into a piece of wood. How the energy is transferred is however quite complex to describe as it depends on the characteristics of both the impactor (falling mass) and the object hit. Furthermore, the energy transfer will not be 100% efficient as other outputs such as noise, vibration and air resistance will subtract from the total energy delivered into the object being hit.

Dynamic compaction of building blocks is of interest to us for a number of practical reasons. We believe that it has some significant advantages over the existing method of slow squeezing (‘quasi-static compression’) of soil blocks that greatly outweigh the potential problems within the process. The first and perhaps most important advantage is that dynamic compaction doesn't require the same level of machine cost or complexity as high-pressure quasi-static compaction does. The absence of large levers, associated bearings and a hydraulic circuit represent a significant savings in machine cost. Furthermore the forces transmitted through the press are less, so that the machine can be made from thinner sections of steel and consequently be lighter and cheaper. The disadvantages of the process are that the safety implications of a falling mass are significant and the time taken to deliver a number of dynamic blows may be longer than a 'slowly' applied quasi-static force.

The instantaneous force generated during a dynamic blow can exceed, by a factor of up to 1000, the weight of the mass used for the blow and hence the force needed to lift it. We therefore have a sort of 'dynamic lever' capable of turning the pull of a human operator into a force of many tonnes. With these possibilities in mind, research into the dynamic compaction of soil for low-cost building began in the 1980s. This chapter reviews where the research in the field had reached prior to the commencement of this project and the current goals of the project.

1.3 Previous research at Warwick

Research has been carried out world-wide for many years into both the process of *quasi-statically* compacting stabilised soil blocks (SSBs) and that of *dynamically* compacting unconfined soil for the civil engineering industry. Unfortunately a bridge between these two spheres of activity did not seem to exist and there was virtually no information on dynamic compaction of constrained soil in order to produce building blocks. Other researchers at Warwick had noted this and hence dynamic compaction of blocks became an area of interest for the DTU. However almost no research into the technique has been identified elsewhere.

In 1984 Agas Groth carried out a final year student project investigating the potential of compacting soil within a mould by dropping weights onto it. The research included varying the mass of impactor and the drop height, but keeping the energy transfer and the material constant throughout the project. He aimed to achieve (with a 95% confidence) a block density of 1870 kg/m^3, a density that corresponded to a cured dry compressive strength of 3 MPa. With his particular soil type, he found he had to apply at least 1.63kJ to form a standard size block of $290 \times 140 \times 90$mm (mass ≈ 6.8 kg), i.e. about 240 J/kg. Using the technique, Groth subsequently built two houses in Botswana which after 15 years are still in good condition.

Bearing in mind the limitations he faced, some comments can be made on his findings. Several blocks must have been made, but there was no record of their characteristics after compaction, only the method of transferring the energy into the block. The recorded density is not defined as wet or dry density, which with 12% moisture would make a significant difference. However, the research did pave the way for future research to be carried out on the process of dynamic compaction.

Dominic Gooding undertook research for his PhD during 1993-6, looking at methods of soil stabilisation for low-cost building. He investigated how various aspects of quasi-static SSB production affect the output characteristics of the block. Factors such as mould wall taper, mould wall surface smoothness and whether single or double-sided compaction were used were examined and all found to have only a minor effect upon densification. He also generated a pressure/cement/strength relationship for his quasi-statically compacted samples.

However the main thrust of his work was an investigation into the dynamic compaction of approx. 1.6kg cylindrical samples. Like Groth he kept the energy transfer constant and varied the method of applying the energy to a given quantity of soil. The results indicated that there were optimum arrangements for transferring the energy into the soil with respect to the number of blows applied and the mass and velocity of the impactor used. They also showed that impact was a more energy-efficient method of compaction than slow squeezing.

None of the dynamically compacted samples that Gooding produced were stabilised with cement. Moreover the soil he used for his tests was recycled several times and that may have caused unintended variations in block properties (subsequent testing indicated the soil had progressively lost most of its fines content). After graduation, Gooding undertook a review of SSB production in 6 developing countries.

David Montgomery (the author) continued this research during another undergraduate project, whose emphasis was upon the design and development of a test rig to manufacture full-size dynamically compacted blocks. The design kept in mind the developing country environment in which such a block press might be both manufactured and used and employed an appropriately low level of technology. Applying a number of blows from a 36kg impactor produced several blocks with varying characteristics. Density was the main measurement made of the finished blocks as they were also unstabilised to conserve materials. The primary discovery was that the impacting process and Gooding’s findings could be extrapolated onto full size samples (approx. 8kg) with a high degree of confidence. Even at full size the impacted blocks required much less energy to form (to a specified density) than quasi-statically pressed blocks.

1.4 Current Goals

The present (PhD research) project can be divided up into two distinct parts; a materials science part and a manufacturing process one. The materials part started with a review of criteria for selecting a suitable soil and such a soil was selected for research purposes and for comparing dynamic compaction with quasi-static compaction whilst a number of variables where manipulated. The objective of this was to gain a better understanding of the material and to determine which variables are of greatest influence in the production of compacted samples.

The manufacturing process part of the project will take the findings from the material analysis and develop a systematic method for block production using the beneficial aspects of dynamic compaction. Variables discovered to be of importance will either be optimised to a single value or will be kept as alterable variables within the production regime where it is possible to do so. This part of the project will involve the design and manufacture of a machine capable of producing full-size dynamically compacted blocks. The design will be selected to ensure that it is appropriate for SSB producers in developing countries to manufacture and maintain.

Another aspect of the whole research project is to clarify the actual physical processes underlying dynamic compaction, as these are still poorly understood. Several process models have been suggested already but none of these have proved to be very accurate. Dynamic compaction of unconfined soils has been modelled as a one-dimensional problem (Scott & Pearce, 1975), but the theories within his paper do not cover compaction of confined samples as is the case in block production. Some analysis of the dynamic forces will be required for the machine design but full analysis of the dynamic compaction process may be outside the scope of this project.

2. VARIABLES OF SIGNIFICANCE IN THE MANUFACTURE OF SABILISED SOIL BLOCKS

The production of blocks suitable for low-cost building involves many different stages from the extraction of raw materials via block manufacture to the transportation of the finished blocks to the building site. The purpose of this paper is to look only at block formation (compaction), its associated constraints and the resultant block characteristics. Selection, extraction of raw materials, pre-processing of them, curing techniques and transportation constraints will be considered either briefly or not at all.

Blocks manufactured from different materials and by different methods have significantly different characteristics. Below is a table showing some common building materials and their respective key characteristics. Unfortunately the large range in values makes useful comparison difficult.

Property	Fired clay Bricks	Calcium Silicate bricks	Dense concrete blocks	Aerated concrete blocks	Lightweight concrete blocks	Stabilised soil blocks (SSB)
Wet compressive strength (MN/m^2)	10 to 60	10 to 55	7 to 50	2 to 6	2 to 20	1 to 40
Reversible moisture movement (% linear)	0 to 0.02	0.01 to 0.035	0.02 to 0.05	0.05 to 0.1	0.04 to 0.08	0.02 to 0.2
Density (kg/m^3)	1400 to 2400	1600 to 2100	1700 to 2200	400 to 900	600 to 1600	1500 to 1900
Thermal conductivity (W/m°C)	0.7 to 1.3	1.1 to 1.6	1.0 to 1.7	0.1 to 0.2	0.15 to 0.7	0.5 to 0.7
Durability under severe natural exposure	Excellent to very poor	Good to moderate	Good to poor	Good to moderate	Good to poor	Good to very poor

(International Labour Office, 1990)

Desirable block characteristics are:-

- a high wet compressive strength – to permit both single and multi-storey construction,
- a low moisture movement – to lessen expansion/shrinkage potential,
- a low density – lighter blocks to make construction easier,
- a low thermal conductivity – for greater dwelling comfort and
- a high durability – securing a long-term investment.

The last column above shows the characteristics for SSB’s and the very large ranges that each characteristic has for SSB’s. During the research reported here, the wet compressive strength was taken to be the key characteristic and production sought to maximise the strength achievable with tolerable physical effort and machine cost.

During the production of an SSB many different variables will be of importance. We will regard as ‘independent’ those variables that can either be controlled by the operator, such as moisture content, or are a result of environmental conditions, such as temperature and can be monitored. The ‘dependent’ variables are values that are determined by interactions between the independent variables. From the viewpoint of the SSB manufacturer, several of the independent variables, including cement content, are of major concern; whereas to the end-user only the dependent variables, such as durability and strength, are of any great interest. For the purposes of this research both the dependent and the independent variables were monitored. The research endeavoured to identify which independent variables have the greatest effect on the dependent ones and what might be the optimum values of controllable variables. The aim of this was to minimise the demand on the manufacturing inputs without compromising the desired block characteristics.

2.1 Dependent variables of interest

This section discusses five key *dependent* variables, identifying values for them that are achievable and desirable in the production of SSB’s. Since they are associated with different stages in the production and use of a block, they are presented in ‘chronological’ order.

De-moulding force – After compacting an SSB in a mould it must be successfully removed from the mould without damage. Moulds that come apart in some way to easily release the finished block are more complex to manufacture and take more time to open and close than simple straight-sided moulds: they are therefore unattractive both from the complexity and time aspects. The compacted material can instead be pushed up out of a cheaper fixed mould, however a certain de-moulding force will needed to overcome the cohesion/friction between the SSB and the mould walls. The size of this force will depend on the mould’s surface finish, the moisture content of the material, the level of compaction achieved and the method with which the energy was transferred into the SSB. For manufacturing purposes we desire the de-moulding force to be as low as possible to make the production of the block easy. Currently a full-size block quasi-statically compressed with 10 MPa requires a de-moulding force of slightly under 2 tonnes. For a human to generate such a force a significant leverage must be employed. Reducing de-mould forces well below this 2 tonnes would be advantageous to both the machine designer and the block manufacturer.

Ease-of-handling – A freshly formed block has a low ‘green’ strength and must be handled with care. If the block had a greater ‘ease of handling’ there could be a lower rate of block breakage both before and after curing. Furthermore high ease-of-handling would permit green blocks to be stacked immediately upon demoulding, which in turn helps to reduce the floor area required for curing. Stacking also reduces the surface area for moisture-loss from the freshly formed block and thus helps to ensure a good curing regime. This ease-of-handling of a block is not a characteristic that can readily be measured directly. However it correlates with green strength which can be measured. A penetration test is usually used to determine the green strength of a formed block. This involves pushing a rod a specified distance into the surface of the block and recording the force required (or conversely measuring the penetration distance resulting from using a specified force). The green strength of the block will not depend on its cement content as the cement particles will not have had time to hydrate and add any strength to the material. The green strength is largely dependent on the particle size distribution (soil type), the moisture content and the level of compaction achieved. These factors work together to give the material the cohesion that enables ease of handling.

Green density – In the same way that the green strength is of interest to the block producer, the green density is another characteristic that can be easily and quickly measured immediately after production. This measurement can serve two purposes, firstly it checks that the block passes a certain standard prior to curing and secondly it can be part of a longer term feedback loop to improve control of the manufacturing process (discussed in Chapter 4). Where a known amount of material has been used to generate a sample, measurements are taken on the overall size of the sample after compaction has taken place. Several different density calculations can be carried out (as is discussed later) but usually either the ‘bulk’ or the dry density is recorded. The bulk density is always higher than the dry density due to the presence of water in the block and so it is important to record which of the two has been calculated. Bulk densities between 2000 and 2200 kg/m^3 are considered to be excellent for SSB manufacture (Houben & Guillaud, 1989). A number of the process inputs (independent variables), but most specifically the energy transfer and the method of compaction, influence the green density of the block. Other factors such as moisture content and cement content have a lesser effect on it.

Wet compressive strength – This characteristic is high on the list of user priorities. Existing low-cement SSB’s manufactured by low-pressure compaction have compressive strengths adequate for the majority of low-rise structures *provided that* water penetration is kept to a low level. Using an external render or paint, both of which require regular maintenance, will reduce the moisture penetration considerably. However, when saturation of SSBs has occurred it has often proved to be too harsh for the material to withstand whilst maintaining a load: surface flaking or even collapse has followed. If the wet compressive strength can be improved then environmental effects such as running water will not cause such early failure to occur in the building material. The wet compressive strength is measured by placing a cured and water-soaked sample into the jaws of a compression machine and slowly squeezing the sample until the maximum load applied is reached. After the maximum the sample has been crushed and will no longer support a load of that magnitude again: it has been tested to destruction. The predominant factors that affect the wet compressive strength are cement content and the level of compaction achieved during moulding. The strength achieved by the cement content depends in turn on the moisture availability for cement hydration and curing regime applied to the finished block. Wet compressive strengths of over 2 MPa are considered to be excellent for SSB’s (Houben & Guillaud, 1989).

Durability – The most desirable of all the dependent variables is durability, taken to be the measurement of how long the material will survive before environmental attack jeopardises the integrity of the building material or renders it unsightly. Unfortunately no measurement of SSB durability is currently available, as real long-term tests need to be carried out. Current literature describes durability via the terms ‘poor’ to ‘excellent’, hardly a quantitative approach. Other research currently underway at Warwick is exploring durability. However it is generally accepted that the durability of stabilised soil blocks is closely linked to their wet-compressive strength: blocks with higher wet-compressive strengths last longer.

2.2 Controllable independent variables

This section briefly describes the different controllable variables that are involved in the SSB manufacturing process. A summary will be given of the ranges used for the controllable variables in previous work and mechanisms by which they might affect the dependent ones will be outlined.

Soil type – In the field this can vary considerably and it is known that some soils are more suitable than others for the production of SSB's. The United Nations guideline for suitable soils for SSB production requires "a well graded soil consisting of 75% sand, the remainder being fines of which more than 10% is clay". Soils with more than 30% clay will be very expansive with the addition of water and hence will exhibit excessive dimensional variation with the seasons. To counteract this a larger degree of stabilisation than normal is necessary, either by extra compaction or by increasing the amount of cement. Very high clay contents (over 50%) are unsuitable for stabilisation with cement, so either lime has to be used or sand must be mixed with the soil to reduce the clay fraction. There is therefore a literature about soil selection for block making. For the purposes of the research reported in this paper, the soil type has not been treated as a variable but instead been kept constant. All the experiments have used the same soil, one selected to be quite suitable for SSB manufacture.

Moisture Content (MC) – Different sources gave conflicting information about the selection of suitable moisture content for the process of SSB manufacture. A drop test is usually given by the SSB manufacturing texts as an approximate method for checking that the MC is suitable. For research purposes a better definition is required.

The *soil mechanics* literature indicates that maximum density is achieved if compaction occurs at what is termed as the Optimum Moisture Content (OMC) but we should rename *Density Optimising Water Content* (DOMC). Compaction tests need to be carried out to determine what the DOMC is for each soil used: values around 11% are typical for the soils of interest.

The *concrete* literature (Neville, 1995) - which however effectively assumes use of a 'soil' having a total absence of fines - indicates that for ideal compaction the Water/Cement (W/C) mass ratio should be extremely low. For practical levels of compaction, W/C ratios of 0.3 to 0.5 normally yield the greatest strength. For the low cement contents (<6%) characteristic of SSB manufacture, such ratios corresponds to around 2% water content. Thus the DOMC and W/C criteria give widely differing values for optimum water content, and a compromise needs to be made. A further complication with SSBs is that too high a moisture content (say >9%) so reduces the "green" strength of the block that its ease-of-handling becomes inadequate and post-compaction breakage rates become intolerable.

Cement content – Cement is usually the dominant variable cost in SSB production, so the reduction of its quantity is very desirable. How much cement is necessary depends on three factors, the clay content of the soil used, the degree of compaction during moulding and the required wet compressive strength of the finished block. The higher the clay content the more cement is required and conversely the higher the compacting effort (as measured by the densification achieved) the less cement is required for adequate stabilisation. If a higher wet compressive strength is necessary then either the compacting effort or the cement content will need to be increased. To some degree increase in the one can be traded for a reduction in the other - a fact that has driven a trend towards increasing the moulding effort in SSB manufacture. Previous stabilised soil research (Rigassi, 1995) has indicated that cement contents below 2 or 3% will not actually enhance the wet compressive strength or improve stabilisation. Consequently 5% by weight is probably the smallest amount of cement practical to employ for SSBs.

Energy transferred – This is a highly significant variable as it can have a marked effect on the final material strength regardless of the route by which the energy is applied to the material. Dynamic compaction has consistently proved to be more efficient in improving SSB properties than quasi-static compaction using the same energy transfer. Previous experiments with dynamic compaction kept the energy transfer, or the energy transfer per unit mass, constant. This generated samples with varying characteristics from the same energy transfer. Producing samples with fixed characteristics via different compaction methods and energies was not carried out. However, it is to be expected that different amounts of energy will be required to produce samples with similar characteristics via the two different compaction methods (dynamic and quasi-static).

Quasi-static compaction involves a certain pressure being applied to the ends of the material confined in a mould of either rectangular or circular cross-section. The pressure can be applied in one or more cycles and the speed of compression can also be varied (5-100mm/min). (Speed of compression is only varied for convenience and accuracy to ensure a good result in a short cycle time.) Previous research indicates that the increase in compaction achieved by having more than one compression cycle is so small that it is not an efficient use of energy (Gooding, 1993), five pressure cycles only increase the density by less than 2%. Generally the applied pressure is calculated in MPa and samples created between 8 and 12 MPa are of most interest for comparison with dynamic compaction.

Dynamic compaction uses the energy from a falling mass to compact the sample. The process entails a number of variables but these can easily be summarised as the number of blows and the impactor momentum per blow. The number of blows applied to the sample can be varied within a pre-determined optimum range of 8 – 32 blows. The impactor momentum ($mv = m\sqrt{2gh}$) and the impact energy ($e = mgh$) both depend on the mass m of the impactor, and the height h through which it is lifted. The lifted height will control the final velocity of the impactor at contact with the soil. Previous research showed that impact velocities of over 2 m/s were potentially damaging to the compacted material as the initial compressive shock wave could reflect at the bottom of the mould into a tensile wave, whose subsequent travel upwards can shatter the sample.

Mould-wall thickness –The mould-wall thickness required for dynamic compaction is different to that needed for quasi-static. It is found that for comparable energy transfer and densification, the forces applied to mould walls during dynamic compaction are smaller and of much shorter duration than those occurring during quasi-static compaction. If mould-wall thickness is chosen on the basis of achieving a particular strength safety factor, the dynamic moulds can be significantly thinner and therefore lighter than quasi-static compaction moulds. For example for highly-densified full-size blocks, the respective mould-wall thicknesses might be 8 mm and 25 mm respectively. This difference is economically significant, since one of the barriers to the take-up of quasi-static presses operating up to 10 MPa pressures has been their excessive weight and cost.

Size and shape – A standard block size is 290 × 140 × 90 mm whereas the standard sample size for compression testing is either a 150mm or a 100mm cube. Previous research has also been carried out on 100mm diameter cylinders with an approximate height of 100mm. All these different sizes and shapes will have an effect on the apparent characteristics of the finished sample. For research purposes it is inconvenient to manufacture full size blocks to check every little variable and characteristic. Furthermore the dynamic compaction of a full size block requires strict safety procedures to be followed and these become much less stringent if the sample size is smaller. For these reasons the research was performed using smaller size samples. Extrapolation of findings to full-size blocks is not straightforward, however the *ranking* of alternatives at one scale is likely to be the same as the ranking at a different scale.

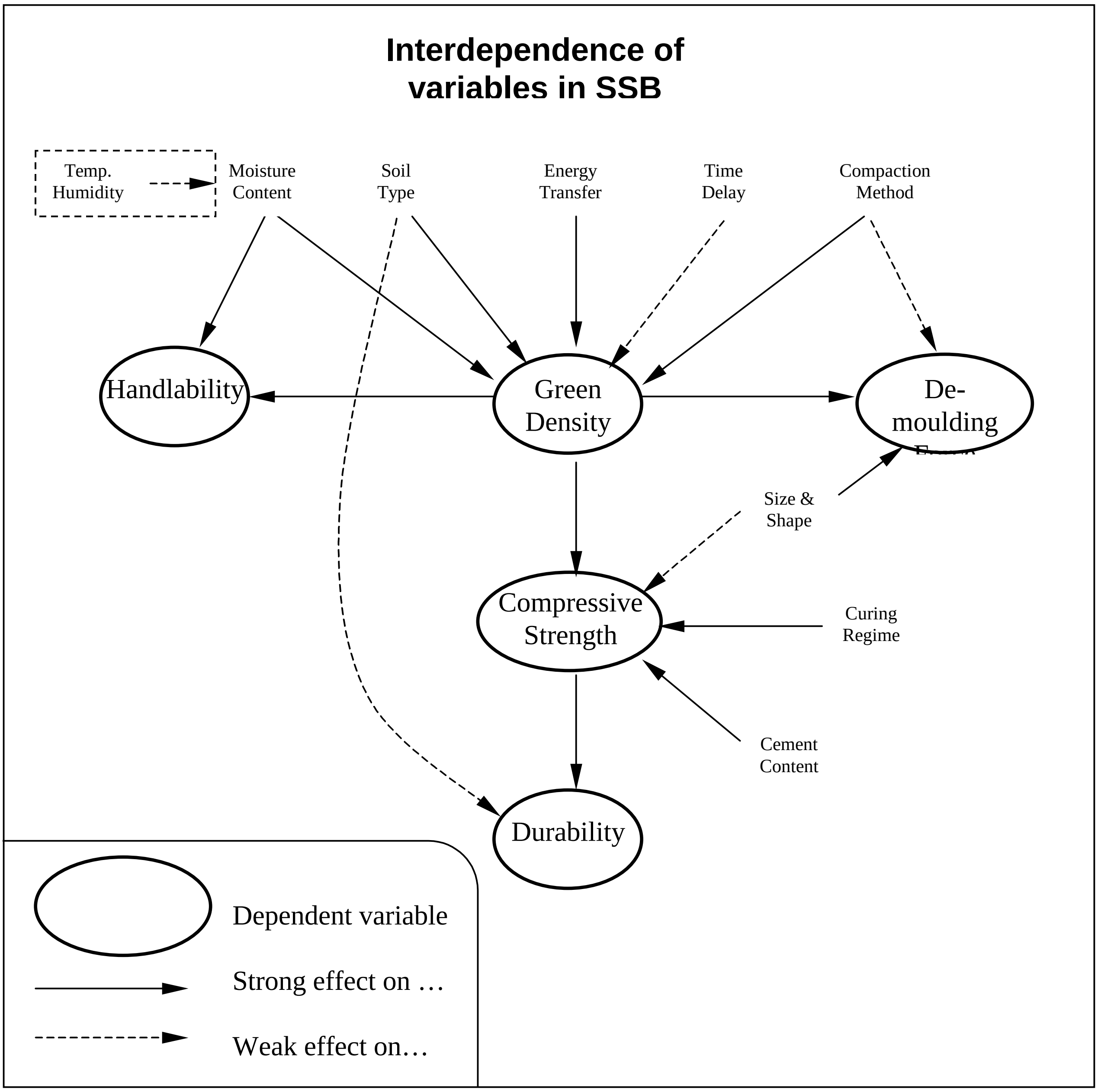
Delay before compaction – As soon as moisture is added to the dry soil/cement mixture the cement will begin to react with the water. As the cement begins to hydrate the moisture levels in the soil available for lubrication becomes less, hindering the compaction process. Meanwhile the crystallisation that is beginning to occur with the cement after the critical period has passed (roughly defined as 15 minutes after adding water) further hinders the compaction. Following the manufacture of a batch of material some parts of the batch are made into blocks before other parts. This variable delay between mixing and compaction has an effect on the ejected density. The order of production within a batch will thus have an effect on the final sample characteristics and while this is not large it is a factor that requires addressing. Indeed it is very useful to know whether a significant loss-of-strength penalty is incurred when a period as long as say 1 hour elapses between mixing a batch and using all of it.

Curing period – Ideally the compacted samples should be left to cure for an adequate time in an environment of nearly 100% relative humidity. The normal period for concrete curing is 28 days, although test data also records 3 and 7 day strengths as well. In reality SSB production seldom includes block curing for 14 days: 7-day or even shorter curing is more normal. Water scarcity and poor understanding of curing concrete often leads to blocks being left out and allowed to dry in the open. This is a very poor production practice as keeping blocks moist in a humid environment will improve their final strength significantly. The majority of previous experiments have been carried out under laboratory conditions, typically at 20°C and with a low relative humidity. During this research samples were cured in sealed bags containing water-saturated air.

2.3 Summary of variables and their interactions

Each independent variable has *some* effect on every dependent (output) variable. Below is a schematic chart that attempts to illustrate the this dependence of the outputs on the inputs. The term ‘significant’ is used to denote a output-to-input sensitivity commonly exceeding unity.

In order to check this interdependence a significant number of different samples needed to be manufactured. As full-size block production was not viable, for reasons rehearsed above, a smaller sample size had to be chosen. In fact a small sample, an approximately 200 gram cylinder, was selected. This permitted fairly rapid production even up to quasi-static pressures of over 10 MPa using existing laboratory facilities. An existing mould with an internal diameter of 54.4mm was found and was used as a standard for other moulds. The sample height was chosen to give the same ratio of mould-wall surface area to compaction (top surface) area ratio as a full-size block has.

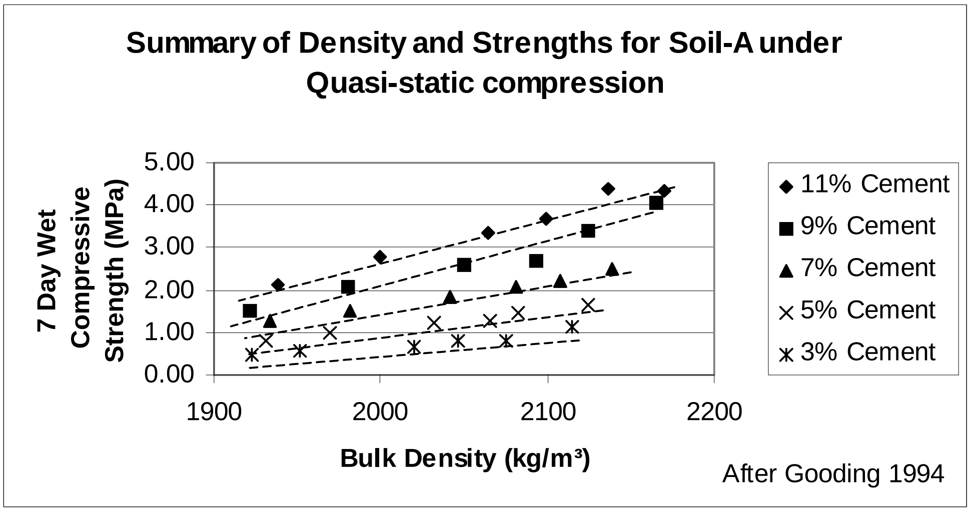


3. FACTORS AFFECTING THE STRENGTH OF CURED BLOCKS

One of the most important characteristics of an SSB is the durability of the finished product. Durability may, as discussed earlier, be thought of as how long the block will be able to support a load whilst experiencing normal environmental attack. Since durability measures do not currently exist for SSB's and the durability is closely linked with compressive strength, then determining the strength of an SSB is probably the best available indicator for its durability. Unfortunately compressive strength can only reliably be measured by rather complex and destructive testing of blocks prior to their incorporation in a wall. This is inconvenient both for research and for quality control in manufacture. Moreover compressive strengths of materials such as we are using here are inherently somewhat variable. As we shall see later, *block density* (which can be measured non-destructively) may sometimes be considered a surrogate for wet compressive strength in research work. In production a simpler modulus-of-rupture (flexural) test may be used instead of a crushing test although both are equally destructive in nature.

The flow chart in Section 2.3 showed the main factors that affect the strength to be the cement content, the curing regime and the green density. If the cement content and the curing regime are kept constant then the only factor affecting the strength should be the density achieved during moulding the block. We already know that the cement content and the curing regime have a significant effect on the final strength, but we don't know fully how other factors affect the green density.

Gooding established that the 7-day wet compressive strength of pressed blocks is directly related to the cement content and the compaction pressure applied to form them. He developed an equation to determine the expected strength if a known pressure and cement content were applied to a specific soil with a moisture content of 8%. Below is a summary graph of his results for wet compressive strength tests on 50mm diameter x 100mm long samples made from his 'soil-A'.



E_D_E_QS_gooding

Although cement content is not one of the variables that have been addressed in this paper, Gooding's work clearly illustrates the significant gain in strength that can be achieved by adding extra cement. The graph above also illustrates that increasing moulding pressure increases both density and strength. Each locus, representing a particular cement content, has 6 points representing moulding pressures of respectively 1, 2, 4, 6, 8 and10 MPa. Not unexpectedly the lowest pressures resulted in the lowest densities and strengths. Another feature visible from this graph is that the sensitivity of the wet strength to cement content is much higher than it is to moulding pressure.

A number of new experiments were carried out, in each of which the wet compressive strength of the sample was measured after compaction and subsequent curing. The effect on strength of varying (a) moisture content, (b) compacting pressure *or* number of fixed energy dynamic blows and (c) time delay before moulding was measured. Other factors such as mould wall thickness and energy transferred were also investigated for their effect upon green block density but not upon cured block strength.

Certain variables were kept constant (at realistic values) during the production of samples, partly because there are too many variables to consider and partly because some of them have already been investigated. The cement content was set at 5% by weight. Rather than vary the type of soil a large batch of stable (and reproducible) soil was manufactured that could be used for all of the experiments. This research soil is gap graded with 80% builder's sand and 20% kaolin clay and is called Soil-B. All samples were cured for a total of 7 days in a humid environment that included a 1-day soaking period followed immediately by wet crushing. Unless otherwise stated the selected sample size was a cylinder of 54.4mm diameter with a dry soil mass of 200g. Moulding and strength testing were conducted in the laboratory at temperatures around 20^{o}C and relative humidity levels around 60%.

3.1 Inherent variability of strength

Two seemingly identical concrete samples will have slightly different strengths. The different arrangement of the particles and the cementitious bonds that join them create a variation in the strength of the material. In order to determine this inherent variability an experiment was carried out in which almost every independent variable was held constant and the coefficient of variation of wet-strength was estimated. This indicated the inherent variability of strength so that future results could be assessed more accurately and sample sizes chosen wisely. For practical reasons, one input variable was allowed to vary, namely the time elapsed between mixing the soil mortar and compacting it. The results were processed in a way that allowed this variability to be compensated for.

During the experiment 35 samples were produced, 18 via quasi-static compaction and 17 via dynamic compaction. The quasi-statically formed samples were compressed to 10MPa and the dynamically compacted samples received 16 blows of a 5kg impactor falling through 200mm. The quasi-static samples received approximately 100J of compaction energy whilst the dynamic samples received 157J (i.e. 0.2m×9.81×5kg×16 blows). Both sets were manufactured at 6% moisture content and in the 54.4mm diameter mould with an 8mm wall thickness. A batch of material was made up to produce three 200g samples and the order of production of each sample within the batch was recorded.

Each batch consisted of a 'first', 'second' and 'third' sample manufactured at different times. Six of each were produced for each (dynamic and quasi-static compaction) process. The strengths of the firsts, seconds and thirds can all be compared and analysis carried out on the results.

Below is a table of results from the dynamic compaction tests.

Table 3.1a

Number	Position	Bulk density			7-day wet strength		
of samples	in Batch	mean kg m^{-3}	s.d. kg m^{-3}	Coef of var %	mean MPa	s.d. MPa	Coef of var %
5	1st	2140	8	0.38	2.38	0.14	6.1
6	2nd	2131	5	0.26	2.23	0.15	6.5
6	3rd	2118	5	0.24	2.12	0.18	8.3

E_D_E_DS_den-ref

It is immediately clear that there is a very low variation in the bulk density (under 0.5%) but a more significant variation in the wet compressive strength (6 to 8%). From this we infer that either strength is highly sensitive to density or that there is an inherent variation in the compressive strength of identically formed materials.

Below is the table of results from the quasi-static compaction tests.

Table 3.1b

Number	Number	Bulk density			7-day wet strength		
of samples	in Batch	mean kg m^{-3}	s.d. kg m^{-3}	Coef of var %	mean MPa	s.d. MPa	Coef of var %
6	1	2067	9	0.44	1.76	0.09	5.3
6	2	2054	13	0.65	1.61	0.13	7.8
6	3	2050	9	0.44	1.63	0.11	7.0

E_D_E_QS_den-ref

The same feature can be noted in these results: a very low variation in the density and a larger variation in the compressive strength. Despite the lower densities and strengths of these samples, the coefficients of variation of strength are similar to those for the dynamically compacted samples. This suggests that variability in strength is not closely related either to the density achieved or to the method of compaction. The above results indicate that the strength coefficient of variation will be not more than 8% under normal conditions or not more than 5% if the average of 3 samples is taken. Consequently a sample size of 3 was adopted and any averaged change in strength of more than 10% was be considered to be significant - i.e. the result of a change in an input variable.

3.2 Effect of moisture content on strength

Early on in the project some full-size blocks were manufactured using a “Bre-pak” high-pressure block-making machine. Three blocks were manufactured from old ‘soil A’ at each of the moisture contents: 4, 5, 6 & 7%, making a total of 12 blocks. All the blocks were stabilised with 5.2% cement by weight and their production and curing cycles were virtually identical. Their measurements were taken at ejection and the bulk density was calculated for each block prior to curing it in a humid environment for 7 days. To make the compression tests comparable with standard concrete tests each block was cut in half and each half was cut to the size of a 100mm cube. This gave 24 samples for compressive testing instead of 12. Then each half of the same block was subjected to different tests. One half was soaked for a day whist the other half was left to dry out for a day prior to crushing. This gave a ‘wet’ and ‘pseudo-dry’ compressive strength test for each of the blocks.

It should be noted at this time that blocks continue curing after being removed from the humid curing environment. This resulted in a curing period of one day more than intended. A block crushed after 7 days curing and one day soaking or drying will have effectively been curing for 8 days total because the core moisture will not evaporate entirely in 24 hours. The concrete literature (Akroyd, 1962) suggests an adjustment of around -7% should be applied to an 8-day strength figure to generate the corresponding 7-day strength. The figures given below have been adjusted in this way to standardise them into 7-day wet compressive strengths.

Table 3.2a

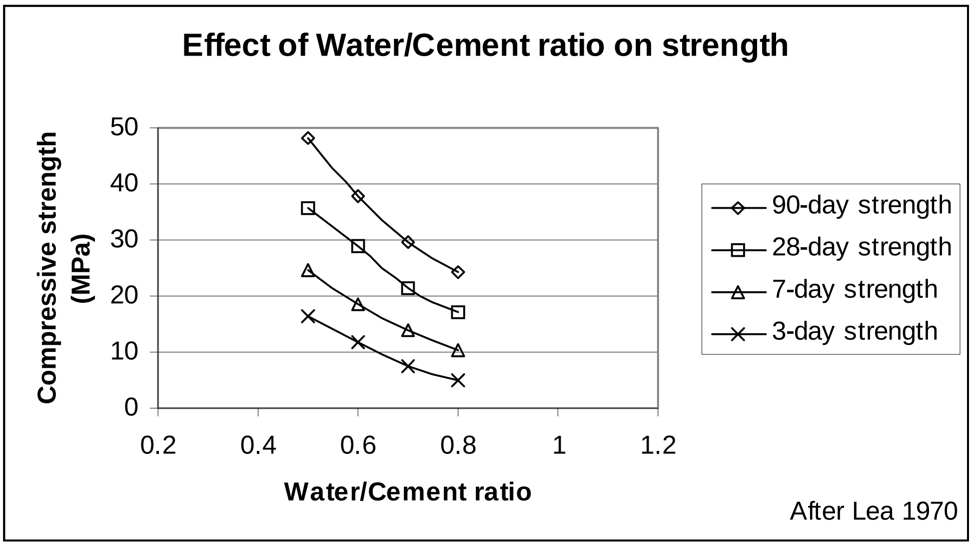
Moisture content % by wt	Bulk density mean kg m^{-3}	s.d. kg m^{-3}	Coef of var %	7-day wet strength mean MPa	s.d. MPa	Coef of var %
4	1943	3	0.15	0.86	0.04	4.41
5	1971	14	0.72	1.40	0.03	2.40
6	1995	2	0.09	2.10	0.08	3.87
7	2020	11	0.56	2.32	0.10	4.13

E_D_E_QB_strength

The above results seem to be consistent with the soils literature in that as the moisture content is increased up towards the DOMC the density also increases. These results follow that trend very well since 7% moisture is still below the DOMC for the soil (discovered to be between 9-10% moisture). The greater variability of wet compressive strength compared to that of bulk density can be seen here again. If we were to ignore the effect of the cement present we would be excused for thinking that the increase in strength is caused solely from the increase in bulk density resulting from compaction closer to the DOMC. This assumption cannot be made and consideration of what is happening between the cement and the water must also be included.

The cement literature suggests that a Water/Cement (W/C) ratio of between 0.3 – 0.5 is the best for concrete strength, provided that the mix is "fully compacted" (i.e. that all air is expelled). The above results unfortunately do not confirm or contradict that statement. The increasing *density* at higher water contents brings about an increase in strength; this may outweigh the loss in strength caused by the W/C ratio being too high. An alternative explanation is that (given the high water-affinity of the clay fraction of the soil) the higher moisture content mix has more free water to hydrate the cement and this, combined with the increase in density, helps to generate a higher compressive strength.

Data taken from (Lea, 1970) on the water-cement ratio and resulting strength (of sand-cement-water mixes containing no fines) is shown below.



E_D_MCTesting

For the experiments done only one of the above W/C ratios could be matched. One sample had a cement content of 5% and a moisture content of 4% giving a water-cement ratio of 0.8. It is obvious from the graph above that reducing the W/C ratio below this 0.8% should increase the strength, whereas the experimental data presented earlier showed strength *increasing* as W/C ratio was raised above 0.8. It must therefore be assumed that the W/C ratios recommended by the concrete literature are inappropriately low for soil stabilisation - either because of the presence of fines in the mix or because of the unusually low cement levels used in SSB manufacture.

In order to try and illustrate the effect of the water on cement curing for very low moisture contents (supposedly best for concrete strength) the 'pseudo-dry' strength was also measured. 'Pseudo-dry' is defined as removing the sample from the humid curing environment and allowing the free water to escape to the atmosphere in the laboratory for 24 hours. This does not dry the block entirely as much of the core moisture is still present. The table below shows the compressive strengths of these samples.
Comparing Tables 3.2a and 3.2b enables us to compare 'wet' and 'pseudo-dry' 7-day compressive strengths over a range of water contents. One normally expects a wet block to be weaker than a dry one, because there is more lubrication between particles

and a slip plane can develop more easily causing failure at a lower stress level. The above results are initially contrary to this assumption. For the 4% M/C case the wet strength (0.86MPa) was actually higher than the corresponding pseudo-dry strength (0.77MPa). This difference is statistically significant (>95% as the difference in the sample means exceeds twice the standard error of difference). One possible explanation is that the extra water available during the soaking process preceding the wet-strength test so advanced the cementitious reaction that it overcame the loss in strength due to the sample being wet. This suggests that the cement had been starved of moisture and more moisture would have been of greater benefit.

Table 3.2b

Moisture Content %	'Pseudo-dry' compressive strength Mean MPa	s.d MPa	Coef of var %
4	0.77	0.02	2.79
5	1.36	0.11	8.08
6	2.12	0.18	8.29
7	2.48	0.16	6.64

E_D_E_QB_strength

For the 4% MC condition the significance test is as follows:

Standard error (wet) = $\frac{0.04}{\sqrt{3}} = 0.023$

Standard error (pseudo-dry) = $\frac{0.02}{\sqrt{3}} = 0.011$

Standard Error of Difference (SED) = $\sqrt{0.023^2 + 0.011^2} = 0.025$

Difference of Means (DoM)= 0.86 – 0.77 = 0.09

DoM/SED = 0.09/0.025 = 3.6 (3.6 > 2 $\therefore$ significant)

Penetrometer tests were also undertaken to determine the surface strength of 'green' blocks. Greatest penetrative resistance, and hence the greatest ease of handling, was found at moisture contents between 4-6%. Penetrative resistances above 0.4MPa were achieved in this moisture range but the penetrative resistance was found to reduce significantly where the water content was increased above 6%. Consequently, 6% water was selected for many of the experiments carried out.

3.3 Effect of 'Effort of moulding' on strength

Densification of material increases the effectiveness of stabilising additives like cement. This densification can take place in many ways, but for the purposes of this paper only two methods will be considered: quasi-static and dynamic compaction.

Quasi-static compaction can be most easily defined by the peak pressure applied to a block causing densification, whereas dynamic compaction can more easily be defined by the number of blows applied and the momentum of each blow. (For purposes of directly comparing the two methods however a common measure may be calculated, namely moulding energy applied per kilogram.) This sub-section will be looking closely at the difference between the two types of effort applied and how they respectively affect the strength of the sample created.

3.3.1 Quasi-static compaction

Different compacting pressures have been selected in the past for the stabilisation and densification of soil blocks. Low-pressure machines apply between 1 and 3 MPa via a lever mechanism, whilst high-pressure machines would apply between 8 and 16 MPa using a lever and a supplementary hydraulic circuit (Houben et al., 1994). The experiment carried out here looked at the effect of pressures within the high-pressure range on the wet compressive strength of samples produced.

Below is a table showing the summary of results from the compression tests of small samples (200g) produced by quasi-static compaction in a cylindrical mould with 8-mm walls and with a soil moisture content of 6%. The soil used was Soil-B stabilised with 5% cement.

Table 3.3a

Compacting Pressure MPa	Bulk density mean kg m^{-3}	Bulk density s.d. kg m^{-3}	Bulk density Coef of var %	7-day wet strength mean MPa	7-day wet strength s.d. MPa	7-day wet strength Coef of var %
8	2047	12	0.57	1.48	0.05	3.60
10	2067	14	0.69	1.75	0.23	13.07
12	2102	15	0.69	1.92	0.30	15.78

E_D_E_DS_density2

As the results from Gooding showed (Chapter 3) the strength increases as the pressure is increased for a given sample and these results also follow the same trend. What is interesting to note is that a 50% increase in pressure (from 8 to 12 MPa) yields only a 30% increase in strength, giving a mean sensitivity of strength to pressure of 0.65. (Gooding, operating with a slightly different range of variables, found doubling the strength requires a tenfold increase in compacting pressure, i.e. a mean sensitivity of only 0.3.) This is a fair return providing the machine is designed to withstand the higher pressures. The significant variation of strength for an insignificant variation in density can again be seen in these results.

3.3.2 Dynamic Compaction

Previous research had already indicated that dynamic compaction was more efficient at increasing the density of a sample for the same energy transfer, but that research did not include any strength testing of the samples produced. Consequently a series of small samples (200g) was produced by dynamic compaction in a cylindrical mould with 8-mm walls and with a soil moisture content of 6%. The soil used was Soil-B stabilised with 5% cement. These were manufactured in the same manner, as the quasi-statically compressed samples described in the previous section. The method of densification and the energy transfer were the only variables altered. The aim was to try and achieve the same density via different methods and to see if the resulting strength was significantly different.

One of the negative aspects of the process of dynamic compaction is the large number of blows that often need to be applied to the sample to achieve sufficient densification. If this number could be reduced then the processing time for making a block would be shortened. Clearly this is desirable, but sacrificing strength to accomplish it is not acceptable. Direct comparison with quasi-static compaction suggests that 16-20 blows should be sufficient to achieve the same strength as using 10MPa quasi-static compaction. This experiment set out to confirm this. Three blocks were manufactured at each of the following number of blows: 8, 12, 16, 20 & 24, making a total of 15 samples. Each blow is from a 5kg impactor falling through no more than 200mm. An analysis of energy transfer will be considered in later chapters.

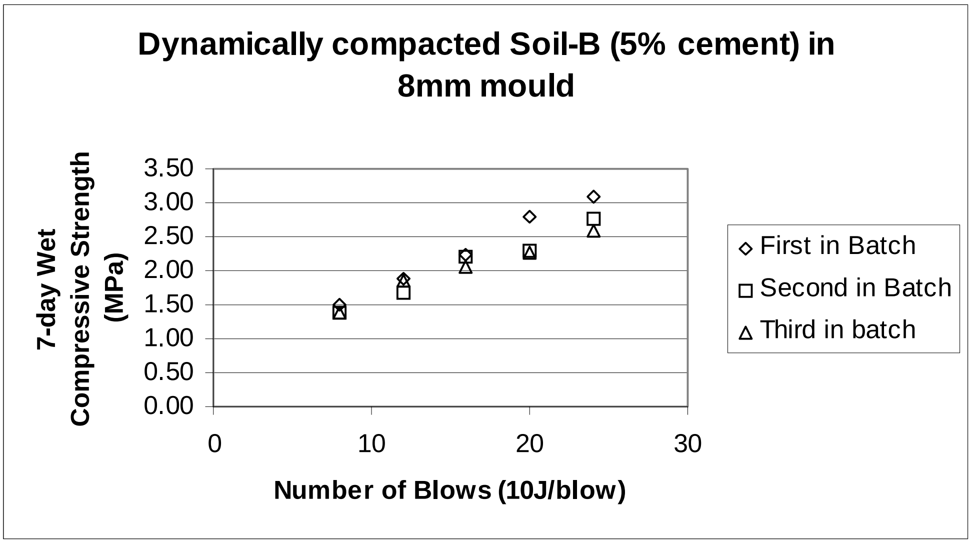
Table 3.3b

Number of Blows	Bulk density mean kg m^{-3}	s.d. kg m^{-3}	Coef of var %	7-day wet strength mean MPa	s.d. MPa	Coef of var %
8	2053	18	0.86	1.43	0.07	4.76
12	2097	13	0.61	1.81	0.11	6.19
16	2113	16	0.75	2.17	0.10	4.79
20	2133	13	0.59	2.44	0.29	11.98
24	2162	19	0.86	2.81	0.25	9.06

E_D_E_DS_density2

As the blow number, and hence moulding energy, are increased the bulk density and wet strength also increase. As noted previously, density is less variable than strength. Increasing the number of blows applied by 50% (from 8 to 12 or from 16 to 24) generates an increase in strength between 25 to 30% - giving a mean sensitivity of strength to effort of 0.6. This is a good return especially considering the machine design does not need to be altered to accommodate the higher number of blows.

These results are represented in the graph below with indicating the order of production for each batch. The graph shows another feature that may be of interest. The strength achieved seems to be related to the position within the batch, with the first sample produced being the strongest and the last generally the weakest. This trend seems to become more pronounced as the number of blows is increased, possibly due to the longer production time necessary for more blows to be applied.



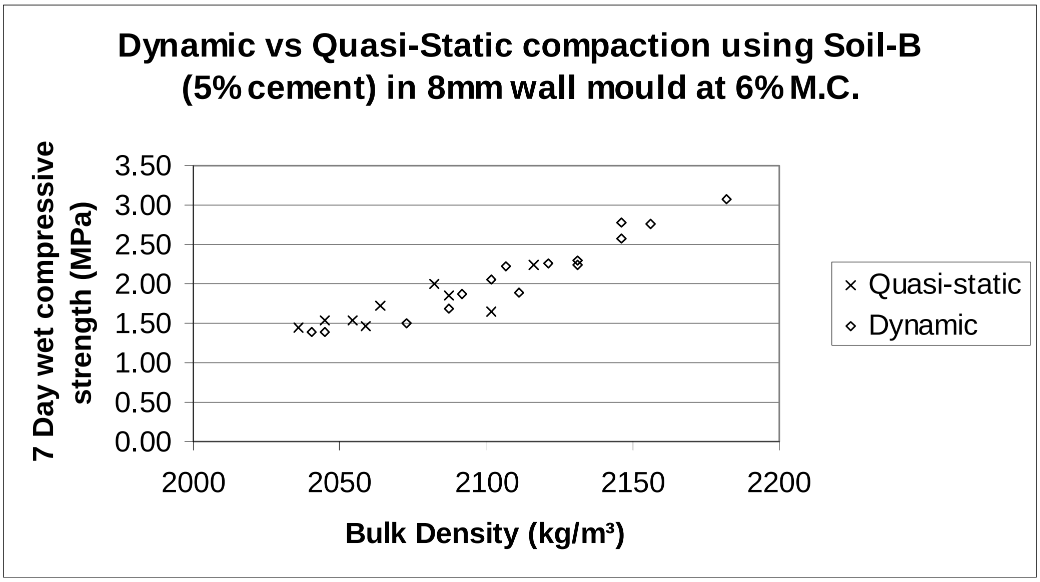
E_D_E_DS_density2

A direct comparison between the dynamic and quasi-static compaction results, show that dynamic compaction is significantly better than almost all of the compacting pressures. The strength achieved with 12 or more blows exceeds the strength achieved with 10MPa pressure. Looking at the highest applied pressure of 12MPa a resulting strength of 1.92MPa is achieved whilst the 12 and 16 blow samples achieve 1.81 and 2.17MPa respectively. A quick significance test shows that there is no significant difference between the 12MPa sample and either of the 12 or 16 blow samples, (E_D_E_DS_den-ref). Consequently it can be suggested that the strength achieved by quasi-static compaction to 12 MPa is the same as the 12 and 16 blow dynamically compacted samples.

These results suggest that the goal of replicating the strength achievable with a 10MPa press should be possible with in order of 14 dynamic blows. This is a very pleasant discovery as Gooding showed that for a constant energy transfer the optimum number of blows is around 16.

3.4 Effect of method of compaction on strength

One of the important tests that needs to be carried out is whether or not the method of compaction makes any difference to the strength of the sample compressed to a similar density. In order to check this, a set of results from dynamic and quasi-static compaction tests were compared. Everything except the method of compaction and the corresponding moulding ‘effort’ is the same for these results, summarised in the graph below.



E_D_E_density2

The graph immediately shows the similarity in the results given by the two methods. What is also of great interest is that the two methods of compaction seem to lie in a similar region on the graph with the dynamic results extending beyond the scope of the quasi-static results.

If we look at the results that overlap (i.e. where density does not exceed 2120kg/m^3) then we find the results for quasi-static pressures between 8-12MPa and dynamic blows between 8-16 are remarkably consistent. A significance test carried out on these results shows that there is no significant difference in the density-strength relationship between the two compaction methods. (In fact the difference is very low indeed with the Difference of Means/Standard Error of Difference equalling only 0.43). These results show that over the region of interest (for any given density in the range achievable by 10MPa compression) the strength achieved via either moulding method is highly similar.

3.5 Time delay between mixing and moulding

In the production regime of block manufacture it is customary to mix materials up into batches from which several blocks are moulded. The time delay between mixing and moulding will therefore increase from the first block made from the batch to the last one; this variation may be reflected in differing strengths of the blocks. As time passes the cement is progressing through the curing process and compaction should take place as soon as possible and certainly not after the critical time. (This critical time is defined by the concrete literature as the time after which working the cement mix causes damage to the cement crystals that have already formed.)

It is possible to see if the time delay has a significant effect on the characteristics of the finished sample by looking again at the results from the reference set of dynamic and quasi-statically compacted samples. For the production of these samples the time delay was in the order of about 15 minutes between the first sample and the third sample of the batch. The first sample in the batch was compacted after the moisture had been mixed into to the soil for about 3 to 4 minutes. Therefore the total processing time for a batch was around 20 minutes.

Below is a table of results from the dynamic compaction tests.

Table 3.5a

Number of Samples	Position within Batch	Bulk density mean kg m^{-3}	Bulk density s.d. kg m^{-3}	Bulk density Coef of var %	7-day wet strength mean MPa	7-day wet strength s.d. MPa	7-day wet strength Coef of var %
5	1st	2140	8	0.38	2.38	0.14	6.09
6	2nd	2131	5	0.26	2.23	0.15	6.53
6	3rd	2118	5	0.24	2.12	0.18	8.26

E_D_E_DS_den-ref

Below is a table of results from the quasi-static compaction tests.

Table 3.5b

Number of samples	Position within Batch	Bulk density mean kg m^{-3}	Bulk density s.d. kg m^{-3}	Bulk density Coef of var %	7-day wet strength mean MPa	7-day wet strength s.d. MPa	7-day wet strength Coef of var %
6	1st	2067	9	0.44	1.76	0.09	5.26
6	2nd	2054	13	0.65	1.61	0.13	7.84
6	3rd	2050	9	0.44	1.63	0.11	6.99

E_D_E_QS_den-ref

In both tables the mean wet strength falls with position within the batch. For the method of dynamic compaction there is a statistically significant difference between the first and third sample, but an insignificant one between the first and second or between the second and third samples produced. The quasi-static compaction results are slightly different as there is a significant difference between the first and second or third samples, but not between the second and third samples. (E_D_E_DS_den-ref).

These results seem to indicate that there is a significant drop in strength if a sample is produced more than 15 minutes after adding water to the soil/cement mixture. This poses some serious production problems and will need to be addressed and double-checked on the full-size block production to see if the same limitation exists.

The significant drop in strength could be a result of a variety of effects, some of which are as follows. (i) Some of the free water (useful for lubricating the particles causing better densification) has been absorbed by the fines content of the soil and partially used in the generation of the cement gel; (ii) there could be active cement crystallisation already occurring and this hinders the compaction process sufficiently to reduce the final strength; (iii) the compaction process actually breaks already-formed cementitious bonds. The last effect is probably of greatest concern as the crystalline growth is being damaged and wasting potential cement strength. The lack of lubrication or compactive effort could be remedied easily, but destruction of crystalline growth should be avoided if possible.

4. DENSITY AS THE SURROGATE FOR STRENGTH

The last chapter described the many tests carried out to determine how different factors affect the final strength of an SSB. We can now state with a degree of confidence that many of the independent variables affect both the cured strength and the green density of blocks. The understanding of the relationship between density and the strength can enable us to know how accurately and under what circumstances density at demoulding can act as a surrogate measure for the potential strength of a sample.

If density is to be the surrogate for strength it is important to decide which density should be used. Several different densities can be measured or inferred during the production of an SSB. Below is a summary of them. As in every case density is taken as a mass divided by a volume, we define different densities by which mass and which volume each uses. In practice we use weight as the source of mass.

Green bulk density – weight is that of material (*including* water) placed into the mould for compaction; volume is that measured upon removal of the green block from the mould. This density is the most commonly used as its calculation is easy to accomplish using simple measurements taken at the time of moulding.

Inferred green dry density – weight is that of material (*excluding* water) placed into the mould for compaction; volume is that measured upon removal of the green block from the mould. This density is not commonly used but is helpful when determining the comparative compaction of different samples with different moisture contents as the moisture variation is removed from the density calculation.

Pre-ejection dry density - this is a similar measure to *inferred green dry density* except that the volume is based upon a block's dimensions prior to its ejection from its mould. This measure is suitable for exploring density variations during moulding and is used in Chapter 5 below.

Cured bulk density – weight is that of the block after the curing process has just been completed, it includes the free water in the block as well as the absorbed water used in the curing of the cement; volume is as measured earlier on demoulding. Other current research has indicated that this density calculation may be the best indicator of the final strength of the finished sample.

Post-cure dry-density – weight is measured after both curing and driving off excess moisture (e.g. in a low-temperature oven); volume is measured at the same time. This is a difficult density to record during a research cycle of curing and subsequent crushing as time is taken to dry out the sample during which some curing is permitted to occur. Furthermore, the subsequent soaking prior to determining the wet strength (by crushing) will permit even more curing to take place, changing the characteristics of the sample.

Post-cure wet-density – uses the weight and volume measured after curing and then soaking for 24 hours (i.e. just prior to crushing). One can determine the voids ratio of the material by comparing the *post-cure wet* and *dry densities*. Their difference is due to the mass of the water filling any voids present.

For the majority of the experiments carried out the *inferred green dry density* was used as a working guide of densification, particularly where the moisture content was not held constant. For ease of communication and understanding these results have been converted into the ejected bulk density which is more commonly understood. Wherever a density is quoted it will be presented as either “inferred dry” or “bulk” density.

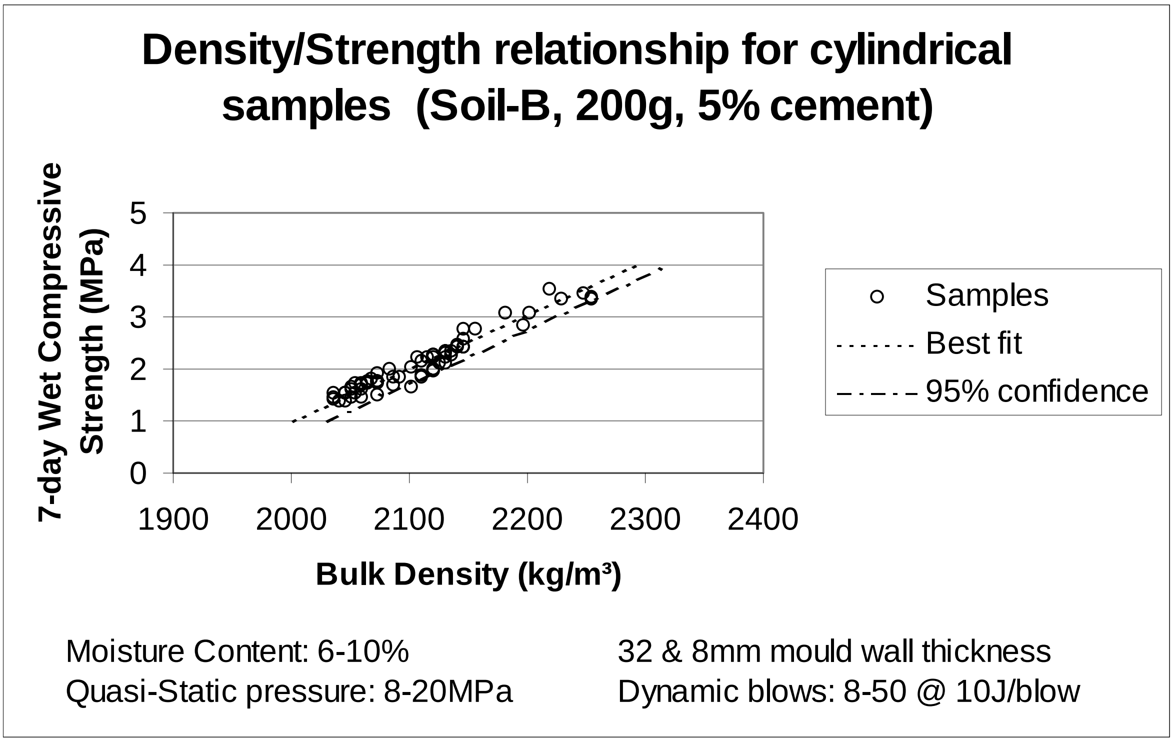
4.1 Summary of Density/Strength data

Before looking at specific variables to see whether or not their effect on the achieved density directly correlates with the effect on strength, a general overview of all the variables would be helpful. This section summarises the production of all of the samples generated and will show the relationship between density and strength even when several entities are varied.

In Section 3.4 above it was shown that at least under some circumstances green bulk density is highly correlated with cured wet strength regardless of the moulding method by which that density was created. That finding, however, does not help determine whether or not the density surrogate can be applied if other variables are changed. Many variables change the density achieved during the production process: these variables may have a greater or lesser effect on the change in strength. A simple method of checking this is to put the data from all experiments carried out in a graph and try to determine the general relationship for strength against density. Some variables (such as cement content) were however held constant because they were already known to have a significant effect on strength for similar densities.

The graph on the next page is a summary of all the tests carried out so far on small cylindrical samples that were stabilised and crushed. The variables include; moisture contents, number of blows, compacting pressure, mould wall thickness. Entities *not* varied were:- soil type, cement content, the size and shape of sample and in the case of impactive moulding, the impactor mass and drop height.

The graph shows a definite trend of strength against density with a reasonably straight-line relationship over the area of interest. The straight-line trend between strength and density is not a surprise because Gooding’s results (displayed in chapter 3) have a similar relationship. What we want to be able to do is to successfully predict with a 95% confidence that a sample compacted to a certain density will have a known strength. In order to do this a best fit line is drawn through the results using the least mean squares method. The results are then normalised to find the standard deviation and a new line is generated two standard deviations away from the best-fit line. This line is shown as the 95% confidence line on the graph and has the equation parameters described below.



For 95% confidence

The equation of this line: $y = mx + c$ is

Bulk Density (ρ in kg m^{-3}) $= 97x10^{-6}$ x Strength (σ in Pa) + 1993

Or more helpfully: $\sigma = 10{,}300\ (\rho - 1933)$

From this we can suggest that over the relevant density range we can use density to predict the 7-day wet compressive strength. This is a very useful property. In research work it allows us to sometimes replace the destructive and cumbersome measurement of strength with a quicker measure that leaves the samples undamaged and hence available for other tests. In block manufacture, green density provides an immediate feedback on block quality that is fairly easy to interpret. Inadequate density points to using a larger charge in a fixed volume mould or more effort in a variable volume mould. Setting density targets is straightforward.

Note incidentally that at mid-range, e.g. σ = 2 MPa, the sensitivity of strength to density is only 0.092, or put another way a 1% increase in density corresponds to an 11% increase in strength.

Whereas the formula above (valid for a particular soil, cement content and thoroughness of curing) is very useful for predicting the strength of these 200g samples, the coefficients may well be different for full-size blocks. The range of values that the above relationship is correct for is only a small variety of possible combinations. The relationship above is only for 200g small cylindrical samples made from Soil-B and stabilised with 5% cement. The moisture can range between 6 and 10% and the mould wall thickness can vary between 32 and 8 mm. Compaction can be either from quasi-static compression between 8 and 20 MPa or from dynamic impact using between 8 to 50 blows at 10J per blow via a 5kg impactor falling through 200mm.

It is hoped that the relationship between these variables at this scale can be extrapolated and adjusted for full scale block manufacture and still exhibit the same trends that have been noted here. This will be examined later on in the project and cannot be reported here.

4.2 Other variables needing consideration

The accuracy and reliability of the results above certainly suggest that the density can be a very good indicator of strength. However, in the normal production of an SSB many different variables would be changing. Some of those would either be out of the range investigated here or be one of the variables that were not investigated. This section looks at some of the possible variables that could either affect the strength without changing the density or variables that will affect the strength that have not been considered earlier in this paper.

Cement content – We have reduced the cement content to the lowest possible value for production and assumed that the manufacturing process would control these quantities very accurately. In reality the cement content will vary and possibly quite significantly depending on the production method and the conscientiousness of the production team. If the cement content varies blocks produced with the same achieved density will have significantly different strengths because of the large effect that cement has on the strength. It is not possible to account for a badly controlled variable, so either a factor of safety must be applied or the cement content vigorously controlled for each block.

Curing regime – Similarly, the curing regime will not be constant in a real production situation. Inadequate curing of the cement is probably one of the most common mistakes in the production of cement SSBs. The difference in strength between a well and a poorly cured block will be highly significant and will not be obvious from the density alone. The achieved density will only successfully indicate the strength if the curing regime is consistent with the reference set and the produced blocks.

Soil characteristics – The soil type used in the manufacture of SSBs may also change during production. The samples produced for this paper were made from a stable and consistent material carefully measured and mixed with cement. A different soil type will have a direct effect on the density that can be easily noted, but it will also have an effect on the ability of cement to add strength to the block. Moreover, due to the expansive nature of certain clays, the soil type may affect the durability of the block even more than its strength. Only long term tests would be able to confirm this proposition and that is currently outside the scope of this project. It would be very useful to show that the particular soil used has only a small effect on block strength or at least that any *change* in density results in a similar *change* in strength regardless of soil type. This analysis has not been carried out and consequently a safety factor would need to be applied to accommodate the possible variation in the soil composition.

Size and shape – The dimensions of the sample produced will have a direct effect on the measurable strength achieved for the same density. Larger samples usually have a lower compressive strength and consequently the short small cylinders used in this research will show a higher strength than would a full-size block of the same density.

Correction factors for compressive strength of concrete samples with a cylinder length to diameter ratio other than 2 can be found in (Orchard, 1979) (p. 79). The 200g cylinders have a diameter of 54.4mm and a height between 41 and 45 mm. Therefore the ratio of length to diameter is between 0.75 and 0.83 giving a strength of between 136% to 125% that of a reference cylinder (with L/D = 2). So normalising to such a reference cylinder for compression testing requires the data in this paper to be multiplied by about 0.77. The small stabilised cylindrical samples that Gooding produced had approximately L/D = 2 and were the minimum size suggested for compression testing of concrete samples, (i.e.: 50mm diameter and 100mm length). Note: The sample size selected for this project is smaller than the recommended minimum, but was so chosen for two reasons. Firstly we wished to keep the ratio of side wall area to compaction surface area the same as for full size blocks. And secondly, the particle size distribution of the soil used contained much smaller particles than the normal aggregate mix used in concrete.

Type of strength test - The usual method of production-testing an SSB is to carry out a rupture test on it. This is done by supporting a block only at both ends whilst loading its centre (by stacking other blocks on top until it fails). This type of test is not as accurate as laboratory compression testing and indeed measures a *tensile* strength that is likely to be not more than 25% of compressive strength.

The principal finding above (that for a given soil, cement content, block shape and curing regime, green density is a good surrogate for strength) could be extrapolated onto full size blocks with a reasonable degree of confidence. Ideally a reference set of compacted blocks would need to be made on start-up of manufacture, the densities and strengths measured for each one and a target density thereby set. In practice cement content, block shape and curing regime may be standardised. This leaves the variation in soil type to be accommodated and hopefully future work will show this not to be a high sensitivity variable.

5. ENERGY INPUT AND CURED BLOCK STRENGTH

5.1 Energy Productivity

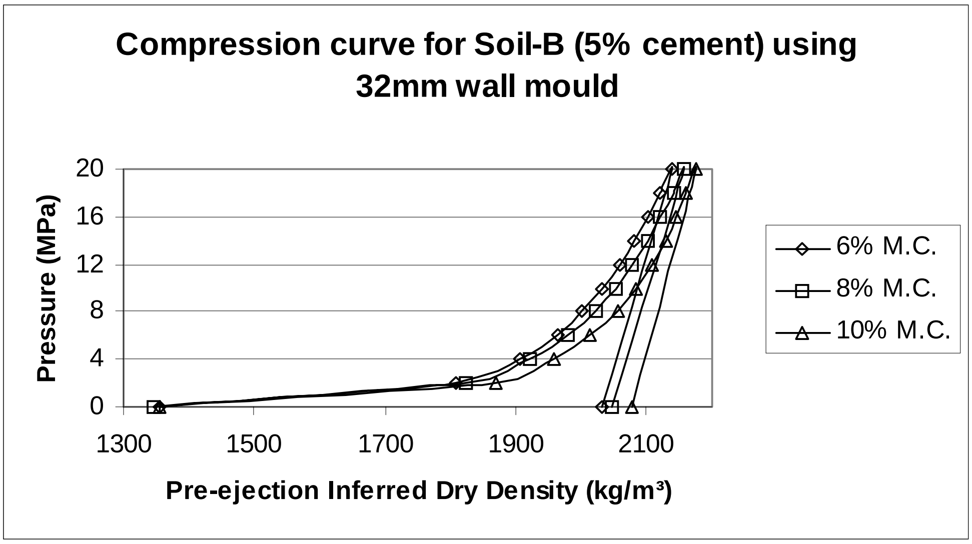
Establishing the factors that affect the final strength of a sample and the possibility of using density as a surrogate for strength were just some of the useful discoveries described in earlier chapters of this paper. To compliment these findings other aspects of the SSB production and performance were noted during these experiments. This chapter will give a brief description of the different observations that were made during the course of the project so far.

One of the variables little studied by previous researchers was 'energy productivity', namely the block strength achieved (in Pa) per unit of energy cost (in J/kg). For manual production, energy is of considerable interest, because labour time is a significant part of production cost. Labour time in turn s closely, although not solely, related to energy input needed per block.

During previous research Gooding discovered that the energy transferred into a full-size block quasi-statically compacted to 10MPa was 279J/kg and this achieved a density of 2038 kg/m³. The soil ('Soil A') he used to make this 8kg block was different from that employed in the current research, so his findings cannot be directly compared with newer data. However, Montgomery later used Gooding's Soil A to make a 10kg block *dynamically* compacted 2040 kg/m³ using only 192J/kg of energy. This indicated a 30% energy reduction for full-size block compaction and is a trend that we would wish to confirm in these results here.

Calculating the energy transfer via dynamic compaction is a trivial calculation using the drop height, the mass of impactor, the number of blows and the gravitational constant. (However if the impactor is released from a constant height, suitable adjustment must be made for the fact that the block's top surface drops, blow by blow, causing the drop distance for the later blows to be higher than for the earlier ones.) By contrast measuring the quasi-static energy transfer is far from straightforward. A force displacement graph needs to be generated during the compression process so that the area under the curve (i.e. energy applied) can be calculated. Doing this has proved quite difficult to achieve for full-size blocks and where necessary previous results have been used for reference.

Three different moisture contents were used in investigating the pressure-density relationship for quasi-statically compressed Soil-B reported above. We can employ the same data to investigate energy productivity. The samples were compressed up to a maximum of 20 MPa and 9 in total were produced. The results of the compaction can be seen in the graph below. Results from other dynamic compaction tests (mentioned earlier in this paper) are used in this comparison.



E_D_E_QS_press'den2

It is immediately clear that there is significant "spring-back" in the compressed material when the load is removed. It has already been established that the difference in achieved density from the different moisture contents is reflected in their respective strengths. We want to use this data to discover what is the approximate energy transfer for different compaction pressures. The cumulative energy transferred for each moisture content and at each pressure can be summarised in the following table.

Table 5a - Q-S compression of 200g cylinders

Pressure MPa	Force N	Average energy transfer (J) 6% M.C.	8% M.C.	10% M.C.
0	0	0	0	0
2	4649	37	39	41
4	9297	54	55	55
6	13946	70	71	70
8	18594	83	86	84
10	23243	97	99	96
12	27891	111	112	108
14	32540	125	126	119
16	37188	139	140	131
18	41837	154	154	142
20	46486	168	168	153

E_D_E_QS_press'den2

From the table it appears that moisture content does not have a great effect on the energy necessary to achieve a certain pressure. A pressure of 10MPa requires approximately 97J of energy to be applied to the sample (i.e. 500 J/kg). This would be equivalent to 10 blows of a 5kg impactor falling through 200mm. Actually the earlier impact blows fell less than 200mm; although the exact distance was not recorded, the energy applied during the first 10 blows was estimated at 95J and we may therefore treat 10MPa Q-S compression and 10-blow impact compression as requiring almost identical energy inputs.

In order to compare the efficiencies of the two methods of compaction the achieved densities of each process needs to be examined. Back in section 3.3.1 the results of several quasi-static compression tests showed an average bulk density of 2067kg/m³ could be achieved with a pressure of 10MPa. Then in section 3.3.2 the results of dynamic compaction tests indicated that 8 blows and 12 blows achieved a bulk density of 2053 and 2097kg/m³ respectively. Taking as a tolerable approximation the average of the 8 blow and 12 blow densities, we obtain a 10 blow density of 2075kg/m³. Thus there is an almost negligible density increase of 8kg/m³ when the same 97J of energy was applied dynamically rather than quasi-statically. The inferred strength improvement is less than 4% and lies within the variability of the strength measurements of the two processes. Consequently it can be stated that for this scale of production there is no significant difference in energy productivity between quasi-static and dynamic compaction.

This finding is a bit of a disappointment, because it was hoped that the higher energy productivity of dynamic compaction detected on a 10kg scale would also hold for smaller blocks. This phenomenon may be a possible result of the very small total energy transfer for these samples, or from probably sub-optimum momentum of the small impactor used in these tests. The better energy productivity of the impactive method may still exist for larger samples and full-size blocks, but this will have to be confirmed later in the research programme.

Combining data from tables 5a and 3.3a indicate that over the range 8 to 12Mpa Q-S compaction pressure, the energy productivity is fairly constant (at about 3500 Pa/J/kg) and hence that for this process the sensitivity of block strength to energy input is about unity. From table 3.3b the energy productivity for impactive formation falls from about 3600 Pa/J/kg at 8 blows to only about 2400 Pa/J/kg at 24 blows - a sensitivity of strength to energy input of only 0.6. Thus beyond a certain point, extra blows give diminishing returns of block strength.

5.2 A good use for excessive strength

The results so far have indicated that a wet compressive strength of around 2.0MPa is possible after 7 days of curing. This would be considered pathetic in the concrete industry, however, as we have already shown much of the concrete literature is not appropriate for the production of SSB’s. The earth building literature (Houben et al., 1994) suggests that a dry compressive strength of 2.0 MPa is adequate for single storey dwellings. This value already has several safety factors to cope with production defects, environmental effects and construction technique. Furthermore the text lists various materials and puts them into classes A, B, C, D; (‘A’ being the best and ‘D’ the worst).

Class ‘A’ building material is considered to have a wet compressive strength of 2.0MPa after 28 days of curing. The graph in section 2.2 shows how the strength increases with time from 3 to 90 days. As 7-day strength is approximately 60% of the 28-day strength for concrete samples, we may assume that blocks shown in the various tables above as having 7-day strengths of 2MPa would be likely to reach 3.3MPa by 28 days. This therefore puts these produced samples well into the ‘A’ class of building materials and a competitor to industrially-fired brick and superior to clamp-fired brick. Indeed they probably are too strong.

It might be said that there is no such thing as too much strength. However, in the efficient use of building materials it is unwise to make a brick several times stronger than necessary. If it turns out that the full-size blocks are much stronger than necessary then it may be beneficial to (i) use less cement for the same amount of material, (ii) use less energy in their production but the same amount of material or (iii) modify the shape of the block to save material. Which of these three options is best will depend on the sensitivity of strength to that input (cement, energy, volume) and the fraction of total cost attributable to that input. In fact the sensitivities of strength to cement, energy and soil volume are approximately 1.1, 0.6 (impactive) to 0.8 (quasi-static) and 1.0 respectively. As all these figures are close to unity, sensitivity alone does not distinguish which option to choose. However, reducing cement usage by 1% will usually save more money than reducing energy usage by 1%, which will in turn save more than reducing soil volume by 1%. Therefore if there is excess strength the most economic course is probably to reduce the cement content. (This conclusion holds less strongly for impactive compaction than for quasi-static because of the former’s somewhat lower strength-energy sensitivity.)

6. BLOCK EJECTION FORCE

As mentioned in section 2.1 the de-moulding force is a dependent variable of interest. The force required to eject a number of fully compacted 200g samples was recorded., and from this data we can make some useful observations. However the factors determining the size of demoulding forces have not been exhaustively investigated and the findings reported here are only provisional. In particular they depend upon data from samples much smaller than a normal building block.

These findings relate to machine design and production technique rather than to block characteristics. However, it could be said that the higher the ejection force the better the surface finish of the final block will be, as significant wiping of particles will occur and a smoother and less penetrable surface results. This characteristic may enhance the durability of the block and perhaps also reduce the rate of moisture loss from the block surface during curing.

We start with the expectation that the greater the achieved density, and hence the higher the moulding forces, the higher the potential force required to eject the block from the mould. Other possible determinants of that force are the compaction method used (quasi-static or impactive), the extent to which the cementitious action has progressed prior to ejection, the ‘stickiness’ of the particular soil used and the mould geometry (e.g. thickness, proportions, taper). We only have data suitable for investigating the first three factors, namely moulding force, moulding method and (indirectly) cement action.

For impactive compression, Table 6a relates ejection force to degree of compaction - as measured by moulding energy or by density achieved. It confirms that the required block ejection force rises with the degree of compaction. The sensitivity of that force to the *energy* input is quite high (over 0.8) so that we might crudely assume that ejection force is proportional to moulding energy. Unfortunately with impactive forming, we cannot easily measure the maximum moulding *force* and hence we cannot explore the interesting ratio of the ejection force to moulding force.

Table 6a Variation of ejection force with degree of compaction

Number of Blows	Energy Transfer J	Av. Bulk Density kg m^{-3}	Ejection Force Average kN	S.D. kN	C.o.V %
8	78	2053	0.77	0.10	13.36
12	118	2097	1.18	0.08	6.94
16	157	2113	1.28	0.08	5.90
20	196	2133	1.50	0.05	3.40
24	235	2162	1.91	0.11	5.98

E_D_E_DS_density2

Tables 6b and 6c allow us to assess the effect of choice of moulding method upon ejection force. We can see that the force required to eject the 200g cylinders formed by the two methods to very similar densities (means of 2054 and 2050 kg m^{-3} respectively) are significantly different. The quasi-statically compressed samples have an average ejection force of 1.07kN, whereas for the dynamically compacted samples it is only 0.77kN - 28% lower. A test on these results confirms that the two ejection forces are significantly different.

Table 6b Ejection force for quasi-static samples

Number of samples	Position in Batch	Av. Bulk Density kg m^{-3}	Ejection Force Average kN	S.D. kN	C.o.V %
6	1st	2067	0.95	0.18	18.92
6	2nd	2054	1.07	0.14	13.40
6	3rd	2050	1.13	0.09	8.03

E_D_E_QS_den-ref

Table 6c Ejection force for dynamic samples

Number of samples	Position in Batch	Number in Batch	Av. Bulk Density kg m^{-3}	Ejection Force Average kN	S.D. kN	C.o.V %
5	1st	1	2140	1.29	0.14	10.61
6	2nd	2	2131	1.35	0.10	7.38
6	3rd	3	2118	1.36	0.12	9.02

E_D_E_DS_den-ref

It can also be noted from Tables 6b and 6c that the 'stiction' to be overcome by the ejector mechanism increases as the position in the batch increases. As successive samples are produced from a particular batch of mixed soil, their density decreases but the force to eject them increases. This increase in 'stiction' cannot be a result of the *fall* in density, but is probably an effect of moisture acting within the soil. As time progresses since mixing a batch, the moisture in the soil become redistributed or even lost, reducing the free moisture available for lubricating the sample against the mould walls. As mentioned in section 3.2, moisture content has a significant effect on the block characteristics, both in terms of achieved density and cement curing. Here we see water also having a significant effect the ejection force required during production. Other earlier experiments had also shown easier ejection with wetter mixes.

Earlier experiments had also shown qualitatively that ejection forces are lower with thick-walled moulds than with thin-walled ones, but that phenomenon needs further study.

7. IMPLICATIONS FOR MACHINE DESIGN

The second objective of this research project is to extrapolate the findings from experimentation with small-scale samples into full-size block manufacture. This requires the design and development of a machine capable of dynamically compacting full-size blocks. Several findings in the previous chapters are significant and should be implemented into a comprehensive machine design and production regime. This chapter summarises these findings and explores how the production of full-size SSB's could be modified to accommodate them.

The basic principle behind quasi-static compaction is an excellent one: the great majority of manual presses in developing countries are based on it. Low-pressure manual presses are readily manufactured and maintained in such countries using local materials and skills. High-pressure machines are more complex and robust, they are more difficult to locally manufacture and maintain and they are significantly more expensive. The higher forces in a high-pressure machine need to be dissipated thought stronger bearings and a thicker steel body. This adds considerably to the weight, production and material costs of the machine, probably putting it out of the reach of the urban poor in developing countries. The high-pressure presses yield a much stronger block and therefore permit production of blocks with only a low cement quantities. But the potential cement savings using the higher-pressure machines do not fully offset the greater machine cost. Gooding conducted a survey for the ODA in 1996 assessing the financial pro's and con's of increasing moulding pressure and reducing cement. He discovered that in every country investigated it was more economic to have a high cement content (8-15%) instead of increasing the compacting pressure.

Gooding's hope was that the high-pressure machines could be replaced with a dynamic compaction machine of comparable cost to the low-pressure manual machines. If this was possible then the reduction of cement content became a viable alternative and the overall cost of SSBs could be reduced. Research since then has shown that excellent stabilised samples can be made via dynamic compaction using only 5% cement. It remains however to show that *cheap* dynamic compaction machines can be devised.

7.1 Machine design specifications

Several dynamic compaction rigs have been manufactured throughout the different periods of research, but dynamic compaction of full-size block was only performed once and using a temporary test rig. A full-size machine should be manufactured both to facilitate research and to show that such a machine can be economically made and operated. The first design will of course not be the final design. There will be inevitable alterations to the design as extrapolation of smaller-scale experimentation to full-size block production takes place. The prototype full-size design therefore needs to incorporate more flexibility than a production model in order to continue effective investigation of the process of dynamic compaction.

Apart from the research requirements of the machine there are several other requirements that can be noted from the research carried out thus far. The safety of the machine is paramount. A falling mass of 50kg is a significant potential hazard and precautions need to be taken to ensure that the impactor cannot fall onto any part of an operators body. If possible the mechanism should be inoperable unless all necessary guards are in place and the operator is well away from the falling impactor. Neither ejection of the block nor routine cleaning of the machine should require operators to place any part of their body underneath the impactor.

Changing to dynamic compaction has shown some reduction in the force required to eject a compressed block. This force is still however substantial - perhaps 10% of the peak moulding force - and will need to be applied manually via a lever by a force not exceeding that which can be repeatedly applied with the hand or foot. Since existing machines require levers over one metre in length to achieve this necessary force *by hand*, it may be possible to use a smaller lever if the force is applied by foot. Either system is acceptable, but a foot-operated lever will be more compact and therefore require less material.

The design of the machine needs to be restricted to a level of complexity that lies within the user’s understanding, and requires only simple and sporadic maintenance. The machine’s construction should also be constrained to use materials and a level of production technology that is readily available in the countries where it is to be made and operated. In spite of the prototype being manufactured in an institution where advanced production facilities are available, the constraints above need to be applied to its design to facilitate its subsequent dissemination and acceptance in the field.

A feature of dynamic compaction is that the peak pressures generated are lower (for a specified final block density) than with quasi-static compaction. This means that the mould walls do not need to be as thick as in traditional presses. An experiment was carried out with dynamic compaction using four different moulds. Each had a different mould wall thickness that followed a geometric progression as follows: 32mm, 8mm, 2mm, 0.5mm. For the same energy transfer it was noted that the 8mm and 2mm moulds generated slightly higher density samples, which was an unexpected yet pleasant finding. Furthermore the samples compacted in the thinnest walled mould achieved a high density yet exerted a stress in the mould walls corresponding to only 30-40 microstrain. The yield strain for steel is around 1200 microstrain, so this shows that the material even when very thin is able to deal with the forces present during dynamic compaction. It should therefore be possible to design a mould for a high-densification impact machine that is comparable in thickness with those in traditional low-pressure quasi-static machines. Such a mould will be much cheaper and lighter than those required by the heavy 10 MPa presses needed to produce comparable densification by the quasi-static method.

7.2 Some production guidelines

This section will attempt to outline some of the key areas of concern in a production regime for manufacturing SSB's. Some of the issues have been discussed above whilst others are a result of past research and experience. As full-size block production has not been carried out as yet it is foolish to categorically state anything as being the most important aspect of production. However, it is possible to suggest some important requirements of the manufacturing process that have been revealed in this research.

Moisture content has proved to be a highly significant variable in the production of SSB's. The available water will have an effect on the workability of the soil/cement mix, the achievable density, the de-moulding force, the ease-of-handling and the curing of the cement. Since these are all important, it is difficult to say which has precedence over the others. Having said that, it is in the handling of green blocks that the greatest production losses can occur and therefore achieving adequate ease of handling might be made a first priority when choosing water content. Many experiments were carried out at around 6% moisture where blocks are both easily handled and readily brought to a high density, so this could be a good starting point for full-size block tests.

Thorough mixing of the materials prior to compaction is very important when the quantity of stabiliser is so small. For these experiments the soil/cement/water mixture was very carefully proportioned and mixed together to give a good consistency between experiments. This thorough mixing is difficult to achieve on full-size block and will be even more difficult to practise in the field. Nevertheless, it is advisable to include a measure of care when mixing the materials together as poor mixing will not bring out the full potential of the SSB. It has been said for concrete that "good concrete and bad concrete are made from the same ingredients, it's the method of production that will determine the finished product". The same could be said for the production of SSB's. It may be that it is one of the tasks of a machine designer to address not just moulding but also ingredient batching.

Handling after moulding - once the finished block is compacted it needs to be ejected and carefully moved to a curing area. Even where moisture content has been chosen to enhance ease of handling, design attention needs to be given to reducing breakage prior to curing. Boards can be used to carry blocks around and special block lifting apparatus can be employed to help transfer the blocks to and from these boards.

Curing - a book could be written about the use and abuse of cement in developing countries. Possibly the greatest and yet commonest mistake in a production regime for SSB's is to leave the freshly formed blocks out in the open to "dry out". This makes the cementitious reaction slow down and stop, as the moisture is lost to the environment. Consequently the crystalline growth does not get very far and very little strength is added to the material. Subsequent wetting of the material may help to further cure the cement, but this will not be as effective as thorough curing of the blocks immediately after moulding.

Batch size: Time delay in the production of the SSB’s from a batch of soil has been shown to be of importance. If the time, between adding moisture to a batch of soil and the production of the final block from that batch, can be minimised then the SSB’s produced will be both stronger and more consistent. The extreme case of having a batch size only large enough to produce one block at a time, is not economically viable and the material for several blocks will need to be made at the same time. However this will introduce unwanted variation in the characteristics of the finished blocks and the blocks produced later in the batch will be inferior to the ones made first. Consequently using a small batch size not only facilitates good manual mixing, but also gives consist block attributes with the batch itself. The ideal of continuous mixing and the design option of adding water only as the charge enters the moulding chamber will be investigated further.

8. CONCLUSIONS AND RECOMMENDATIONS

The first and most important conclusion to make is that the process of dynamic compaction works as effectively as the current method of quasi-static compaction in densifying stabilised soil building blocks. The tests described in this paper have shown that, at least on the small-scale used for tests, the dynamic method performs as well in terms of energy productivity (Pa/J/kg), achieved density and the wet compressive strength of cured blocks. We also have good reason to believe that larger scale dynamic compaction will achieve higher energy productivity than reported here. This finding will need to be confirmed on full-scale tests to be carried out in the future.

The research has also brought to light several factors that affect the stabilisation of soil blocks. Stabilisation is mainly concerned with adding durability and strength to a material that would otherwise be unsuitable for construction. Since the durability of the samples produced could not be checked then compressive strength was used as the best available surrogate. Using the same fraction of cement and the same curing regime, the wet compressive strength of samples was found to depend entirely on the level of compaction that they achieved. Consequently density on demoulding can be used for a satisfactory indication of the subsequent cured strength of a block.

Dynamic compaction brings one significant advantage to the process of heavily compressing soil: it requires much lighter equipment. High-pressure quasi-static compaction is much more complex and expensive than the lower-tech method of dynamic compaction. The experiments reported indicate that a machine for dynamic compaction could be similar in cost yet much superior in performance to the low-pressure quasi-static presses that are popular throughout developing countries. Thin mould walls, short levers and the absence of hydraulics make dynamic compaction available to a wide group of people in need of low-cost housing solutions.

Changing the method of compaction from quasi-static to dynamic is potentially straightforward. However comprehensive machine design is yet to be done and a full-size prototype is yet to be built and tested. This is should be achieved in the next year. The safety of the machine is the only major factor of concern as this needs to be fully addressed before a design can be propagated.

Once a prototype has been produced and laboratory tested then the technique needs to be fully tested in the field using potential users and readily available soils. This process will help to determine the obstacles to successful dissemination of the technology in the areas where it is most applicable. Possibly the largest issue is the public's attitude towards soil itself. Soil, even when stabilised to perform as well as fired brick and concrete, currently has an unfavourable image and other alternatives still exist for the uneducated urban poor. Small brick clamps are currently the most popular method of soil stabilisation but these use increasingly scarce resources such as firewood in a very inefficient way. Substantial improvements in the performance and economics of the cement-stabilisation alternative are needed if it is to make much headway in the short term.

Future research will initially focus on the extrapolation of the findings to date onto full-size blocks. This should include the fabrication of some sample walling for realistic durability testing. The production process for full-size blocks needs to be carefully refined to ensure that the maximum benefit is obtained from the inputs of effort and cement. Once this research has been completed then it will be necessary to disseminate the findings (especially through demonstration) and to assess reactions to the new technology. The level of technology and understanding required for this technique is such that if the reaction to it is favourable its dissemination could occur by normal processes of copying and commercial initiative in urban areas of relevant low-income countries.

Bibliography

1. Akroyd, T. N. W. (1962). Concrete properties of manufacture Pergamon Press.

2. Gooding, D. E. M. (1993). Improved processes for the production of soil-cement building blocks. Unpublished doctoral dissertation, Warwick, University of Warwick.

3. Houben, H., & Guillaud, H. (1989). Earth Construction: A Comprehensive Guide Intermediate Technology Publications.

4. Houben, H., Rigassi, V., & Garnier, P. (1994). Compressed earth blocks: production equipment (Technologies (CDI) No. 5). Brussels, Belgium: Centre Devt Industry (CDI).

5. International Labour Office. (1990). Small-scale brickmaking (Third ed.). Geneva: International Labour Office.

6. Lea, F. M. (1970). The chemistry of cement and concrete (3 ed.).Edward Arnold.

7. Neville, A. M. (1995). Properties of Concrete (Fourth ed.).Longman.

8. Orchard, D. F. (1979). Concrete Technology Applied Science Publishers.

9. Rigassi, V. (1995). Compressed earth blocks - Volume 1: Manual of production GATE.

10. Scott, R. A., & Pearce, R. W. (1975). “Soil compaction by impact”. Proc. ICE Symp. Ground treatment by deep compaction (pp. 19-30). London: Institute of Civil Engineers.

DEVELOPMENT
TECHNOLOGY
UNIT

D
T
U

Working Paper No. 54

The Problem of Evaluating the Performance of Landmine Detection Systems

February 2000

R Gasser & T H Thomas

Development Technology Unit
School of Engineering, University of Warwick
Coventry CV4 7AL UK
Tel: +44 (0) 2476 523122 Fax: (0) 2476 418922
email: dtu@eng.warwick.ac.uk
http://www.eng.warwick.ac.uk/DTU/

DTU Working Paper No. 54 **February 2000**

The Problem of Evaluating the Performance of Landmine Detection Systems

by Russell Gasser and Terry Thomas

Contents

1 Introduction

Even in densely mined areas mines tend to be widely separated, and very few mines will remain undetected by a good mine detector system. Objective analysis of the performance of mine detectors based solely on the percentage of mines not detected is thus difficult at more than an anecdotal level. Field conditions, the types of target and operating procedures vary so widely that testing in a laboratory or test area may not reveal the limits of the performance of a mine detector in a specific mined area with a particular type of mine/UXO contamination. Generalisations about humanitarian demining equipment performance can have only a very limited validity.

Reports of promising technologies of necessity quote results such as a "95% success rate" or "16 out of 16 targets found" [Ano00] but usually do not attempt to present an analysis of the statistical confidence of such data. This working paper examines the difficulties in assessing the performance of mine detecting equipment both quantitatively, by developing a statistical analysis and presenting the results, and qualitatively, by analysing the practical difficulties of evaluating equipment performance specific to demining. Use of the concept of "margin of detection" is proposed as a possible way forward.

2 The three main problems

The three principal difficulties in assessing prototype demining equipment are:

1. Testing equipment that is still under development, and hence not yet capable of finding every mine, in live areas is not possible because of the risk to the operator. Testing under simulated conditions does not yield the same results; the major impediments to finding mines, such as vegetation, have to be removed or altered to place surrogate mines. Deminers, no matter how carefully they seek to re-create their working practices, are likely to act differently in live areas from ones they know to be safe. The ethical justification for asking deminers to perform a trial in a safe area while leading them to think that it is mined in order to simulate live conditions more accurately, is debatable.
2. Finding enough mines or surrogate mines to provide a meaningful statistical analysis of the detection rate at a useful level of confidence is quite impractical as meaningful trials require hundreds of targets.
3. Sensitivity to factors beyond the control of the test protocol may be greater than sensitivity to the parameters being measured. For example, the exact depth of a small buried target may strongly influence the probability of detection. Placing a target and then re-filling above with soil makes precise determination of the depth difficult, moreover it may well not give the same results for some methods of detection as long-buried mines, and the precise depth of soil may vary slightly after heavy rain or vegetation growth.

The rest of this working paper addresses problems two and three above.

3 The statistics of missed mines

The only published paper of note on detection probability in humanitarian demining and associated confidence levels appears to be that of Voles [Vol98] who presented his argument somewhat cryptically. His method permits the calculation of results for only a limited number of levels of confidence, namely the values of the cumulative Poisson function $F(x;\lambda)$ where x= 2, 3, 4 or 5 and $\lambda = 1$. The analysis outlined below is both simpler and more general, and is believed to illuminate an inconsistency in Voles' assumptions.

Mine detection satisfies the conditions for a Bernoulli trial [MF85, page 57]:
(i) there are only two outcomes (mine detected or mine not detected),
(ii) the probability of success is the same for each trial,
(iii) there is a constant number of trials (the total number of mines), and
(iv) the trials are independent (locating a mine does not affect the performance of the mine detector when attempting to locate the next mine).

Thus the use of the Binomial distribution for probability calculations is justified. The conditions for using the Poisson approximation to the Binomial are also satisfied if the number of mines (n) is large and the probability of failing to detect each mine (p_{fail}) is small. In general the Poisson approximation can be considered valid if $n \geq 20$ and $p \leq 0,05$ or if $n \geq 100$ and $n.p \leq 10$ [MF85] [Bar94].

3.1 A conceptual model

A useful conceptual aid to understanding the statistics of mine detection can be to consider the model of a population of mines of which a fraction u are undetectable, and a detector capable of always finding all the rest. This gives the same results as a population entirely made up of theoretically detectable mines and a detection system with a probability of only $(1 - u)$ of locating each mine. A practical example of this conceptual model is the type of minimum-metal mines that are supplied with steel discs which can be optionally fitted as the mines are emplaced in order to make them easy to locate later with a standard metal detector (e.g. the TMA-4 anti-tank mine formerly made in Yugoslavia). If the detection performance required is the commonly accepted 99,6% success rate then it can be assumed that on average four mines per thousand have not been fitted with discs, and the metal detector will find all of the mines with discs and none that lack discs. This is a reasonable assumption in practice if the mines have been laid at the correct depth. Defining the exact reason for indetectability makes no difference to the statistical analysis provided that it is a random process. In practice this condition may not be completely satisfied as failure to detect a mine may be due, for example, to specific soil conditions that prevail over some mines and not others in a non-random manner, but the consequences of this are considered to be negligible.

3.2 Definition of the problem of sampling

The problems of obtaining meaningful results from limited-size trials can be expressed thus:

Test results show that in a sample of n mines (the trial size) x (= 0, 1, 2, 3,..., n) mines were ***not*** detected. What is the probability that an arbitrary proportion π of similar mines from the same population would also escape detection? What is the limit of confidence in this result?

Clearly if the number of mines used in the test (n) is very large there is a high degree of confidence that $\pi = \frac{x}{n}$. If the sample size is small it will less accurately represent the stock of mines as a whole and there is a certain likelihood that the value of π is larger than $\frac{x}{n}$ (or smaller than $\frac{x}{n}$ if $x > 0$). The trial result therefore depends on (i) the probability that any individual mine is detectable and (ii) the confidence that the sample accurately reflects the stock as a whole.

Taking the standard promoted by the United Nations of 99,6% mine clearance, an example of the conceptual model outlined above might be a crate of 1 000 mines of which 996 are detectable (they have steel discs that

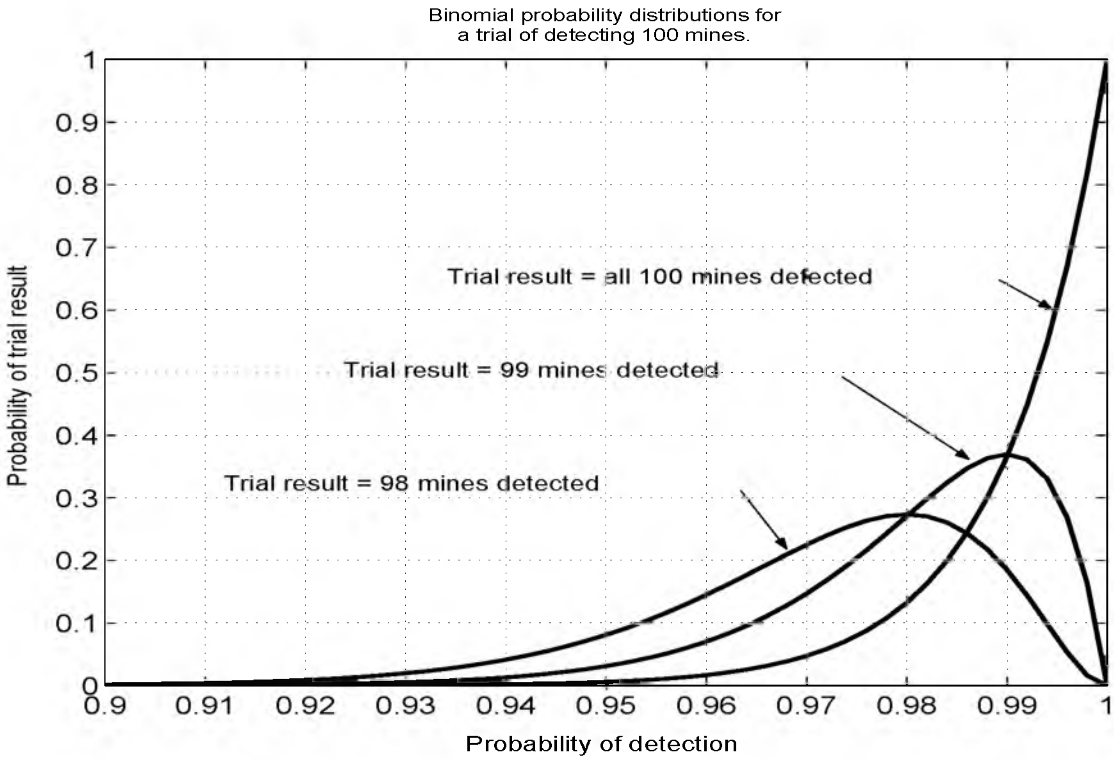


Figure 1: **Probability of result versus assumed probability of detection (fraction of mines that are detectable) for trials of 100 mines.**

make detection possible correctly installed) and four are undetectable (without the discs). It is obvious that a small sample, such as 20 mines, taken at random from the crate might well not include any undetectable mines. Clearly, it is not correct to conclude that because a small sample contains no undetectable mines that there are no undetectable mines in the crate. The question is, how many mines must be detected in a test to be sure that there are indeed very few undetectable mines in the crate, and what are the precise percentage probabilities?

3.3 Analysis using the Binomial probability distribution

The likelihood that a sample of size n, taken at random from a stock of mines having a proportion π of undetectable mines, will contain exactly x undetectable mines is given by the Binomial probability distribution

$$f(x; n, \pi) = {}_nC_x \pi^x (1-\pi)^{n-x}$$

where ${}_nC_x$ is the number of combinations of x events in n events.

Thus the likelihood that the sample will contain from zero to k (inclusive) undetectable mines is given by the cumulative Binomial function

$$F(k; n, \pi) = \sum_{x=0}^{x=k} {}_nC_x \pi^x (1-\pi)^{n-x}$$

Figure 1 shows the results of calculating the probability of the outcome of trials of attempting to find 100 mines, plotted against the probability of detection; the Binomial distribution relates the two probabilities as

$p_{result} = binomial(x, n, \pi)$ where x mines escape detection in a trial of n mines by a detector with a p_D of $(1 - \pi)$. Clearly if the detector is perfect (p_D=100%) the probability of finding all the mines is one, and the probability of failing to detect any number of mines is zero. When the p_D is 98% the most likely outcomes are one or two mines not detected.

In practice it is usual to discuss the case of a *lower limit of probability of detection* on the basis that any improvement in performance is welcome. When a detector is referred to as "having a p_D of 99%" the more complete statement is that the p_D is 99% *or greater* under certain conditions.

3.4 Using trial outcome to predict p_D

Predicting the outcome of a trial from a knowledge of the detector is less useful than using the results of a trial to find values for the probability of detection and the confidence in that probability. This is a standard problem in statistical quality control and is covered in text-books, for example Yamane [Yam73] who bases his text on the earlier work in the 1930s of Clopper and Pearson [CP34] and Barnes [Bar94] who presents a nomogram adapted from Johnson and Kotz [JK69].

Two hypotheses are tested. These are:
H_0 : The mine detector performs to the required standard.
H_A : The mine detector is defective.

The associated errors are $\alpha = P(H_0$ rejected when true) and $\beta = P(H_0$ accepted when false). In quality assurance α is known as the producer's risk and β as the consumer's risk.

In testing mine detectors (1-α) is the confidence that the detector is accepted correctly from the test results (confidence in the result), and (1-β), known as the power of the test, is the confidence that an unsatisfactory detector will be rejected. In the case of a trial where all the mines are detected β is clearly meaningless as the detector has been shown to function with a theoretical maximum p_D of 100%. In trials where one or more mines are missed β can be used to define the confidence with which an unsatisfactory detector would be rejected, though this is likely to be so low that it is not useful.

In any distribution the parameter α can be visualised as the fraction of the area under the tail of the probability curve lying below the value p_{limit}. This is illustrated in figure 2.

In most statistical quality assurance the areas under both "tails" of the distribution curve contribute to the probability α; manufactured items that are oversize as well as those that are undersize should be rejected. In mine detection there is no meaningful interpretation of a detector that is "too good" so α is exclusively the probability that the detector is not good enough, the area under the curve below the required minimum value of the probability of detection. This is the lower limit of the possible range of values of p_D for that value of α.

An arbitrary decision has to be made as to the relative magnitude of the p_D and confidence (1-α) in order to analyse test results. This decision can also be visualised as moving the vertical "decision" line to the left or to the right on the graph of the probability function. As the line moves leftwards the area under the curve to the right of the line increases illustrating that as the minimum probability of detection required is reduced the confidence that this can be achieved increases.

Direct calculation of the Binomial distribution is straightforward and can be performed rapidly using a digital computer. There is no longer any need to employ analytical methods including further assumptions to reduce the problem to a form that is more readily calculable; this was necessary until computer power

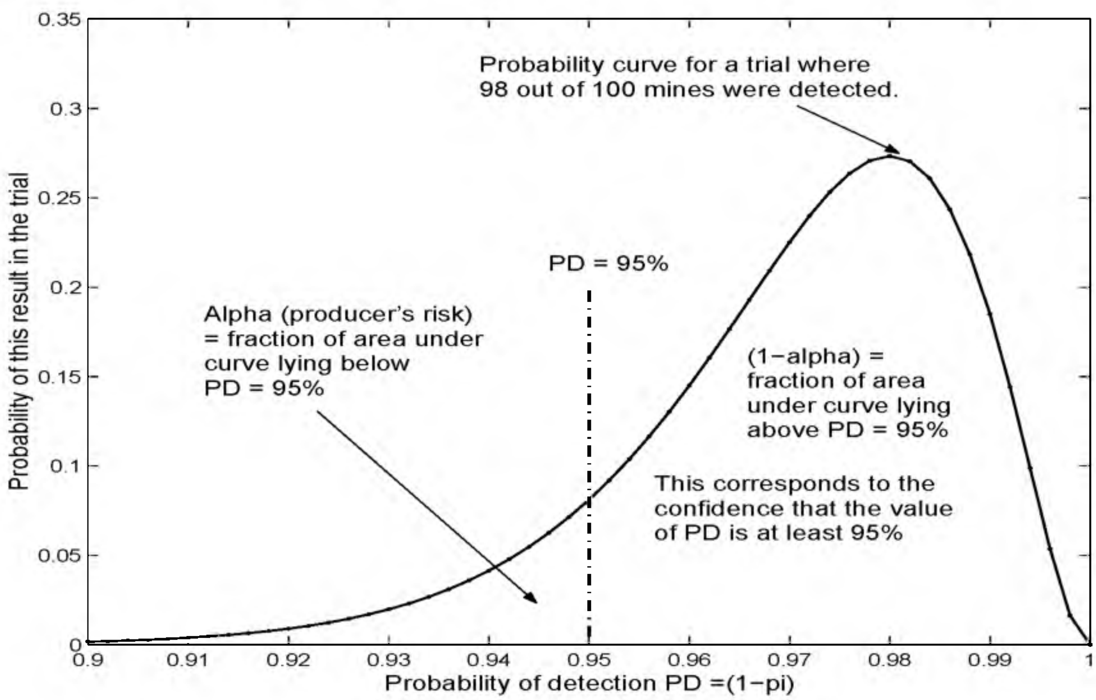


Figure 2: **Probability of result versus probability of detection for a trial of 100 mines, to illustrate α for a p_D of 95% .**

became cheaply available in recent years and was therefore widely covered in textbooks. The solution to the calculation for evaluating p_D from the results of a trial of a detector can be performed by a direct numerical approach of seeking solutions that fit. Although this may appear clumsy and inelegant it produces unequivocal results without the need to make further assumptions about the data, and once the computer programming has been done it is a very quick method. To avoid computational difficulties that arise when the sample size is large, the result can be calculated using the identity $b(x; n, \pi) = b(n - x; n, 1 - \pi)$.

Numerical solution of the Binomial approximation leads to the conclusion that Voles [Vol98] may have misinterpreted one of his own assumptions. Not only is a numerical solution to the case of no undetected mines possible (which is not the case for his method) but the results obtained for the case of zero undetected mines are the same as for the results he offers for the case of zero or one undetected mines. It appears that his assumption that the number of mines undetected is *less than or equal to one* should in fact be that the number undetected is *strictly less than one.* The interpretation of the limiting case is that the mine detector just managed to find all the mines and with any reduction in p_D would have missed a mine, instead of the case that the detector just failed to find one mine and any increase in p_D would have led to finding all the mines.

3.5 Selection of p_D and confidence

The selection from the results of an appropriate p_D and its associated confidence depends on the circumstances of the testing. The result of a trial is the set of points that form the curve of p_D versus (1-α) for the number of mines detected and undetected, as shown in figure 3. While investigating a new detection

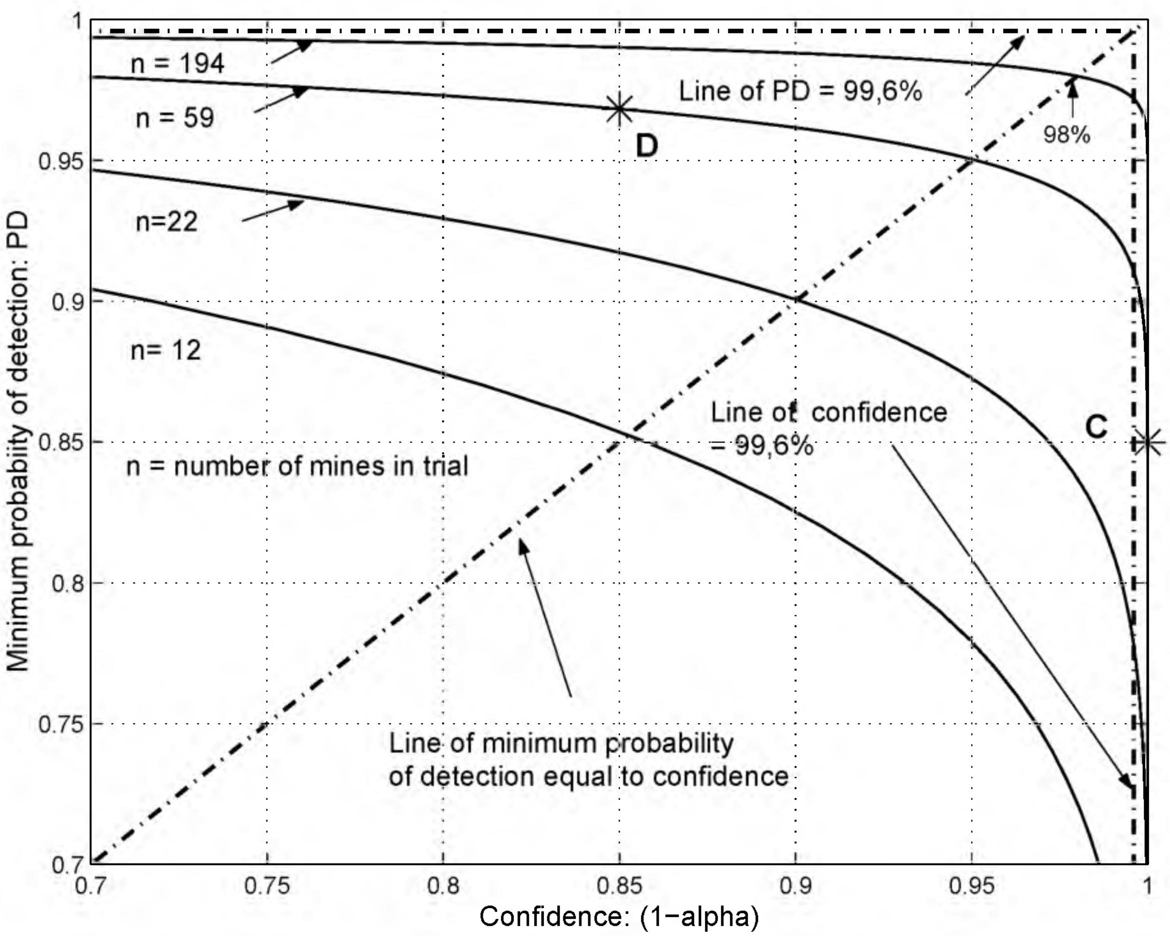


Figure 3: **Minimum probability of detection vs confidence in the result, for trials in which 100% of the mines are detected.**

Confidence (1-α) = Minimum probability of detection, %	**Number of mines in trial**		
	All mines detected	1 mine not detected	2 mines not detected
50	1	3	7
75	5	10	18
80	8	14	18
85	12	22	35
90	22	38	59
95	59	94	139
98	194	290	411
99,0	459	662	913
99,2	602	861	1177
99,4	851	1204	
99,6	1378		

Table 1: **Number of mines required in trial for a given minimum probability of detection, when minimum probability of detection is numerically equal to confidence in the result.**

method it may be initially useful to build prototype equipment that is known to have an unacceptably low p_D in order to investigate how, under field conditions, this p_D varies with soil type, temperature, moisture or other factors. In this case a higher confidence and lower p_D would be appropriate. On the other hand, during detailed testing of a detector that is known to perform extremely well setting the p_D to the desired performance level and calculating the resultant confidence level might appear a better strategy.

Figure 3 shows that the curve is not symmetrical about the line of equality (p_D = (1-α)) and whereas in a 100% successful trial of 59 mines a probability of detection of 85% can be stated with practically 100% confidence — point C on graph, a confidence level of 85% corresponds to a p_D of only 97% — point D. This serves to re-emphasise the difficulty of obtaining any meaningful results from measurement of the rate of detection in reasonably sized trials. The artificial separation of the two parameters of a test result, and the difficulty of the concept of confidence compared to the relative simplicity of probability of detection, lead to correct, but misleading, claims of such figures as 95% or even 100% success in trials. A p_D of 100% can only occur when the confidence is zero but the asymmetry allows confidence levels of 100% to be closely approached at values of p_D of well over 50%.

A useful approach to reduce the difficulty of presenting the results of testing mine detection equipment is to calculate a single value for both p_D and confidence by using the point where the two are numerically equal. The locus of these points is shown on figure 3 as a diagonal line. Solving numerically for the limiting case of $(1-\alpha) \geq \pi$ (i.e. confidence equal to or very slightly greater than probability of detection) gives the results shown in figure 4. Table 1 cites examples from the data. A trial of reasonable size can be seen to have p_D well below 100% by this method, finding 22 mines and missing none in a trial has a p_D (= confidence) value of just over 90%. In practice this is probably a more useful measure than expressing the same result as, for example, "99,6% p_D at a confidence of just over 8%." A small improvement to the equipment is unlikely to change the 99,6% p_D figure by a significant amount but will be clearly indicated by an increase in the value of joint value for p_D = confidence.

Table 1 shows the size of a trial needed to demonstrate a probability of detection and confidence level both equal to 99,6% to be 1 378 mines successfully detected and none missed. This is completely unrealistic.

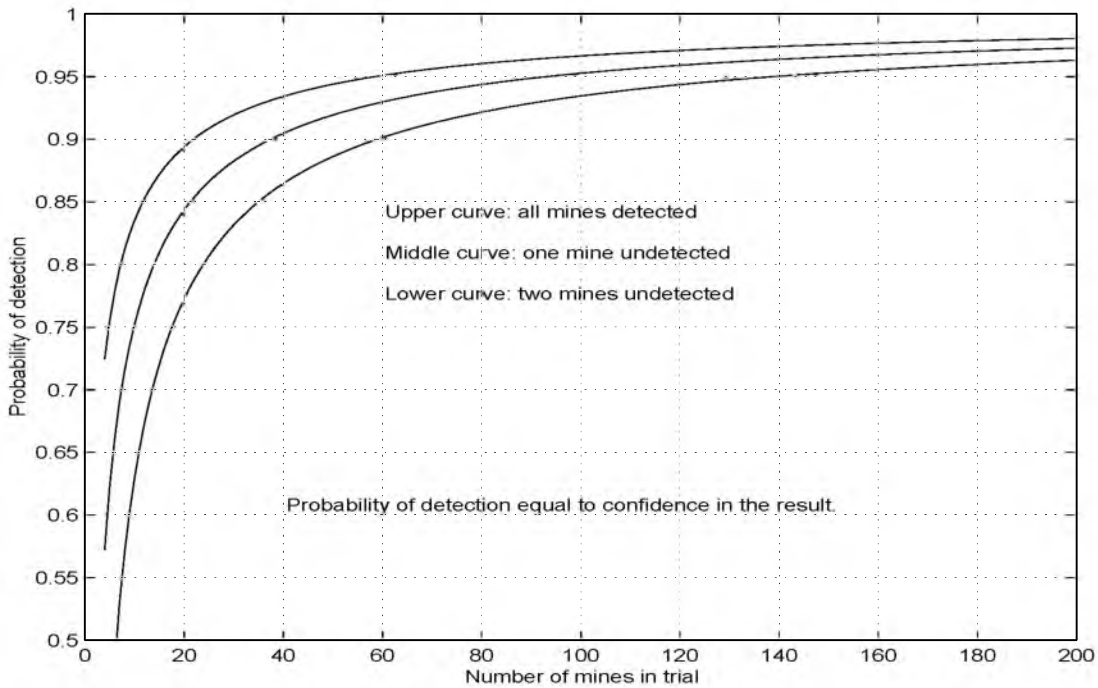


Figure 4: **Probability of detection vs size of trial when the minimum probability of detection is numerically equal to the confidence in the result.**

4 Qualitative factors

Testing mine detectors "in the field" under realistic circumstances introduces so many variables that trials of a size that are feasible cannot be expected to yield results within an order of magnitude of the desired failure-to-detect rate. More information than just the crude detection rate must be included, either in a comparative manner or analytically, if the results are to be useful.

4.1 Limitations of "crude detection rate" data

The approach of defining a single overall detection figure determined by limited testing is not only unworkable in practice but conceptually flawed. Manufacturers of safety-critical systems such as passenger aircraft or nuclear power plants do not define the probability of failure of their products by waiting for occasional failures and then projecting a probability of failure rate from the results. Whilst the development of an exact method for testing humanitarian demining equipment is beyond the scope of this working paper, it is clear that mine detectors are not the only safety-critical equipment that requires testing, and that sophisticated methods have been developed to deal with this situation in other industries. The most distinctive characteristic of humanitarian demining is the wide variability of the environment. This requires focusing on the development of test methods and procedures that are as insensitive to the environment as possible. In this regard a crude detection rate of 99,6% is a very poor measure as it depends heavily on the environment, and on the training, skills and supervision of the operator and not just on the performance of the mine detector.

4.2 Human factors

The limiting sensitivity of a mine detector can depend in part on the skills and hereditary characteristics of the operator. The ability of different people to detect a small change in the pitch of a tone is known to vary by more than an order of magnitude [VLFM99], yet many metal detectors use a change of pitch to indicate a target.

The operator interface of mine detecting equipment has a large impact on how the equipment is used and ultimately the probability of detection. There are many non-technical aspects to designing equipment with an intuitive feel; this topic is beyond the scope of this working paper.

4.3 False alarms and unwanted alarms

Analysing the results of testing humanitarian demining equipment requires a workable method for dealing with false alarms. Taking once again the example of metal detectors, increasing the sensitivity increases the probability of detecting a minimum metal mine, but also increases the number of small pieces of scrap metal detected. In areas where it is considered that there are no minimum metal mines, deminers are known to reduce the sensitivity of the metal detectors to decrease the false alarm rate. Yet in doing so they are making a decision to reduce p_D.

At present no distinction is made in defining the false alarm rate between mine-like objects that are identified by the detector as possible mines, and false alarms due to interference or sensitivity to non-mine-like objects. The statistical analysis presented above does not include any analysis of false alarm data.

For example, a small piece of scrap metal found by a metal detector is considered a false alarm just as an area of highly mineralised soil that triggers the detector is considered a false alarm. For the purpose of analysing the detector's performance these are two separate cases. The small piece of metal might be the firing pin of a minimum metal mine that had already detonated; clearly it falls within the range of items that should be detected by this method and as such is an unwanted alarm but not a false alarm. To distinguish an unwanted alarm from a mine, a separate detection method must be used which seeks to identify a different characteristic of the mine such as its dielectric constant in the case of radar systems, the presence of explosive in the case of olefactory and quadrupole resonance systems, or the physical presence of a mine case by prodding and excavation.

To a deminer both types of alarm are a nuisance. However, for the purposes of analysis one is an unwanted but correctly identified alarm and the other is truly a false alarm. If false alarms can be clearly distinguished from unwanted alarms then the high unwanted alarm rate of metal detecting can be used to provide an improvement in the statistical basis of quality assurance. By considering all unwanted alarms as valid targets in a statistical analysis, the number of targets is increased to a level at which a measure of the crude rate of detection can begin to offer some meaningful results. However, this is not enough of a benefit to outweigh the nuisance of large numbers of unwanted alarms.

Currently, it is common to have a detector with the highest possible probability of detecting both mines and unwanted alarms in order to reduce the number of mines not found (i.e. p_D is at a maximum). A subsequent discrimination method is then used to distinguish between mines and unwanted alarms. The common secondary method in humanitarian demining is to excavate manually and look for a mine, which is costly and tedious. The maximum usable p_D may in practice depend not only on the equipment under test but also on the associated secondary discrimination method and the operating procedures used. This makes a meaningful statistical measure of p_D more difficult to define.

Area cleared, m^2	Mines found	UXO found	Metal fragments found
19 489	5	7	20 015
76 264	6	9	35 931
35 290	6	93 (in 20 groups)	72 220

Table 2: **Number of mines/UXO and scrap metal items found in three cleared areas in Cambodia.**

5 Quality Assurance

Sampling methods of quality assurance (QA) are not appropriate to demining, yet they have been used. Some humanitarian demining organisations have, for example, carefully rechecked a part of a cleared area to be sure that they are confident that the demining has been done thoroughly. The commonly adopted current practice of removing all metal fragments from an area and then performing a check that it is entirely free from metal is a non-analytical form of QA.

The scarcity of mines in a mined area can be seen from the data from three mined areas visited in Cambodia in 1999, presented in table 2. It is clear that with any reasonable probability of detection there will be few, if any, mines/UXO overlooked. If one mine has been overlooked then the chance of it being found in a sampled QA test area is minimal — if 1% of the entire site is re-sampled then the chance of finding the missed mine is 1 in 100. If there were five mines found in the entire area and one not detected this amounts to a probability of detection, p_D, of only 83%. QA methods must demonstrate a high probability of being able to detect such a low p_D, certainly far more than just 1%.

Statements about the required clearance rate of 99,6% lack value if only five mines have been found; the confidence level with p_D equal to 99,6% in a trial of five mines, even when they are all found, is tiny — less than 2%.

There are also important issues concerning the need for an independent mine locating technology to be used for QA (i.e. one that is unrelated to the primary detection method) to avoid missing the same mine for the same reason. One of the more promising methods is the use of dogs or artificial noses which can tell if explosive vapours are present in an area without necessarily being able to locate any individual mine. Any suspect area can be rechecked by hand.

In practice, the only way to guarantee mine clearance to a very high standard is to introduce not post-clearance sampling methods but methods that evaluate the performance of the demining operation as it is taking place. One way of doing this by using the concept of the "margin of detection" is proposed in section 6 below.

The strict operating procedures and supervision of manual demining are a method for maintaining an adequate quality of clearance. It is generally not the quality of current clearance methods that is a problem, but the time and cost of achieving the required standard and the lack of a way to guarantee that it has been achieved.

6 "Margin of detection"

In a typical trial two metal detectors may both find a set of targets without failure. However, connecting suitable measuring equipment, such as an oscilloscope, may show that one is at the limit of its ability to distinguish the targets but the other has a substantial reserve of performance and could still detect the targets under substantially more demanding conditions. Clearly their performance is not identical but the crude detection rate does not distinguish between them. To do so requires the introduction of a measure of how close the "signal" from each mine is to the limit of detection of the equipment being used. An estimate of how easily the detector identified all the targets, or almost all the targets, adds considerable useful information to its evaluation.

Similarly, in an area being cleared of mines/UXO, if all the mines already located have been found easily, and the geographical conditions are similar throughout the area to be cleared, then it is possible to be confident that further mines of a similar type and depth could be found readily. However, if the mine detector had been functioning at the limit of its performance while one or more mines were detected then it is possible that a mine buried slightly deeper, or one encountered by a tired deminer at the end of a working shift, might be missed.

Thus the concept of the ease of detecting of a target, or the "margin of detection" is one way to resolve the problems of statistically meaningful testing of mine detectors as well as QA.

A further weakness of testing methods that rely solely on the crude maximum detection rate is that a crucial evaluation is made when the desired signal from the mine/UXO is only just distinguishable from the "background" which is noise, clutter, interference and other undesired signals, depending on the detector type, soil, vegetation and other factors. The measurement of a signal barely different from the background is unlikely to give reliable or repeatable results. To avoid this it is common throughout engineering to use methods of extrapolation; the signal is measured under less critical conditions and a curve fitted to the results which is then extrapolated to define a point at which detection is just possible. The goodness of fit of the curve can be analysed statistically to provide measures of confidence and probable error limits. Individual manufacturers of mine detectors may well be using this method to enhance their products, what is required is a more general technique which can be used to compare different detectors.

A suitable measure of the margin of detection might be the ratio of signal to background noise at some point in the detector circuit. For complex detection methods a measure of the effective "signal to noise ratio" can be made from probabilistic considerations. However, in defining the conditions of the measurement of the margin of detection the number of variables is large. The sensitivity of the detection process to some of these variables is also large; in the case of the limiting distance to a small metal target in air, the metal detector's received signal depends on the inverse fourth power of the distance from the detection coil to the metal fragment. Moving from 100 mm to 110 mm causes a reduction in the received signal of 32%. A small pebble on the surface of a test area can clearly cause significant variation in the results depending on whether operators touch the pebble and push it aside or raise the detector over it.

The margin-of-detection parameter attempts to give a readily understandable result for a wide variety of targets. It can be evaluated by presenting a range of known targets to the detector under controlled conditions. Extrapolation of these results should give an acceptable estimate of the limits of the performance of the detector under ideal conditions. The performance of a known detector in the field can then be used to define the effect of the field conditions using the ratio of margin of detection. For example, a soil with a high metal ore content (e.g. laterite) might be considered three times more difficult for detection using a metal detector than a sandy soil, or dry sand might be considered three times more difficult for a radar

Ratio of the power of the signal to the power of the background noise	Ratio expressed as decibels (dB)
100 000	+50
10 000	+40
1 000	+30
100	+20
50	+17
20	+13
10	+10
2	+3
1	0
0,5	-3
0,1	-10
0,01	-20

Table 3: **Power ratios expressed as decibels (dB).**

system than moist soil.

These ratios may be conveniently expressed in decibels (dB) which are defined as $10 \times log_{10}(ratio)$. Table 3 gives decibel values for some ratios of the strength of the signal to that of the background, measured from the power of each.

In practice, an easy-to-detect target might be defined as 45 dB above background, and a difficult to detect target perhaps 5 dB above background using a standard mine detection system. A different detector that is being evaluated could then be measured and might perhaps give figures of 40 dB for the first and 6 dB for the second; this detector is more capable of finding the smaller target. Such ratiometric measurements allow direct comparison with existing “reference” equipment with which deminers are familiar, thus producing numbers with an immediate practical application.

Similarly, by inserting a standard target to a known depth, field conditions can be measured. A standard target might be described as producing, for example, a level 3dB above background in a particular area, whereas it is known to produce 5 dB above background under “standard conditions”. This area thus has a background level 2 dB worse than standard conditions; the impact of this on the maximum depth of detection of small targets can be directly calculated. As most demining organisations check the operation of equipment at regular intervals against known targets this operation adds little extra effort to the work of demining.

6.1 Application of “margin of detection” to QA

When applied to quality assurance the concept of margin of detection has considerable appeal as it provides a working check on both the equipment and its operation.

If the depth of a target, either a mine/UXO or an unwanted alarm is known then the margin of detection can be used to provide a measure of the level of confidence of finding another similar target at the required depth of clearance in the same soil. If, in the case of a metal detector, the target is a small piece of scrap metal that is excavated, it can be compared to an objective detection performance in air by measuring the margin of detection for the object at a known distance. This can be readily measured by dropping the scrap metal into a plastic container of known depth and placing the metal detector on top of the container. For example, if a particular target gives a margin of detection of 10 dB under the test conditions and was found at a particular depth then predicting the probability of detection of known minimum metal mines at a similar depth should be possible. Many of the factors that affect detector performance at the particular site can be combined into a single measure by following such a procedure.

By maintaining a record of all items found and the corresponding margin of detection, the performance of the equipment and operator can be continuously evaluated and a statistically meaningful quality assurance may be possible.

6.2 Use of unwanted alarms and margin of detection to enhance QA

Quality assurance could be immediately enhanced by introducing a simplified form of measurement of the margin of detection. This can be directly implemented with existing detectors. In terms of metal detection, small metal fragments can be characterised after excavation by the distance at which they can be detected in air; this gives a simple measure of their "detectability," and depends on their size, shape and composition. If the depth at which they were detected in the soil is noted, the limits of detection for the particular combination of soil, mine detector and operator can be approximately categorised. From this simple analysis it is possible to verify that a target similar to a minimum-metal mine would be found at a depth that would give adequate safe clearance of all mines. Research into this method should be able to determine if a rule of thumb that is easily memorised can be deduced, or a simple tool based on a nomogram developed. Simple categories that can be readily coded as "acceptable'," "marginal," and "unacceptable" could be used, with appropriate colours or symbols.

Overall, measurement of the margin of detection is a simple and powerful tool that could be applied to both detector evaluation and quality assurance methods in demining.

7 Summary and conclusions

Producing useful results from the evaluation of mine detection systems is surprisingly difficult; testing prototype equipment of unproven reliability in live areas may be hazardous, and testing in safe areas offers limited realism.

The provision of tests of sufficient size to give a statistically meaningful outcome is difficult and the impact of the test conditions and operator training may dominate the results.

Measurement of the crude detection rate of mine detectors in realistically sized trials fails to offer meaningful information on their performance. This paper presents a rigorous statistical analysis that demonstrates this, using a clear conceptual model.

An alternative to quoting *probability of detection*, p_D, and *confidence in the result*, (1-α), data separately is suggested and examined. Combining these two aspects of the result by using the value of p_D that is numerically equal to (1-α) provides a simple way of expressing the two crucial measures of a test result in a single figure, and permits a simple and direct comparison between different tests.

The large impact of qualitative factors such as the effects of the environment and the operator are discussed, and the need for test methods that are not highly dependent on these factors is explained. This is particularly important when using crude detection rate data.

The problem of achieving a statistically significant method of quality assurance after demining is examined and found to be so severe that such post-clearance QA is infeasible.

The concept of "margin of detection" is offered as a potential way to resolve some of these problems. Its advantages and implementation are discussed in relation to both testing mine detectors, and QA during clearance. Simple ways of implementing a crude form of margin of detection that are compatible with existing equipment and operating procedures are outlined, and the need for further research into possible ways of implementing the concept identified.

References

[Ano00] Anonymous. Croatia Sets Feb. 11 "ELF" Lab Tests to Validate Loch/ChemTech Landmine Detector. Internet Website http://biz.yahoo.com/bw/000201/tx_loch_ha_1.html, Feb 2000. Valid as of 12 Feb 2000.

[Bar94] J Wesley Barnes. *Statistical Analysis for Engineers and Scientists.* McGraw Hill, 1994. ISBN 0 07 839608 5.

[CP34] C J Clopper and E S Pearson. The use of confidence or fiducial limits illustrated in the case of the binomial. *Biometrika*, (26):404–413, 1934.

[JK69] Johnson and Kotz. *Discrete Distributions.* Wiley Interscience, 1969.

[MF85] Irwin Miller and John E Freund. *Probability and Statistics for Engineers.* Prentice-Hall, third edition, 1985. ISBN 0 13 711938 0.

[VLFM99] N L Vause, T R Letowski, L G Ferguson, and T J Mermagen. Auditory issues in handheld land mine detectors. In Abinash C Dubey, James F Harvey, J Thomas Broach, and Regina E Dugan, editors, *Detection and Remediation Technologies for Mines and Minelike Targets IV*, volume 3710, pages 778–786. SPIE, Apr 1999.

[Vol98] R Voles. Confidence in the assessment and use of mine detection systems. In IEE, editor, *The Detection of Abandoned Landmines*, volume 458 of *Conference Publications*, pages 28–30. IEE, Oct 1998.

[Yam73] Taro Yamane. *Statistics, An introductory analysis.* Harper and Row, third edition, 1973. ISBN 06 047313 4.

Domestic Rainwater Harvesting
Research Programme

DTU

VERY-LOW-COST ROOFWATER HARVESTING IN EAST AFRICA

(Based on a Feasibility Study performed in the Great Lakes Region during May – July 2000)

D.G.Rees, S.Nyakaana & T.H.Thomas

Working Paper No. 55

September 2000

THE UNIVERSITY OF WARWICK

Development Technology Unit

School of Engineering, University of Warwick

CONTENTS

Appendices

DEFINITIONS, ABBREVIATIONS & COSTING UNITS

A = area of roof (in m^2)

C = cost per litre

D = daily demand (not necessarily constant through the year)

DRWH = domestic roofwater harvesting

Dry season = all days when total runoff in the preceding 14 days < 7 R

E = W / (P x A x F) = efficiency of rainfall capture

F = 'Run-off fraction' = Water volume reaching the downpipe ÷ volume falling on roof (e.g. .85)

K = ratio of dry season water value per litre to wet season value

lpcd = litres per capita per day

P = annual precipitation (in mm)

Q = rainwater harvested (in litres per day per household)

R = P x A x F / 365 = mean daily runoff

RWH = Rainwater Harvesting or Roofwater Harvesting

S = 'security' of supply = fraction of days demand is satisfied

Sf = W ÷ ΣD = fraction of demand volume that is satisfied

T = V ÷ R = tank volume expressed in 'days mean supply'

Target Area = S Uganda, NW Tanzania & Rwanda

U = W ÷ V = utilisation factor for storage (in number per year)

V = volume of tank (in litres)

VLC = very-low-cost (say <$50 per system)

W = annual water supply volume obtained from RWH system (in litres per year)

Wet season = all days when total runoff in the preceding 14 days exceeded 7 R

Costing has been expressed in £ sterling (or in pence sterling 1p = £0.01). At the time of the study the approximate conversion rates into the three local currencies were:

£1.00 ≡ USh.2250/- ≡ TSh.1190/- ≡ RWF540 ≡ $US1.50

1. INTRODUCTION

Although numerous new water supplies have been constructed in rural Africa in the last decade, population growth has resulted in only a small projected increase in the fraction (32% to ca 36%) (WHO/UNICEF,1996/2000) of households having ‘access to adequate quantities of safe water’. Moreover official statistics are based on understandings of the words ‘access’, ‘adequate’ and ‘safe’ that seem inappropriate to rural Africa. ‘Adequate’ is taken to mean over 20 litres per person per day (lcd) and ‘access’ is taken to mean a water source within 1 kilometre of the home. Actually 20 lcd is well above current usage and immediate aspirations; it is quite incompatible with a carrying distance as long as 1 km. Collecting even only 10 lcd for a household with 6 members requires 3 round trips per day. If the source were 1 km away, this would take at least 2 woman-hours per day (collection is predominantly by women and children). “Water equals walking” has long been an accurate adage in rural Africa. It will be decades before point sources like wells or standpipes are sufficiently numerous, and hence close-spaced, that walking for water is no longer a major household burden. Yet most water programmes are still solely concerned with providing new point sources, often using techniques that have proven operationally unsustainable.

In recent years rainwater harvesting, for long an informal water technology (Agarwal, 1999) has been acquiring a higher official status world-wide. Its main domestic form, *roof*water harvesting (DRWH), has been aided by the rapid growth in the use of hard roofing (usually corrugated iron sheeting) in areas formerly dependent on grass roofing. DRWH has thereby become feasible in most of Sub-Saharan Africa. It is a technique with the great attraction of delivering water to the very door of the user’s house. Its main perceived disadvantages are its high cost and its individual nature. The former however only applies to some forms of DRWH, not all, and the latter’s unattractiveness to promotional agencies like NGOs is diminishing as they lessen their former overwhelming emphasis on *group* enterprise.

The Target Area (see map in Appendix III) of this Feasibility Study is technically favourable for very low cost DRWH, by reason of its good rainfall and convenient rainfall distribution. Even so, DRWH is likely to be affordable only when it is combined with some other ‘back-up’ source. Fortunately multiple sourcing can be shown to be already a common rural water practice across much of the tropics. The Target Area has characteristics that make existing water sources rather unsatisfactory. Its poverty means that clean sources are few in number. Its topology results in a paucity of perennial streams and springs and arduous carrying conditions in most places. The water table is commonly deep except near swamps (where dwellings are understandably sparse) and in some areas the ground water is so mineralised that it is dangerous to ingest or objectionable in taste.

In association with local organisations in Africa and tropical Asia, the Development Technology Unit at the University of Warwick has been researching DRWH systems for some years, looking for better understanding, lower costs and higher performance. It has found that DRWH development has reached a point where ‘partial’ DRWH systems could be affordable by the bulk of rural households in the Target Area. Such systems would typically increase a household’s annual water consumption by 50% while reducing its water-fetching time by 70% - at a cost as low as $\$40_{US}$ which equals about half the cost of roofing a small house. We may call this form of RWH ‘very-low-cost’ roofwater harvesting (VLC DRWH).

A Study – financed jointly by the Laing Trust and by the University of Warwick – was therefore initiated in May 2000 to evaluate its apparent promise, in an area where rainwater currently providing under 2% of household water because it is largely restricted to expensive forms. The purpose of the study was to confirm or rebut the apparent promise of VLC DRWH, examining both its performance and its unsubsidised affordability by the bulk of rural households.

The form of the three-month study, whose headquarters were an organic farming training centre, Kyera Farm, near Mbarara in southern Uganda, was;

(a) to field-test and refine candidate VLC DRWH technologies, and

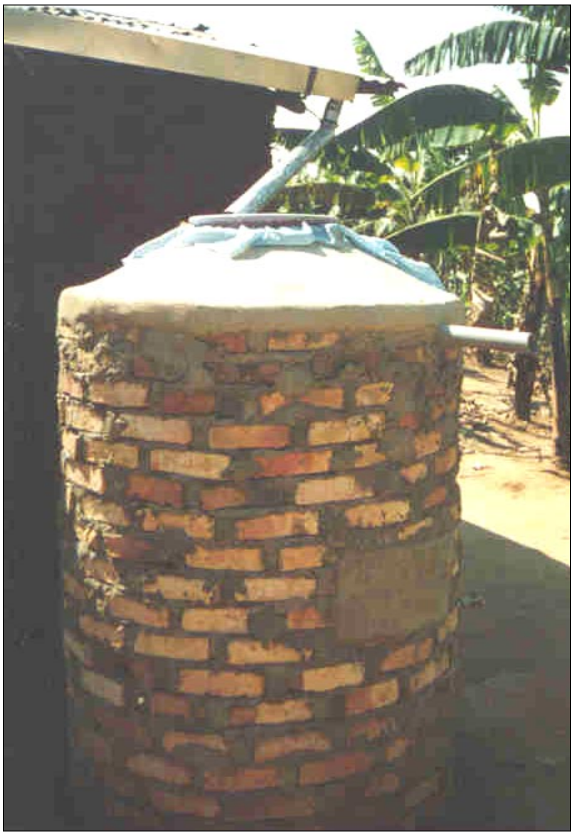
(b) to interact with 9 agencies already involved or interested in RWH in the target area.

At a concluding seminar in July 2000, the findings were presented to all these parties for their information and comment. The Study was intended not only to assess the desirability of switching to much smaller DRWH designs than used hitherto, but also to prepare for a major programme with such agencies to kick-start the adoption of VLC DRWH in the region.

In the ensuing sections technical, economic and social analyses are presented, backed by appendices containing more detailed data such as design drawings of novel system components.

The DTU team of three are very appreciative of the financial support of the two Study funders (Laing Trust and The University of Warwick), the energetic input from local staff in Uganda and the unstinting collaboration of the partner agencies in Southern Uganda, Rwanda and NW Tanzania.

2. THE CANDIDATE TECHNOLOGY (VLC-DRWH)



A roofwater harvesting system comprises a roof, a storage tank and a means such as guttering of connecting the one to the other. Other possible components are filters or ‘first-flush’ diverters to reduce the quantity of dust or debris entering the tank, access points for cleaning, a means of extracting water from the tank and in-tank devices to aid water management or to maximise water quality.

The most costly system element is usually the store (tank), which in a ‘stand-alone’ system may be designed to hold all the water required throughout the longest expected dry season. Middle class households in the *humid* tropics might have upwards of 10000 litres of storage, while in a Monsoon (summer rains) climate storage may be two times larger. Such large structures are expensive unless use can be made of some natural rock foundation. Underground stores are less space-consuming and are generally cheaper, volume for volume, than surface mounted stores; however the former need a pump and are prone to failure modes that are difficult either to calculate or monitor.

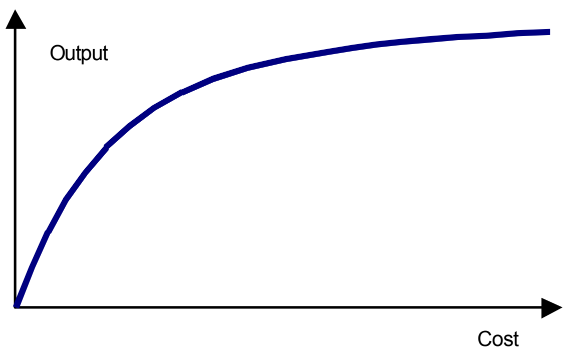
The quality of the collected water is usually quite high although it drops following the arrival of the first rains after a dry season due to dust on roofs. Bird droppings and other contamination may cause a sharp temporary rise in such pollution measures as counts of faecal coliforms. In rural areas it is thought that contamination by human pathogens is uncommon - 30% of farmers have long drunk roofwater in Australia, a country with high environmental health standards - but untreated roofwater does not reach the strict standards used for urban supplies in industrialised countries. Besides bacterial quality, there are other health and taste factors affecting DRWH and these are discussed in Section 6.

A striking feature of DRWH systems is the strong law of diminishing returns that operates for tank sizing. As the graph below shows, a system containing a very small tank (holding only 7 day’s household consumption) might yield 75% of the water per year of a system with a very large tank (capable of holding 100 days’ consumption). This suggests a route to cost minimisation, provided that an alternative, albeit more costly per litre, alternative back-up water source is available. In rural areas such a back-up supply is likely to be the distant spring, well or pond formerly used.

Besides keeping tanks very small, economy measures in LDCs include constructing tanks more efficiently, using cheaper materials, devising slimmer gutters and downpipes, substituting low-cost local labour for more capital-intensive production and devising management strategies that minimise the cost:benefit ratio. These matters are discussed in Section 4.

Currently DRWH systems in developing countries commonly use, for their storage element, mortar jars, ferrocement jars, cylinders and cuboid shapes of plastered brick, oil drums and corrugated iron cylinders, reinforced concrete tanks or (for richer households) plastic drums. Usually these are not tightly designed and have failure safety factors that are uneconomically high. (Very large tanks however need and get engineering design, as their failure can be dramatic and dangerous.) There is therefore considerable scope for material savings. Little-used materials of promise are stabilised earth, plastic sheets and the ground itself, especially if designs separate the functions of strength and water-tightness. Prior to this Study the DTU had identified three promising designs for small stores (500-800 litres) as well as a large (8000 litre), but relatively low-cost, 'partly-underground' tank, with associated village-manufacturable pumps. It was estimated prior to the Study that complete systems containing 750 litres (ca '7 days') of storage might be producible in the Target Area for $\$40_{US}$ and 2000 litre systems could cost under $\$80_{US}$. These estimates took into account the high prices of items like cement in the Area. Field-testing and construction were needed to confirm these estimates and affordability studies focussed on identifying what figure to design to.

Figure 2.1: 'Diminishing returns' Output v Cost curve



Fortunately DRWH is amenable to 'staged' construction, with guttering and storage being increased in steps over several years. In Thailand and Cambodia it is common to see a house surrounded by several large, mortar, rainwater jars - presumably not all installed at the same time.

Development agencies are understandably nervous of involvement with technologies having a bias to the rich, and DRWH has been accused of that tendency. Concentrating on small systems is one way of counteracting that danger. Developing the ability to service the grass roofs of the poorest would be another. Water can be collected from crude thatch, but it is coloured and turbid and its capture requires wide gutters. There are some directions for possible progress, including clarifying the stored water and employing sheet-plastic gutters. For the Study reported here, ability to work with grass roofs was decided to be desirable but not essential if the fraction of homesteads with hard roofs were found to exceed 70% and to be still rising. In fact the fraction of homesteads in the Target Area with at least one hard roof does generally satisfy this test: the prevalence of iron roofs in particular has risen dramatically in the last decade.

3. WATER NEEDS IN THE REGION AND THE POSSIBLE CONTRIBUTION OF VLC DRWH

3.1. Existing water sources and collection times

Data on water sources in the Target Area is not very readily available. In Rwanda there is a national register of springs. In Uganda the 1991 census recorded some relevant data from which the following tables for (old) Mbarara District, whose population was then 0.9 million people, were constructed.

Table 3.1: Use of different water sources in Mbarara District

Source type	Percentage of households using
Piped water	3.3
Boreholes	5.5
Protected well/spring	8.6
Open well/spring	45.2
Stream/river	19.6
Lake/pond/dam	17.4
Total 'clean'	**17.5**

Source: Uganda Nat Census (1991)

Table 3.2: Distribution of roofing types in Mbarara District

Roofing type	Percentage of households using
Iron	37.2
Tile/asbestos/concrete	0.7
Total 'hard' roofs	**37.9**
Grass/papyrus	39.2
Banana	22.9
Total 'soft' roofs	**62.1**

Source: Uganda Nat Census (1991)

Since 1991 however, Uganda has undergone significant economic and demographic growth and there has been some improvement in the fraction of households using such 'clean' sources as protected springs, protected shallow wells and boreholes. Piped water supplies in the few serviced urban areas have also improved. Nationally the fraction of the population having 'access' to clean water in 1994 was deemed to be 47% for urban areas and 32% for rural areas. Figures for Tanzania and Rwanda have not been obtained, neither country is listed in the source below. However the source suggests that for Africa as a whole clean water coverage has changed little from 54% over the last decade.

(Source: WHO/WSSCC/UNICEF http://www.thewaterpage.com/coverage_figures.htm)

Locally in the Study Area there are areas like Rakai District where highly mineralised groundwater forces reliance on surface sources such as swamps.

The Ugandan building data above is seriously 'out of date'. It indicates that only 38% of roofs as being suitable for RWH whereas the proportion of dwellings for which at least one building has a hard roof is now probably over 65%. None of the 12 NGOs contacted, all working in DRWH, felt that absence of hard roofs was a serious restriction in the uptake of the technology. Corrugated iron roofs, that cost about £1 per m^2 of building plan area, have become the norm for both housing and institutional buildings.

Rwanda is characterised by very steep but not mountainous terrain. In much of the country hillside springs have been the traditional water sources, augmented in the 1970s and 1980s by gravity-fed distribution piping. However since 1990 a growing fraction of the population may be found living considerably *above* the spring line and are carrying their water up through considerable heights. In the drier and flatter parts of the country to the East where springs are few, former National Park land has been recently settled by returned refugees. Rwanda was well-known for its attractive fired-tile roofing, today however cheaper corrugated iron has gained in popularity there.

In Tanzania the Target Area comprises Kagera Region, which is much wetter than the national average and thus more prone to have hard roofing. The terrain is less steep than in neighbouring Rwanda and good springs are far less common. Piped water is rarely encountered and shallow wells (some protected, some not) are widely used. The area is very 'peripheral', being 3 days journey from the capital, so material prices are relatively high.

Figure 3.1: A typical traditional water source in NW Tanzania

In all three countries, water collection distances are significant and the terrain is rarely flat. A survey of (only) 120 households in parts of the area is summarised in Table 3.3. From the table it can be seen that it took about 3 hours per household per day to collect water. These figures were based on a mean round trip speed *on the flat* of 65 meters/minute and lower speeds on slopes. This norm may be rather high although it is compatible with the few direct speed measurements made. Queuing time is not included and if it were collection times might be about 25% higher.

In some well-populated plateau locations, water collection in the dry months is especially onerous because convenient sources dry up. Water then has to be hauled up from valley sources as much as 5 km from (and 200 m lower than) the homestead.

3.2. A survey of small-scale RWH systems in Kabarole District, Western Uganda

A survey being carried out in Kabarole district, Western Uganda, gives some idea of the benefits that can be obtained using a small tank. The survey covers 6 households distributed around the district. To date the data for the months of May, June and July 2000 has been collected and analysed. The survey will continue for a further 3 months into the wet season. The survey analysis to date can be considered as a dry season analysis. The jars had been built as part of a study into water quality from cement jars and were 400 – 500 litres in size (the variation due to manufacturing variability).

Table 3.4 shows the initial analysis of the survey data. It is also worth pointing out that the survey form was designed to measure water carried from the traditional water source. The survey therefore measures the minimum benefit, as water consumption is likely to be higher when water is taken from the jar during the wet season.

Table 3.3: Analysis of water-collection distances/times

	Mbarara, Uganda	Biharum'lo Tanzania	Mbarara, Uganda	Karagwe*	Mbarara*	ALL
Agency	IVA	BRATIS	DTU	KARAD'A	MUST	
People (no)	210	385	175	381	359	1,510
Water (litres)	3,400	5,260	1,560	3,440	2,360	16,020
Households	40	60	20	60	60	240
Sources	2	3	3	3	3	12
Time (mins)	4,030	12,639	7,209	12,929	11,250	53,464
Distance (m)	207,300	725,894	640,364	789,739	415,580	2,586,768
Notes				*December	*November	

Averages for all households/people

people per H/H	number	6.3
water per H/H	litres/day	66.8
time per H/H	hour/day	3.7
distance per H/H	km/day	10.8
water per person	lcd	10.6
time per person	hour/day	0.6
distance per person	km/day	1.7

It can be seen that the percentage coverage for the period looks quite low, only 35% to 57.5%. It should be noted, however, that the percentage of rainy days during the period (19% - dry season) is low compared with the annual average (35%) and so the annual coverage figures will be higher. Also it can be noted that where the number of persons in the household is low, the savings are greater (with the exception of Kaahwa).

An indication of the walking time and walking distance shows that particularly high savings can be made when the distance to the traditional source is high (Katenta and Kayula), or where lpcd consumption is high (Mugisa) which is obvious. The actual daily time and walking saving are very significant – 55 minutes and 125 minutes being the outstanding examples.

It is interesting to note that there is no strong correlation between distance walked and lpcd consumed, which is generally believed to be the case. The lpcd figures do correlate well with estimated consumption figures for the region and with observations made by the authors

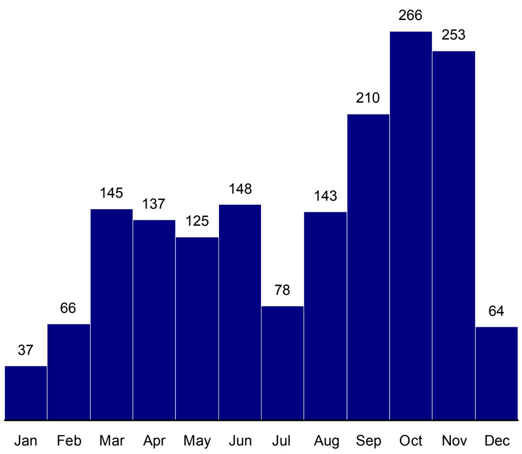


Figure 3.2: Average monthly rainfall for Kyenjojo, Kabarole, Uganda

Table 3.4: Analysis of survey data

		Kandole	Katenta	Mugisa	Kaahwa	Karamagi	Kayula
General data	Distance to source (m)	200	500	400	400	300	1500
	Number of occupants	6	5	4.5	4	9	8
	Roof area (m sq)	20	27	24	22	24	30
	Total days considered	106	106	92	85	92	106
	Rainy days during period	15	28	20	11	21	18
Calculated data	Jerry cans carried from source (daily average)	3.4	1.3	2.5	2.2	3.2	3.3
	Jerry cans consumed (daily average)	5.4	3.0	5.3	3.0	4.9	5.0
	Average lpcd consumed	18.0	12.0	23.5	14.8	11.0	12.4
Estimated savings	Litres (daily average)	40.7	34.5	57.5	18.1	35.5	34.8
	kilometres walking (daily average)	0.81	1.73	2.30	0.73	1.07	5.21
	Minutes walking (daily average)*	19.5	41.4	55.2	17.4	25.6	125.1
	%age total water consumed	37.7	57.5	54.3	30.6	35.9	34.9

*assuming a walking speed of 2.5kms per hour – in the majority of cases the terrain is steep

4. ECONOMICS OF DRWH

4.1. Economic overview

All households already have some access to water from point sources. For some days per year, many also employ 'informal' rainwater harvesting, placing bowls and jugs under eaves or even trees during rainfall.

The introduction of more formal (and productive) RWH will normally be accompanied by three benefits. The most obvious is a reduction in the time spent carrying water from point sources – a reduction more or less proportional to the volume of water no longer carried. The second is an increase in household water consumption wherever it was previously constrained by the effort of collection. The third is a common, although not invariable, increase in water quality. All these benefits rise with DRWH storage capacity, albeit in a way showing diminishing returns.

Figure 4.1: Typical informal RWH using an old 200 litre oil drum at a household in SW Uganda

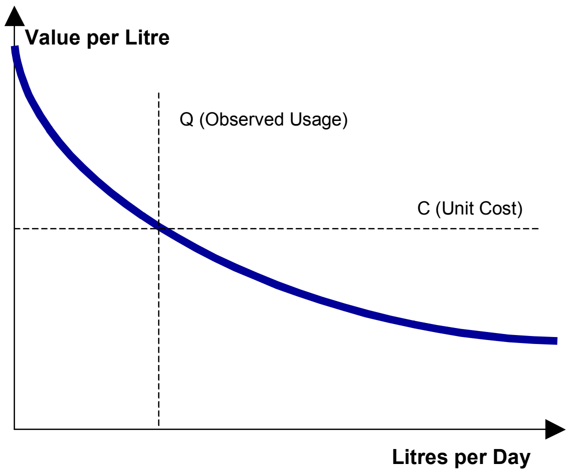
The increase in water consumption with VLC RWH has not been widely measured. Generally any increase is restricted to the wet seasons. DRWH (and VLC DRWH in particular) is not generally capable in the dry seasons of supplying quantities larger than already obtained from point sources: this means that it will be used to supplement, but not to substitute point-source water.

The costs of DRWH are overwhelmingly capital costs, as neither operation nor maintenance usually involves significant expenditure. Storage-tank cost is usually the dominant item, by contrast guttering accounts for only about 25% of the total system cost. These capital costs are subject to economies of scale. The sensitivity (elasticity) of tank cost to storage capacity is about 0.8. The sensitivity of gutter cost to gutter capacity is even lower, so that it is usual to install gutters that are so large (e.g. designed for rainfall intensities up to 2mm per minute) that they can catch all but 1 or 2% of the annual run-off reaching them.

4.2. Value of water

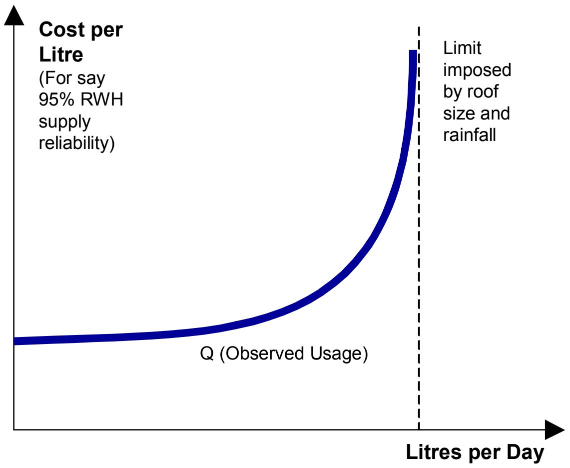
As with many other goods, water has a declining value with quantity. The first litre per day is worth more than the tenth. By examining the limited data available that relates household consumption per day to the effective unit cost of water (i.e. cost per litre), we might construct a curve such as shown in Figure 4.2. Each socio-economic group would have its own curve.

Figure 4.2: value v quantity



The cost line on Figure 4.2 is horizontal, which reasonably represents the situation where water is fetched, each successive litre requiring the same input of labour. Such a line does not fairly represent harvested roofwater, where the effective cost general rises with daily consumption despite the economies of scale in tank construction. A typical cost v volume characteristic for RWH supply is shown in Figure 4.3.

Figure 4.3: cost v volume



Sometimes we can find examples of water purchase and use them to infer the value of water. Richer households, or those experiencing illness, may pay for water to be brought to the house. More usually we have to infer costs indirectly through conversion of fetching distance/height into time and then time into money. Such costs, like the value of water discussed above, will be lower for poorer households than for richer ones.

4.3. Time cost of water carriage

This is a function of a household's distance to, and height above, a water source, of the means of transport used, and of the persons involved in carriage and their respective unit time costs (actual or opportunity). So we will examine each of these factors in turn.

(a) Haulage distance

Table 3.3 shows the results of a small survey of walking distances (users of 6 sources). Although there are some homesteads in the Study Area that haul water from distances greater than 5 km away in very dry months, the dry season average for 120 users of 6 point sources in the Target Area was 1.5 km to the source.

A much larger survey is required to give reliable averages for the whole Area and to measure the seasonal variation of haulage distance, walk time and water consumption.

(b) Height

Point water sources are generally *lower* than homes, so that the laden return journey is usually uphill. A round trip that comprises walking downhill with an empty water container and returning laden uphill is always slower than one of the same distance on the flat. For calculation purposes it would be convenient either to replace any climb height by an extra horizontal distance of equivalent carriage time, e.g. "add 1 km for every 100 m climb". Alternatively we might assign a different round-trip mean speed for each gradient. It is likely that a very steep (return) uphill slope of say 1-in-5 will halve the round-trip mean speed, especially for climbs exceeding 100m. A gentle gradient of say 1-in-30 will have little effect on round trip time. Experiments were undertaken to measure the effect of gradient on walking speed, but they gave rather inconclusive data because the samples were small and it proved impractical to control other variables such as youths' desire to impress, sense of urgency or tiredness etc.

(c) Walking speed

The speed of movement of a person collecting water depends upon many factors and varies between about 1.5 and 5 km/hour.

For short haulage distances some people use a strategy of hurrying to minimise time or arm strain; this strategy cannot be maintained for more than about 200m. Running down a gentle slope with an empty jerrycan, some young people exceed even 5km/hour. Conversely, long uphill hauls require a slow steady pace with regular rests. Young children tire more quickly with distance than adults, even though they usually carry only 3 or 5 litre loads.

For distances over 1.5 km but only where slopes are gentle, pushed or pedalled bicycles are sometimes used to carry 1 or 2 x 20 litre jerrycans (especially by 'commercial' water fetchers) at speeds of about 3.5 km/hour. There is virtually no evidence of water carriage by pack animal in the target area – neither mules nor donkeys are commonly available. Such animals have to be driven slowly (3 km hour) but carry up to 80 litres at a time (recent Mexican experience).

Table 4.1: Survey of variation of walking speed with path steepness

Route No		1	1	2	2	3	3	3	4	5	6
Slope when carrying water		flat	flat	easy down	easy up	med up	med up	med up	steep up	med up	med up
Date	d/m/2000	16-7	16-7	16-7	16-7	16-7	16-7	16-7	8-8	8-8	8-8
Slope angle (up) source-to-house	Degrees	0	0	-5	5	11.5	11.5	11.5	14.2	7.6	9.5
Person carrying	Sex	f	m	f	f	m	f	f	2f	2f	2m
	Age	15	17	16	NK	20+	15	15	14/20	14/20	14/20
Height of 'source'	m	1424	1424	1450	1424	1434	1434	1434	1662	1662	1655
Height of 'house'	m	1424	1424	1424	1450	1478	1478	1478	1844	1702	1688
Rise *H* from source to house	m	0	0	-26	26	44	44	44	182	40	33
Distance *D*	m	300	300	300	300	300	300	300	740	300	200
Time (empty)	T_o secs	240	223	251	232	206	238	245	1114	360	188
Time (full)	T_b secs	270	230	315	270	265	347	297	1875	485	270
Total walk time (round trip)	$T_r = T_o+T_b$ secs	510	453	566	502	471	538	543	2989	845	458
Gradient (full)	H/D %	+0	+0	-8.5	8.5	14.5	14.5	14.5	24.5	13.3	16.5
Speed out (empty)	m/min	75	81	72	76	87	76	73	39.8	50	44.4
Speed back (full)	m/min	67	78	57	67	68	52	61	23.7	37	44.5
Mean speed (round trip)	m/min	70.5	79.5	63.5	72	76.5	67	66.5	29.7	42.5	64
Speed ratio	(full/empty)	0.89	0.97	0.80	0.86	0.78	0.69	0.82	0.59	0.74	0.70
Experiment	Number	A1	A2	A3	A4	A5	A6	A7	B8	B9	B10

Notes:

a. 'easy' slope = 3% to 10%, 'medium' slope = 10% to 20%, 'steep' slope = >20%

b. NK = not known

c. Men walked faster than the females during some of the early trials but were slower on the later trials. The "macho" image may be the reason for the early trials being at faster rate. The slower rates on the later trials may be due to the fact that the men did not pace themselves, having less experience than the females in carrying water.

d. The considerably slower rates on route No4 of 29.7 may be partly explained by the fact that the actual route was longer than 740m as the path meandered its way through the plantations. Also negotiating the rough ground on the way down tended to impede ones progress.

e. There are many factors which could give rise to data varying, some of which may be:
   - Whether the person is aware of being timed or not a (10% increase in the walking speed maybe a reasonable value when the person is aware of being observed)
   - The number of people queuing at the water source
   - The number and length of rests a person takes
   - The flow rate of water at the source (this decreases during drought periods)
   - Who is performing the task, i.e. children are prone to the least distraction whereas a women may walk quickly to get back to other household chores
   - Tiredness of the person

(d) The carrier's age, gender and urgency

Water is most commonly carried by women and therefore by (in Africa) busy people. Babies may be left behind (a reason to increase speed) or may be carried (which reduces speed). Women often carry together, waiting at the source until a friend has filled her container. School children regularly carry water (more often girls than boys) especially at weekends. They are usually in less hurry than adults and more prone to combine water collecting with 'social' activities. There has been some recent discussion of the (moral/AIDS) danger to teenage girls of going alone to fetch water at dawn or dusk, which might lead to parental pressure on them not to loiter en route. In dry periods when distances are greater, men play a larger role in water fetching and probably travel a little faster than women. However, even strong men do not carry two jerrycans over any significant distance – the adult unit of water carriage is largely standardised at 20 litres (= 20kg).

(e) Total time and time cost

The table above indicates a mean time for water collection of 3 hours per household per day. Insofar as the survey was small and the month was dry, this can only be taken as a crude estimate. It is perhaps an over-estimate in that none of the households surveyed were in trading posts or other population concentrations. However the figure is only an inferred walking time and does not include waiting time. In the drier months it is not uncommon to see a queue of 20 jerrycans at a source yielding under 5 litres per minute, implying a waiting time there of 80 minutes. The queue rarely includes adults; women try to avoid such queues by fetching water before dawn, an expedient not without physical dangers (falls in the dark, snakebites etc.).

The opportunity cost of 3 hours per household is, in rural East Africa today, between $\$_{US}0.2$ and $\$_{US}0.6$. The payment to a youth in a trading post to carry four jerrycans (a typical quantity - see Table 3.3) from a source 1.5kms away is currently about $0.5.

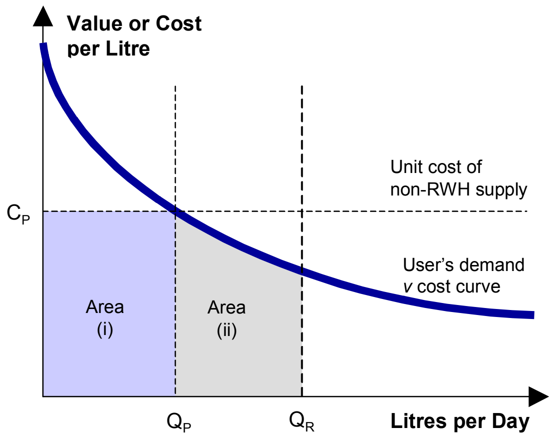
4.4. Combining RWH with other water sources

The following two Sections (4.4 & 4.5) are an in depth study of the economics of combining water sources (multi-sourcing) and the effect of using differing water management strategies on seasonal water security. Those who are looking for a brief overview can skip these sections.

For a given size and location of RWH system and for a given operating strategy, there will be a limit on the water it can supply per day, per week or per year. The maximum per year, corresponding to zero tank overflow, in litres will be the product of roof area (m^2), the annual rainfall (mm) and a run-off capture factor (typically 0.85).

Consider first the situation where we can disregard seasonal factors, and assume that before RWH arrived, daily consumption from a point source was Q_P (litres/day). Q_P is determined by the interaction of the user's demand (cost v volume) curve and the unit cost C_P of supply from the point source. The daily cost to the user was therefore Q_P x C_P.

Figure 4.4: Value of rainwater



If the water Q_R available per day from RWH is *less* than Q_P, then the users will draw Q_R from the RW system and the remainder Q_P-Q_R from the point source. The total consumption will not increase and the effective value of the harvested rainwater will be the saving Q_R x C_P .

If the water Q_R available per day from RWH is *more* than Q_P, then the users will increase their consumption from Q_P to Q_R and the rainwater will be worth more than the former total cost Q_P x C_P. Exactly how much more will depend on the user's demand curve. The situation is represented in the diagram below, where Area (i) is the saving (Q_P x C_P) while Area (ii) is the value of the extra water.

Note that Q_R is the daily amount *available* from RWH, whereas Q_P is determined by the price of supply (from non-RWH sources). The total value Area(i) + Area(ii)) is less than (Q_R x C_P) because the extra water is per litre less valuable to the user than the water 'replaced'.

4.5. Seasonal effects and water management strategies

In the last section we ignored seasonal effects, although one can identify the condition $Q_R<Q_P$ as representing a dry season and $Q_R>Q_P$ as representing a wet one. However seasonality is central to the operation and performance of a RWH system. A user can choose to emphasise dry season security or alternatively to emphasise roofwater capture. To some extent the dry and wet season water needs are in competition with each other. Consider the following four water management strategies for an already built RWH system.

To make the strategies easier to visualise, assume a scenario typical of a homestead in the Great Lakes region where mean daily roofwater runoff is R = 100 litres). Assume that 'dry' weeks (runoff less than 350 litres per week) comprise 1/3 of each year and that the RW storage capacity is 700 litres (7 x R or '1 week'). This storage is only modest, but corresponds to perhaps 50 days drinking water or 14 days total water under very careful management.

Strategy 1 – *High Water Capture* – Water is withdrawn at a high rate, Q = 1.5 R, (e.g. 150 litres/day under our scenario) whenever it is available. This will result in fairly low occurrence of tank overflow, but leave little reserve for dry weeks.

Strategy 2 – *High Security* – Water is withdrawn at a low rate, Q = 0.5 R, (e.g. 50 litre/day) whenever it is available. Much water will overflow the tank, so annual capture will be low.

Strategy 3 – *Adaptive* – Water is withdrawn at a rate Q determined by how much is in the tank, thus:

Q = 1.5 R (e.g. at 150 lpd) if tank > 2/3 full;

Q = R if tank < 2/3 but >1/3 full;

Q = 0.5 R, if tank < 1/3 full.

Strategy 4 – *Maximum Security* – Water is saved for the dry seasons and drawn frugally (e.g. 50 litres/day) *only* after nearby point sources have run dry or after 2 weeks without rain.

The trade-offs involved between these alternatives are summarised in the following table, in which the word 'security' is taken to mean the fraction of days the demand is met by RW (the tank does not run dry). The factor K is the dry-season value of water (valued at its cost from the nearest point source) divided by its wet season value. Thus K=1 represents places where point-source water is unvarying through the year, whereas the extreme value K=10 represents places where in the dry months all local sources dry up, so water must be queued for, then carried from, very far away. A typical value of K in the Target Area might be 2.

Table 4.2 suggests how we might account for seasonal differences in our economic evaluation, namely by assigning different wet and dry season values for water and operating the system to maximise their sum.

Table 4.3 represent the simulation of the four strategies applied to respectively a small DRWH system (storage volume V = 7 x mean daily run-off, R), a medium size system (V/R = 21) and a large system (V/R = 63). Data from Mbarara (daily rainfall for 10 years) has been used and a roof area of 45 m^2 has been selected to give the assumed mean run-off R = 100 litres/day. For Mbarara the dry season (defined by rain in the last fortnight being under 50% of mean fortnightly rainfall) is 36% of the year.

As well as water supplied (column 5), a 'weighted' water supplied column is shown alongside in which effectively K = 5. This yields the weighting (a 'wet season litre' is a cost-equivalent volume): 1.0 dry season litre is deemed to be worth 5.0 'wet season litres'

The **bold** columns in the table contain the performance measures of most interest.

Column 3 shows 'Capture efficiency', (*E*) – a high value indicates that most of the roof run-off is being consumed.

Column 8 shows 'Dry season water security', (S_d) – the fraction of dry season that tank does not run dry and so demand has been satisfied; note however that under Strategy 1 the dry season demand is maintained very high at 1.5 *R*, whereas the other strategies are using demand of only 0.5 *R* for the dry season.

Column 6 shows weighted annual water consumption, Q_5, which is a measure that attempts

Table 4.2 System Performance under Different Operating Strategies

Strategy No	Annual consumption	Relative value of annual water harvested		Wet season security	Dry season security
		if K=1	if K=10		
1	high	high	med	high	v. low
2	low	med	med	high	low
3	medium	low	v. low	high	low
4	very low	med	med	nil	med

to combine quantity, and security measures, by valuing wet season water much more highly than dry season water.

Examination of the top part of the table – which is for a VLC system with V/R only equal to 7 days – indicates that Strategy 1 (in which water is drawn generously whenever available) gives the highest annual water yield E, the lowest level of dry season security S_d, yet a high value for the seasonally-weighted yield W_e.

By contrast Strategy 4 (water is drawn sparingly and only in the dry season) gives the highest dry season security at the cost of the lowest annual yield. The seasonally-weighted yield is however also low. In fact we can dismiss Strategy 4 because even here, where per litre we have valued dry season water at *five times* wet season water, it still gives the lowest output valuation.

Table 4.3: Relating RWH system performance to operating strategy and storage volume

Strategy number / type	V/R	Capture Efficiency	Tank Utilisation	Mean daily consumption Q in litres		'Security' (*S*) = fraction of days demand is satisfied by roofwater		
	tank size days	*E*	*U*	*Q_1 K=1*	*Q_5 K=5*	*S_w* Wet	*S_d* Dry	All year
Small tank, VLC system								
1 High demand High capture	7	**0.70**[1]	36.5	70	**95**	0.75	**0.22**	0.56
2 Low demand High security	7	**0.41**[3]	21.4	41	**80**	na	**na**	na
3 Adaptive	7	**0.66**[2]	34.4	66	**93**	0.94	**0.38**	0.74
4 Max security in dry seas	7	**0.17**[4]	8.9	17	**84**	na	**0.52**	na
Medium size tank								
1	21	**0.91**	15.8	91	**125**	0.90	**0.25**	0.67
2	21	**0.47**	8.2	47	**107**	na	**na**	na
3	21	**0.86**	14.9	86	**138**	1.00	**0.66**	0.88
4	21	**0.26**	4.5	26	**128**	na	**0.73**	na
Large tank								
1	63	**1.00**[1=]	5.8	100	**165**	0.92	**0.38**	0.72
2	63	**0.51**[3]	3.0	51	**123**	na	**na**	na
3	63	**0.99**[1=]	5.7	99	**203**	1.00	**0.98**	0.99
4	63	**0.37**[4]	2.1	37	**182**	na	**1.00**	na

Notes:
1. Data is for Mbarara, Uganda
2. Annual run-off = annual demand
3. na indicates strategy does not allow demand to be met.
4. Highlighted cells indicate best strategy or within 3% of best
5. Strategy 1 gives best **Q_1** (highest water capture)
6. Strategy 3 gives best **Q_5** (highest benefit if K = 5)
7. Strategy 4 gives best **S_d** (highest dry season security)
8. Strategy 3 is always best or second best by all measures.

'Value' is calculated assuming first litre per day is worth 1.5 falling via 0.5 at the 100th litre to zero at the 150th litre

Strategy 1 is to withdraw 1.5 times base demand when available (and otherwise what is available)

Strategy 2 is to withdraw 0.5 times base demand when available (and otherwise what is available)

Strategy 3 is to withdraw 1.5, 1 or 0.5 times base demand, according to amount in tank

Strategy 4 is to withdraw nothing in wet season and in dry season base demand when available (and otherwise what is available).

Strategies 2 and 3 are intermediate in performance, with Strategy 3 (adaptable) generally outperforming Strategy 2 (fixed low-demand).

From this table we can conclude that unless dry season water has exceptional value – e.g. it is per litre worth more than the 5 times wet season water assumed in the table – Strategies 1 (high usage) and 3 (adaptive) are superior to the other strategies.

The bottom band of the table is for a much more expensive system with 9 times larger storage. With such a large tank, the relative superiority of Strategy 3 is increased. We also see the benefit of the larger store. Comparing say Strategy 3 for the very large tank with that for the small one, we find a 50% increase in water harvested (E), a nearly 4-fold increase in dry season security (S_d) and under the assumed value ratio (K=5) a 120% increase in water value. The graph below shows the variation in value of water harvested for varying values of K and for various sizes of tank. It confirms that VLC systems (V/R < 10 days) give a generally acceptable performance unless dry season water is deemed very much more valuable (e.g. K=5) than dry season water. Note the clear 'diminishing returns' with increase in tank size. If water value had been plotted against tank *cost* rather than tank *size*, the same pattern of diminishing returns would appear but with a slightly reduced strength.

A VLC system in the Target Area, attached to a $50m^2$ roof, might be expected to harvest around 25,000 litres of water per year (say 75% of run-off), averaging about 90 litres per day in the wettest 8 months and 30 litres per day in the driest 4 months.

Table 4.4**:** Performance under Strategy 3 – Table showing variation of value ratio, capture efficiency and security with tank size

	dry:wet value per litre	**Normalised tank size – V/R in days**								
		1	**3**	**5**	**7**	**14**	**21**	**30**	**60**	**90**
Benefit ratio = value of water harvested ÷ value water demanded	if K=1	0.29	0.49	0.60	**0.66**	0.79	0.86	0.91	0.98	1.00
	if K=2	0.24	0.40	0.49	**0.53**	0.65	0.72	0.79	0.90	0.94
	if K=5	0.18	0.30	0.35	**0.38**	0.48	0.56	0.65	0.82	0.88
Capture efficiency		0.39	0.66	0.76	**0.82**	0.90	0.93	0.95	0.99	1.00
Security		0.15	0.41	0.57	**0.67**	0.81	0.86	0.90	0.98	1.00

Notes:

1. Under this strategy the demand is varied from 0.5 to 1.5 times the mean daily runoff according to how much water remains in the tank
2. V/R is tank size (normalised to mean daily run-off); K is dry-to-wet season water value ratio; the bold column shows the performance of a typical very-low-cost RWH system

5. THE MANAGEMENT AND SOCIAL IMPACT OF DRWH – QUOTATIONS AND EXAMPLES

During the recent study in Uganda, information was gathered from communities regarding the social aspects of water collection and the impact of the RWH systems. The information is given in anecdotal form in the following examples. The experience with small scale RWH in Thailand is shared in Section 5.6

1.1. Children and water collection

Children collect water in containers of varying sizes. The containers used by the children vary from 3litres to 10litres for children below 8 years of age. Children above 8 - 10 years of age use larger containers and take on a more responsible role.

Where water is close to the house, say within one kilometre, children may be the sole collectors of water. When water sources are more distant, women will help the children. During the dry spells, the men may also help as the nearest water source may be 4kms or 5kms distant. Men tend to use bicycles, carts or donkeys for water collection.

Some children do not find water collection such a burden, as we see in the example below:

Moreen. Birere, a young girl from Rukungiri was asked how she spends her time.

> *"Our parents are very strict, the only time to let us out from home is, when going to school, church or collecting water. It's harder in the holidays. The only chance to meet friends is at the spring (water collection)"*

1.2. Sickness and water collection

The elderly and the sick tend to suffer disproportionately. This is due to the fact that the sick find it difficult to collect water and usually carry smaller quantities of water.

Timanya: An elderly women of (65yrs) in Kabale, Uganda.

> *"Water collection has become more of a problem than before in my life, I suffer from backache, all my grand children stay with their parents in turn, every day I collect 5 litres. At times children from the neighbours help me".*

1.3. Ranking of water sources

A ranking system is used by beneficiaries of RWH to determine the value of the captured rainwater. There are three main categories used for ranking water:

Ranking of sources by quality

Communities consider rainwater to be of high quality, hence other water sources are used where high quality is not an issue e.g. making mud for houses, mixing building mortar.

Ranking source by effort cost

For certain activities requiring large quantities of water, e.g. washing clothes or watering the animals, rainwater is not used. Usually people will take their clothes or animals to a water point for cleaning and watering as the effort involved in carrying the water is reduced greatly this way.

Ranking of sources by seasonal reliability

Communities vary their source of water depending on season. The closest available source is preferred, but sources that are used for cattle watering in the wet season may get priority for human consumption during the dry season.

1.4. Rainwater and water security

Householders consider RWH to be a water source that is supplemented by other sources. This has been an advantage in the dissemination of small rainwater tanks, as the beneficiaries see the tanks as a partial supply and their expectations are not too high.

Member of Rakai women's group:

> *"We know our jars are small, they cannot meet all our water demands, but we use the water sparingly, to prepare tea, drinking only after boiling".*

1.5. Rainwater management

Rainwater is managed in a number of ways. The main management strategies are listed here:

Maximum security

The water from the tank is not utilised not until all the possible water sources are completely depleted. In this case, in the rainy season, after the tank has been filled it is locked up. This has a disadvantage of not maximally using the water from the roofs; the tank is left to over flow.

Maximum capture

The tank is used as a water source throughout the rainy season. It keeps filling as the stored water is being utilised. However, there is a tendency of reducing the daily consumption as the dry season sets in.

1.6. Comparative experiences with small-scale RWH in Thailand

Rainwater harvesting is more widespread in Thailand than any other country in the world. More than 10 million 1000 to 2000 litre rainwater jars and hundreds of thousands of 6 – 12m^3 rainwater tanks were constructed between 1985 and 1992. Most of the households in north-eastern Thailand have at least one, and some have many, rainwater jars. The Thai RWH programme is considered to be one of the most successful examples of how potable water supplies can be increased on a national scale.

The rapid growth and success of the Thai programme was made possible by a combination of factors that may be relevant to other countries interested in developing broad, as well as limited scale, RWH programmes. Government commitment was very strong ad national objectives and targets were clearly defined, and there was popular support at all levels including NGO's, community based initiatives and the private sector. RWH is a long-standing tradition in Thailand and the annual rainfall is high relative to many other regions of the world, with a rainfall pattern favourable to RWH. The demand for improved water supplies in rural areas was tremendous, and this demand led to the emergence and growth of many independent jar making micro-enterprises. Thailand also experienced a period of national economic growth and an increase in private affluence during the life of the programme which made it easier for families to invest in RWH technologies. Funding came from a number of sources, including the well-established Rural Job Creation Project, the Provincial Development Fund, the Provincial Administrative Organisation, as well as the private sector and non-profit organisations.

Originally, the jar construction programme was to be financed by a revolving fund, using start-up money from the government. However, the programme expanded so rapidly that the administration of the funds could not keep up with demand and these funds were generally not used. Many districts provided construction materials, tools and training and people contributed labour to construct their own jars under the guidance of experienced technicians.

It was initially envisaged that villagers would construct their own jars, but as the programme evolved the private sector became very much involved in rain jar construction. Small jar making factories sprang up and developed into successful micro-enterprises in many provinces. The price of a 2m^3 jar in 1992 was US$40 and many of the village based companies were manufacturing up to 30 jars per day. Subsidised and affordable cement added to the favourable conditions.

Some of the early tank designs suffered form major problems. More than 50,000 bamboo reinforced tanks were constructed and these suffered from attacks by fungus, termites and bacteria. An interlocking block tank design was abandoned because the skill levels required were not suited to local conditions. The eventual design that was adopted by the Ministry of Health is a cement mortar jar that has a lid on the top to prevent contamination; a tap for easy access to water; and a drainage plug for easy cleaning. Commercially made jars often did not have these essential features. Numerous moulds have been used, including jute bags filled with rice husks, a 54 piece cement mould, and the star fruit (segmental) steel or cement mould. Larger jars of up to 3 – 5m^3 were also constructed for individual households with iron reinforcement. Thousands of larger tanks have also been constructed at schools, clinics, temples and private homes.

A major rainwater quality study, published in 1989, showed that only 40% of samples met the WHO guidelines for total bacterial count for drinking water. It was convincingly shown that much of the contamination came from secondary causes, such as poor water handling. Despite the problems found with the water quality, the study concluded that rainwater is still the safest and most economical source of drinking water available in most rural areas.

(Source: Raindrop, Rainwater Harvesting Bulletin, July 1992)

6. HEALTH ASPECTS

6.1. Health

Water relates to health in complex ways. It is conventional (Cairncross & Feacham 1993) to identify five types of water-related illness

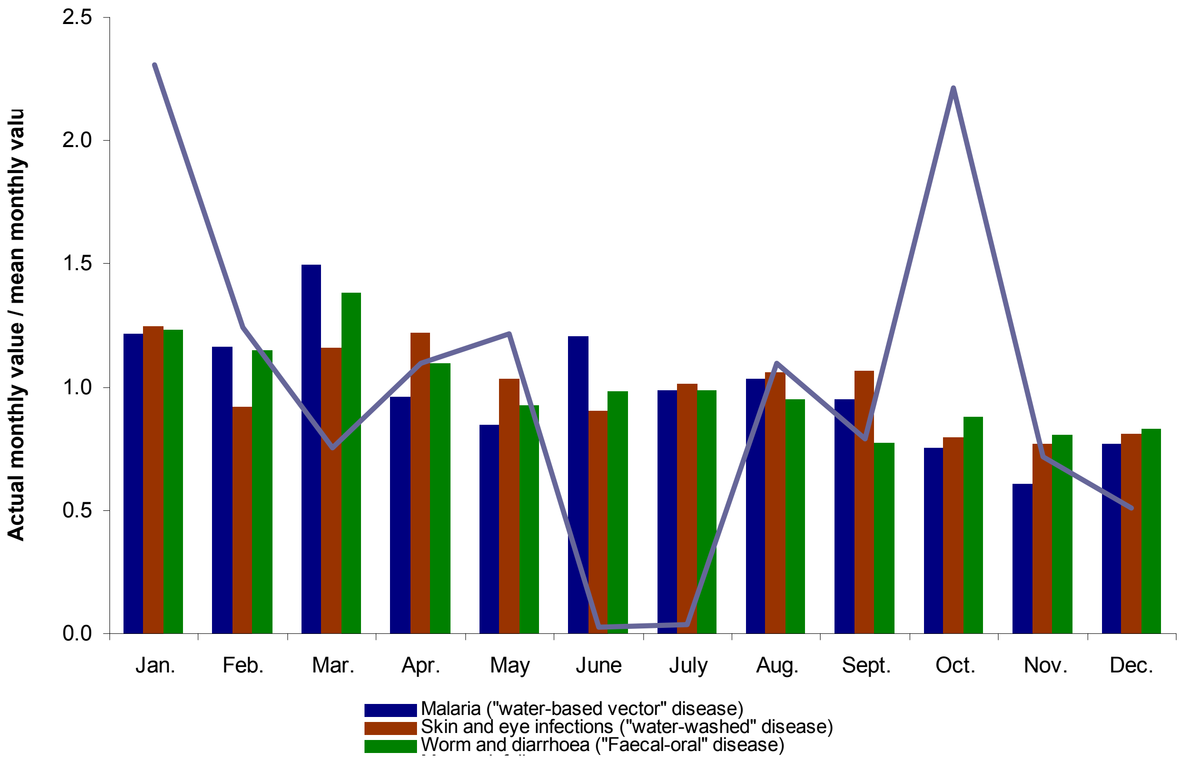
1. Water-born or faecal oral diseases caused by biologically contaminated water
2. Water-scarce or water-washed diseases, mainly skin and eye infections – however as water scarcity lowers hygiene standards, some diseases are both water-borne and water-washed.
3. Water-based diseases involving agents like bilharzia parasites that have an aquatic stage in their life cycle
4. Water-related (insect) vector diseases like all those carried by mosquitoes
5. Poisoning by substances dissolved in water.

Practically the introduction of DRWH might be expected to definitely reduce categories 3 and 5 diseases. Only where it increases water consumption might it reduce category 2 disease.

The impact of DRWH on category 1 disease depends on whether it is a bacterially cleaner or dirtier source than was used hitherto. The debate on RW quality is a complex one (Gould & Nissen Petersen Chap 6). Generally, RW stored in well-covered tanks is as clean as water carried to a house from even a very clean source and is certainly cleaner than water from swamps or streams. Conversely water from a RW tank containing drowned rats is unhealthy.

The impact of DRWH uptake on mosquito breeding has also been much, but not very conclusively, discussed. It is not easy to maintain effective anti-mosquito screening throughout the life of a water tank. So RWH might be expected to increase malaria, particularly during the dry seasons when other mosquito breeding sites are scarce. Conversely

Figure 6.1: Fluctuations of normalised malaria cases, skin+eye cases, worm+diarrhoea cases and rainfall for 1998



present water sources are often valley-bottom wells close to swamps. Queuing at these at dawn or dusk carries a high risk of being bitten by mosquitoes, a risk that DRWH should reduce.

In the absence of DRWH one might expect a fall in malarial cases and a rise in water-washed diseases during the drier months. Medical data was collected from Mbarara hospital and grouped to emphasise any such seasonal variations. Many thousands of cases were summarised. Skin+eye infections were taken to represent water-washed disease, worm+diarrhoea cases to represent water-borne disease and malaria to represent water-related insect vector disease. Figure AA below plots the normalised incidence of each disease group alongside rainfall for each month of 1998. Inspection of the bar chart does not confirm the seasonal relations forecast above. Indeed formal statistical analysis shows insignificant correlation between monthly variations in rainfall and any disease group ($R^2<0.5$). Against this background it is very hard to predict the health implications of any large increase in DRWH use.

Insofar as DRWH replaces water haulage by women and children, there are safety benefits associated with mothers not having to leave young children attended only by older ones (babies burnt in fires) and children not having to venture in lonely places before dawn or after dusk (an oft-expressed concern). A reduction of jar-carrying probably also reduces the incidence of arthritis and back injury. Falls are a danger during water carriage on steep slippery slopes during or soon after rains, so that being able to avoid trips at this time is particularly valued. East Africa does not generally have the sort of society where water-collecting gives women their only legitimate reason for leaving the home; even so a reduction in walking might slightly reduce social interactions.

Table 3.3 indicates a mean daily water consumption of about 10 litres per capita. This figure was confirmed by a small survey in Kabarole District and thought to be representative by officers of local water NGOs. The WHO recommended minimum is 20 lcd but this figure is rarely reached where water has to be hauled for significant distances. A generation ago Bradley and White (1972) explored in *Drawers of Water* the influence of haulage distance and family size on per-capita consumption of water drawn from point sources in East Africa. Their study is reportedly being currently updated under a project named *Drawers of Water II,* but no data could be obtained from that source for this study.

Lastly come the issues of reduced exhaustion, better nutrition and the use of released time, all of which have a difficult-to-quantify health impact.

Qualitative evidence from Rwanda indicates no change in malaria morbidity with DRWH introduction.

Other 1991 census evidence from Uganda, shown here, suggests that Districts within the Target Area having better access to safe water do broadly have a lower infant mortality rate. However the definition of access is a very broad one. Kabale and Rukungiri Districts are very hilly so water carriage there is especially onerous.

6.2. Findings from a recent study into water quality in DRWH systems

A study was carried out recently at the Indian Institute of Technology in Delhi, India, to determine the quality of rainwater from domestic rainwater harvesting systems. The major conclusions reached

Table 6.1: Health data from the Target Area

District	Infant mortality	Access to clean water	Population in 1991	Population density
Units	per live birth	%	1000	Persons/km^2
Rakai	0.199	6	383	99
Kabarole	0.136	6	746	92
Mbarara	0.145	19	931	98
Rukungiri	0.125	27	391	151
Kabale	0.114	58	417	246

Source: 1991 Uganda Census

are as follows:

1. Generally, the physico-chemical quality of water in terms of colour, odour and taste, pH, total dissolved solids (TDS) and total hardness (TH), meet the prescribed standards. Occasionally pH has been reported to be low (acidic) or high (alkaline).

2. Toxic metal ions and toxic chemicals are reported only in rare cases and may arise from material used for the roof or atmospheric pollutants adsorbed on dust.

3. Most of the material used for storage tanks e.g. cement, iron, wood and plastics do not negatively affect the physico-chemical quality, with a few exceptions.

4. The physico-chemical parameters can be tested easily by using available field kits.

5. The main problem with the quality of stored water in DRWH lies with its bacteriological quality. The following are the main issues:

    - Dust from the soil, and droppings of birds and animals can also be the source of contamination by the above bacteria.

    - In any case where first flush eliminating devices are absent, all the indicator bacteria are generally present in water samples in numbers beyond what is acceptable by any standards. Higher temperature reached by a metallic roof due to solar heating may lead to reduction in bacteria.

    - From the health point of view it is important to clean the gutter from time to time and ensure that water does not stagnate. This leads mosquito breeding.

    - Tree hanging in the vicinity, definitely enhances the possibility of contamination due to increased access of the roof to birds and animals.

    - On storage, generally due to limitation of nutrients, bacterial count falls. Different indicator bacteria under study decay over 7-20 days depending on the initial amount of bacteria, nutrient availability and other storage conditions.

    - Increase of temperature due to sun's heat or exposure to UV radiation of sun, reduces and ultimately eliminates bacteria. However, exposure to sunlight in the presence of nutrients can lead to algal growth, especially when the storage is open.

    - Mosquito breeding generally occurs if mosquitoes are already available in the vicinity of storage. Water quality deteriorates with the breeding of mosquito. The only way to prevent mosquito in the tank is by covering the openings by appropriate screens.

Thus the basic conclusion from the study, substantiated by actual experimentation under the project are that DRWH must be designed, taking the following into consideration:

1. Convenient first flush device must be integrated. Roughly the first flush to be may be taken to be 2 mm rainfall and the volume is obtained by multiplying this by the area of the roof.

2. Storage must be tightly lidded and all entry points must be closed by a mesh to prevent entry of mosquitoes and eggs.

3. It is preferable to allow the water to stand for some time before drawing. The bacterial count is more at the bottom. Hence the water may be drawn from a higher level, e.g. withdrawing water from an over flow system may be useful. Thus, instead of one tank of large capacity, more tanks in a series may be used, but increase in total cost has to be considered.

4. Some rapid testing methods like H_2S test methods are useful in the field for indicating presence of biological contamination. The safest methods of treatment are exposure to UV & boiling.

7. TECHNOLOGY – DOMESTIC WATER STORAGE

It is difficult to understand why certain technologies prosper over others. There are many examples of situations where inferior technologies thrive whilst the ideal (in the eyes of the technologist) is shelved or dropped in the dust bin. The reasons are often political or market driven, rather than technology driven and a good salesman can be a wonderful asset. In the case of developing countries, technologies which are well-suited to improving the lives of rural poor are also overlooked on occasions. Again, there are a variety of reasons, the main reasons usually being a poor access to knowledge and information, traditional cultural practises and a lack of political will. Small-scale RWH is one such technology that has been largely overlooked by the majority of poor rural households in LDC's. In countries where the technology has been embraced (Thailand being the most prominent example), great benefits have been seen and large steps taken in alleviating the daily drudgery faced by householders in the task of meeting their water needs.

7.1. Requirements of a domestic water storage tank

Any vessel used for storage of potable water in a domestic context should have certain attributes. These are investigated below in some detail:

Strength.

Any tank that is to store water must have sufficient strength. Water pressure inside the tank creates stresses, which, if not dealt with properly, can cause the tank to fail, which could in turn lead to serious damage of the tank and injury to persons and /or damage to surrounding buildings. Ideally a full engineering analysis should be carried out for any new tank design and tests carried out to confirm the findings. In practise, tanks are usually designed and built, based on previous experience with the material being used and/or previous experience with similar vessels. A good safety factor is usually incorporated in such cases. In Section 2 the shape of tanks was discussed. Existing tanks come in a number of common shapes. The relative merits of these shapes are discussed in Table 7.1

Impermeability.

A water vessel should obviously be impermeable. This is achieved in one of a number of ways, depending on the material from which the tank is made. Some materials are inherently water proof e.g. corrugated steel sheets or fibre glass, and require no (or little) treatment to provide an impermeable barrier. Traditional materials, such as masonry and brick, are usually dealt with by applying an internal render of sand and cement, which can be treated with a water proofing agent or given a final coat of 'nil' (cement slurry). Ferrocement technology uses this concept by applying a cement slurry onto the wall of the tank when complete. Modern plastics may allow low-cost linings to be produced although little has been done in developing countries to

Table 7.1: Relative merits of some common tank shapes

Tank shape or type	Stresses	Material usage and construction
Cuboid	Stresses are unevenly distributed and difficult to calculate	The ratio between material usage and storage capacity is lower than for a cylindrical and doubly curved tank. Construction is quite simple
Cylindrical	Stresses are more evenly distributed and are easier (though trivial) to calculate	There is an improvement in the material use to storage capacity ratio (a saving of 7.5% given a good height to diameter ratio) Construction becomes more difficult with traditional materials e.g. bricks
Thai Jar Style (doubly curved tanks)	Stresses are ideally distributed if the proportions of the jar are correct	Material usage to capacity ratio is very good (savings of up to 20% over a cuboid) but construction can be very difficult, often relying on specialised moulds.

develop a suitably sized off-the-shelf solution. Other modern materials, such as bituminous paints, suitable for use with potable water supplies, are slowly becoming available on the market in LDC's.

Durability

of storage tanks is a critical question. Engineering techniques for determining the durability (through accelerated ageing) are expensive and so the only way to properly 'test' a new technology is usually to apply the test of time. This is problematic when we are looking for a useful life of 20 – 30 years. Little information seems to be available on existing tanks and their useful life spans. The experience in Thailand (documented in Section ??) shows how some unsuitable technologies can be widely disseminated before major flaws appear. In the Thai case more than 50,000 bamboo reinforced mortar jars were manufactured, many of which failed due to termite and fungal attack on the bamboo.

Sufficient storage capacity.

This topic is discussed in far more detail in other sections of this report. Many techniques are available in the RWH literature for determining the ideal size of a tank for full water coverage throughout the year, but none exists for determining the size with modified consumption (during the wet season for example), or for partial coverage.

Maintenance of water quality.

A good storage vessel should maintain and improve the water quality. This is achieved in a number of ways:

- a good fitting, light-proof cover will prevent debris, animals or humans from entering the tank and prevent light from causing algae growth
- water quality can enhanced by putting water into the tank and taking it out of the tank at the correct location – low-level tank entry and floating off-takes are devices designed to aid this approach
- good sanitary conditions around a tank will prevent disease being spread
- water extraction should be such that the water is not contaminated while being drawn
- filters improve water quality are discussed in a following section

No increase in health risk.

Sometimes, with all good intentions, a water tank can become a serious health hazard. This is particularly the case when mosquitoes are allowed to breed in the tank. This can be avoided by sealing the tank well and preventing the mosquitoes entering and breeding by covering any openings with mosquito gauze.

7.2. Tank size – ideal tank size vs. affordability

Tank sizing techniques usually only consider the optimum size for a tank based on the rainfall available, the size of the catchment area, and the demand on the system. Little consideration is usually given to the affordability of the tank. It is assumed that the customer will be looking at capturing all the water from the roof or enough to meet all their demand. But in some cases, people will be happy with *some* water from their roof. In many cases, the customer may not be able to afford a tank suitable for catching the optimum amount of water. In such cases the tank size is determined by the tank cost and so, in this case, we need to maximise capacity for a given (low) cost.

Below, in Table 7.2 we have classified *domestic* tank sizes into three distinct groups – small, medium and large scale.

Affordability is a strong function of tank size and tank design. The smaller the tank the cheaper it will be and the cheaper the construction materials and labour costs, the cheaper the tank will be. For increased affordability we are therefore looking at small-scale, locally produced RWH systems that use local materials. Local manufacture and use of local

Table 7.2: Tank scale classification

Scale of domestic tanks	Description
Small-scale	Any tank or jar up to seven days storage or up to 1000 litres
Medium-scale	A tank up to several weeks storage or between 1000 and 20,000 litres storage
Large-scale	Any tank with several months of storage or above 20,000 litres storage capacity

skills are design issues, and have been given great consideration during the design process described in Sections 7.4 and 7.8. Affordability is a function of a number of socio-economic factors and is decided at the household level.

As an indication of actual costs for a number of different tank types, a cost analysis of commonly available small and medium scale factory made tanks has been made, and compared with locally manufactured tanks. This is shown in Table 7.3 and shows the actual costs while Table 7.4 shows the cost per litre storage.

As expected, economies of scale show the cost per litre dropping as tank size increases. Also, as expected, factory made tanks are generally more expensive than locally manufactured tanks. The general advantage of off-the-shelf, factory-made, plastic tanks is convenience, a good range of sizes and usually a guarantee of quality. The disadvantage is the high cost. The advantage of the GI sheet tanks is again off the shelf availability, but the quality is dubious with the manufacturer claiming a 15 year life and local contacts stating a more realistic figure to be 2 – 3 years. The usual mode of failure is that the base of the tank rots out and the usual method of repair is to surround the base with concrete. The cost is much lower than that of the plastic tanks. They are manufactured primarily on the outskirts of Kampala and some of the major Ugandan towns by micro-

Table 7.3: Cost comparison between 'imported' and locally made tanks in East Africa (all cost figures in £ Sterling)

Tank size (litres)	Plastic Tanks [2]	GI Tanks [3]	PBG Tanks [4]	F/C jars and tanks	Brick jar 5	Plastic tube jar [6]	Tarpaulin tank [7]
100	20						
250	36						
500 – 600	62			28[5]		21	
750	88				33		
1000	115						
1500	158						
2300 – 2500	219	72					
3000	289			79[1]			
4000	379	88					
5000	463	100					
6000	590	132					40
8000	747	147					
10000 – 11000	976	159	155	264[1]			
12000		207					

Notes:

1. Costs take from 'Rainwater Catchment Systems for Domestic Supply' Gould and Peterson (1999). Costs are from 1998 and converted from Kenyan Shillings at a rate of 113.7 (15/8/2000)
2. Costs from price list, Poly Fibre (U) Ltd, P O Box 3626, Kampala, Uganda - cost of filter and tap not included. Factory made, spin moulded, plastic tanks.
3. Costs from price list, Tank and tanks, PO Box 1219, Kampala, Uganda. Cost of filter and tap not included. These tanks are made from curved galvanised iron sheets which are riveted together and soldered to make them waterproof. Estimated useful life 15 years (by manufacturer) or 2 to 3 years (by local contact). These tanks are also available in Kampala or Masaka (2 hrs drive from Mbarara).
4. Partially below ground tank. Design by DTU. Approximately 10 have been built in SW Uganda of between 5,000 to 20,000 litres. Cost is for 10,800 litre tank not including handpump (approx. £10 extra), based on costing exercise carried out June 2000.
5. Cost based on actual construction cost during study, July 2000. Cost includes tap and filter. See Section 7.8 for design detail and full cost breakdown.
6. Cost based on actual construction cost during study, July 2000. Cost includes handpump and filter. See Section 7.8 for design detail and full cost breakdown.
7. For detail see http://www.eng.warwick.ac.uk/DTU/cs/cs20.html. Costs based on actual construction costs, July 2000.
8. All costs (other than Note 1) were converted from Uganda Shilling prices converted at a rate of 2509 Shillings to the pound (15/8/200)

Table 7.4: Cost comparison – pence per litre storage capacity of tanks in East Africa

Tank size (litres)	Plastic Tanks [2]	GI Tanks [3]	PBG Tanks [4]	F/C jars and tanks	Brick jar 5	Plastic tube jar [6]	Tarpaulin tank [7]
100	19.6						
250	14.5						
500 – 600	12.5			5.6		3.4	
750	11.8				4.4		
1000	11.5						
1500	10.5						
2300 – 2500	9.5	2.9					
3000	9.6			2.6			
4000	9.5	2.2					
5000	9.3	2.0					
6000	9.8	2.2					0.7
8000	9.3	1.8					
10000 – 11000	9.8	1.6	1.4	2.4			
12000		1.7					

entrepreneurs, who sell small numbers of tanks. They also make gutters and downpipes from flat GI sheet. These tanks are found throughout Uganda, but not in very great numbers.

The figures given for the locally made tanks and jars are taken from the work carried out during the study (and documented in Section 7.5), as well as from the RWH literature for the region. It can be noted that the costs are generally lower than for the plastic tanks but in line with the GI tank costs. The expected useful life for the majority of the locally-made tanks is much higher than that of the GI tank. It is also noted that only one size is quoted for each of the small jars – this is because the costing exercise was only done for the work carried out under the study. Similar economies of scale would be expected for larger jar sizes using similar materials, but the design would need to be reconsidered. The aim of the small jars is to provide systems for poor rural households who don't have sufficient money to purchase the larger tanks.

The tarpaulin tank, developed by the Rwandan refugees in Uganda uses a 5m x 4m polypropylene tarpaulin, which is fitted inside a lined pit with walls of poles and mud built up to about 1m around the pit. The outhouse-like building is roofed with corrugated iron sheet (see Figure 7.1). The simple design and use of predominantly local materials make this tank extremely cheap for the given, maximum 6000 litre, storage capacity. The cost per litre storage is only 7% that of the plastic tank of the equivalent size. Tarpaulins and corrugated iron sheet are available locally

Figure 7.1: A Figure 7.1 – The tarpaulin tank. Note the inlet into the side of the tank and the door at the front for scooping water

7.3. Choice of tank type

The type of tank that may be chosen will be dependent upon a number of factors:

- space availability will determine the maximum dimensions and whether the tank will be above or below ground
- soil conditions determine whether a tank can be built below ground – rock causing excavation difficulties and sand being liable to subsidence during excavation

- the choice between factory made or locally made tanks is usually a function of wealth
- for low cost tanks (as defined above) the material and construction technique is usually dominated by what is available locally and what is affordable.
- subsidies, often give as part of tank building programmes, can influence the type of tank that will be bought or built
- there are many other factors that influence the choice of tank

7.4. Materials for tank construction

The fundamentals of design for sustainability suggest that where possible, local skills and materials are used for manufacture. This should be carefully considered when designing RWH systems, particularly in rural areas of developing countries. A

Table 7.5: Advantages and disadvantages of a variety of tank types

Tank type	Advantages	Disadvantages	Comments
Plastic tanks	Off the shelf convenience Quality assured Wide range of sizes	High cost Central manufacture High tooling costs Local skills ignored Transport costs extra	Factory made in large numbers
GI tanks	Off the shelf convenience Reasonable initial cost Moderate range of sizes Low tooling costs for the manufacturer Open to local manufacture	Doubtful quality Transport costs extra	Made by micro-entrepreneurs in the major towns
PBG tank	Reasonable cost Good range of sizes available Low tooling costs Suitable for local manufacture Local skills enhanced Use of many local materials Transport costs embodied in material cost	Quality only assured through good workmanship Water extraction device required to prevent contamination of water	DTU design. To date approximately 30 or 40 tanks have been built in SW Uganda by local artisans.
Locally manufactured small jars	Use of many local materials Use of local skills Low tooling costs – suitable to local artisans Transport costs embodied in material cost Suitable for poor rural households Suitable for incremental adoption	Limited range of sizes for given design Quality only assured through good workmanship	DTU designs dealt with in Section 7.8 These are new designs that have been prototyped and are currently under survey.
Tarpaulin tank	Very low cost Uses skills available to most rural farmers Uses only local resources (except tarpaulin and GI sheet) Very few tools required Significant storage capacity for small farms for irrigation or livestock Suitable for poor rural households Quality assured if new tarpaulin is used	Maximum size dependant on tarpaulin size Some problems at present with termites eating poles and tarpaulin Water extraction device required to prevent contamination of water	Tank developed by refugees in East Africa using UNHCR tarpaulin and now built in some number by ACORD and IVA / UNIFA in SW Uganda.

careful study of locally available skills and materials should be carried out before the design process begins. This can vary from dramatically from place to place, depending on natural resources, the range imported goods and tools and local building techniques (which are usually closely linked to availability of natural resources). Local knowledge is invaluable during such a survey. For the work described in Section 7.5, such a study was conducted and the findings are listed below in Table 7.6 and 7.7

Table 7.6: Resources available close to the site at Mbarara town

Local resources		
Item	**Availability**	**Comments**
Sand	Good quality sand is difficult to find in the area. The sand used was transported 30kms from the Oruchinga Valley. Sand of poor quality is available within one km.	Transport is needed and this costs up to *six* times the sand cost for the 30 km trip. Loading and offloading costs need to be considered – can be as much as the cost of the sand. Bulk purchase (4 tonne loads) is cheaper than buying small loads.
Aggregate	Available locally – about 5 kms from site	Again transport is needed and is costly. Loading and offloading to be considered. The stone is quarried and broken locally by hand.
Stone	Available locally at site.	Stones suitable for foundations and masonry work were available from previous work at the site.
Bricks	Good quality bricks manufactured about 40kms from the site. Poor quality bricks are manufactured locally.	There are the same concerns with transport and loading. The bricks are of reasonable quality but dimensionally irregular. Special (angle ended) bricks were needed and this had to be arranged in advance – a mould was supplied and the special bricks were made and burnt in the next available batch.
Wood / timber	Poles for building and for making ladders and scaffolding are available locally	We could harvest these from the site, as they were growing on the land. Sawn timber is available in town or sometimes locally if trees are being felled and sawn by local farmers.
Materials available in the local market place (a selection)		
Item	**Unit**	**Comment**
Cement	Bag 50kg	Used for most of our construction work
Chicken wire 1/2"	Roll (30 x 0.9m)	Used for ferrocement work
4mm mesh	Roll (30 x 0.9m)	Useful for sieves and for ferrocement work
Rebar 8mm	13m length	Reinforcing and cover manufacture
Rebar 6mm	13m length	Reinforcing and cover manufacture
Welded mesh	2 x 1 m sheet	For concrete reinforcement
Binding wire	kg	For tying rebar and other uses
Barbed wire (double strand)	Roll (600m)	Used for our rammed earth work
GI and plastic pipes and fittings	Wide variety of sizes and components available	Water extraction
Sisal rope	Roll	General purpose
Nails	kg	General purpose
Water proof cement	kg	For tank linings
Fencing staples	kg	General purpose
Plastic sheet (250 micron)	87cm wide roll – bought by the metre	For plastic tube tank – quality in local market is dubious as ends get easily scuffed
Tarpaulins	5m x 4m	For tarpaulin tank – available in some hardware shops or possibly from lcoal agencies dealing with refugees
Timber	Any size to requirements	General purpose (not accurately cut)

Table 7.7: Skills available close to the site at Mbarara town

Skill	Comments
Local pole and mud construction	Known to, and practised by, most rural farmers
Brick laying and rendering	Widely used and known to most masons
Stabilised earth technology	Not known locally
Stone masonry	Known to some masons but not widely practised
Ferrocement tank construction	There had been some previous training in the area, so a number of masons had been exposed to the technology. One local mason was very experienced and did good quality work.
Carpentry	Several carpentry workshops in town with a limited range of power tools available (thicknesser, planer, power saw, pillar drill, etc). Most carpentry shops specialise in furniture making. The quality of the work varied enormously. Local village carpenters have no power tools and have limited skills. Accuracy of work is generally low. Lathe work can be done but not very accurately.
Metal work	Welding equipment is available in town, but quality of work is not high at most workshops. No turning or toolmaking equipment available. Angle iron and flat bar available locally but few other profiled sections.
Other	A wide range of services are available in Kampala, 4 hrs drive from Mbarara

7.5. Tank trials at Kyera Farm, Mbarara, as part of this Study

A technical study was undertaken as part of the Feasibility Study to allow the study team to build and assess a number of small-scale RWH systems suitable for local manufacture in the region. The study was carried out at Kyera Farm, a training centre in organic farming techniques and rainwater harvesting techniques, based 8kms south of Mbarara, in SW Uganda. During the study 3 types of small storage vessels were investigated, namely:

- a cylindrical brick jar of 750 litres
- a ferrocement jar of 500 litres
- a partially below ground plastic-lined tank of 600 litres

(Technical drawings of each of the designs is given in the Appendix II. Sizes given are approximate)

Figure 7.2 The plastic tube tank

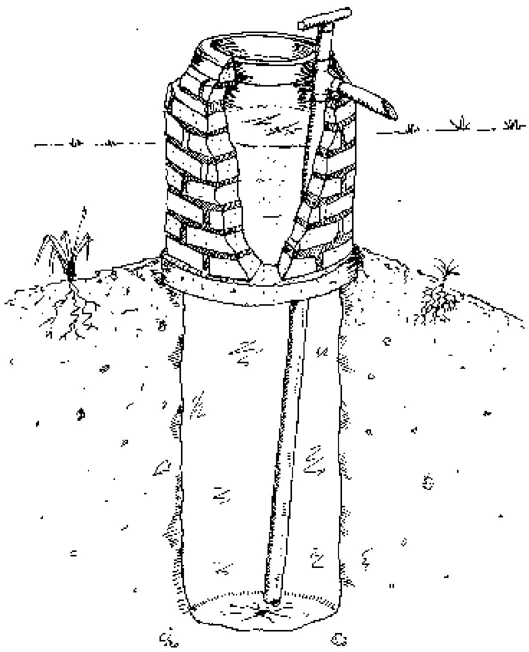


Figure 7.3: The brick jar

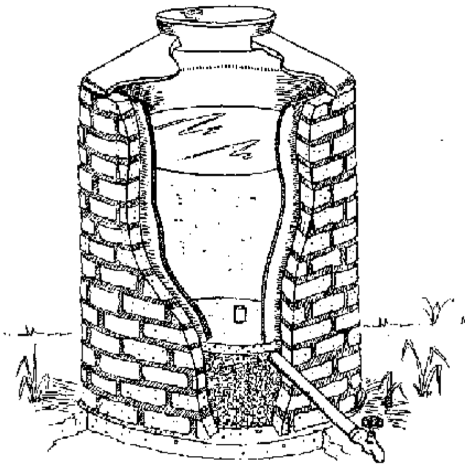
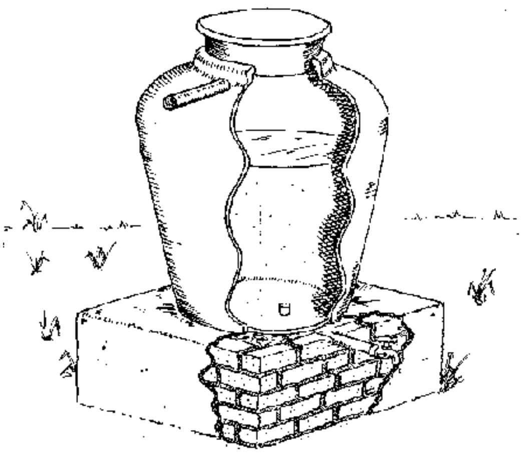


Figure 7.4 The ferrocement jar



The aim of this study was:

- to test three designs of small storage vessel (one well established and two new designs)
- to build prototype / demonstration RWH systems at Kyera Farm to assess the skills and materials required for each of the designs, and their suitability for local manufacture
- to make improvements to the design based on early experiences with the prototypes
- to investigate the use of RWH on grass roofs
- to build a number of systems in the local community to allow a survey to be conducted – the survey will look at the technical suitability of the systems, as well as the use of the jars by householders and the benefits and savings brought about by the RWH system
- to carry out a full costing for each of the RWH systems

7.6. The designs

The design of the jars was undertaken using the principles set out earlier in this chapter.

In the case of the ferrocement jar, the design was taken from the RWH literature (Watt, 1978) and adapted slightly to suit local conditions. The size of the jar was increased from 250 litres, as suggested by Watt, to 500 litres. A tap was incorporated, and the jar set on a plinth, to allow water to be extracted without contamination. Chicken wire was added to the cement jar described by Watt, to give added strength and a combined cover and filter was incorporated to help improve and maintain water quality.

The cylindrical brick tank was developed as it was seen to be a tank, which very closely matches local skills, materials and known building techniques. Brick manufacture is common in the area and brick building techniques well known. The jar is cylindrical, which, as described earlier in this section, reduces stresses and gives a good material:capacity ratio.

The plastic lined tank was developed as a new innovation, specifically aimed at reducing costs. It is an adaptation of a larger partially below ground tank developed by the DTU in Uganda. The tank was designed in such a way that plastic tubular sheet, available in the local market, could be used to line a hole dug to a suitable diameter. The above ground section of the tank is made of brick. The handpump used with this tank was designed during the project and generated considerable interest, enough to warrant a short training course for local NGO technical staff.

It was decided that three designs should be developed, in order that a choice would be available to local artisans and to their 'customers'.

Further information and design drawings are given in Appendix II

7.7. Small tank costs

A detailed costing of the small RWH storage vessels was undertaken and a breakdown of the costs are given in Appendix III. A brief summary of the costs is given in Table 7.8 to allow for easy comparison

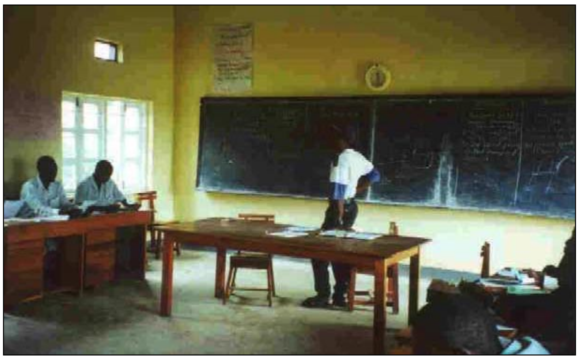
It is worth making a few general comments on the data presented in Table 7.8 and in the tables in Appendix III.

1. Cement is a major expense. A bag of cement costs 3 times the daily wage of a mason. For the jars constructed the cost of cement is dominant – 42% of the material cost for the f/c jar, 45% for the brick jar and 24% for the plastic tube jar. Reducing cement content can significantly reduce cost.
2. Irregular brick size increase cement content as extra mortar is used to fill the gaps. It is worth carrying out a quality control exercise at the brick manufacturing plant.
3. Water extraction can be made cheaper in most cases, but then there is the increased risk of contamination.
4. Further cost reduction exercises should be carried out e.g. reducing f/c wall thickness through proper experimentation, possibly omitting chicken wire from f/c tank, using more locally available materials such as wood poles and mud. It is worth bearing in mind that the jars constructed at Kyera were demonstration/ prototype jars and were constructed to a high standard.

7.8. Training

As part of the study, training was given to eight masons, 4 taken from the local community and 4 taken from a pool of masons who work closely with a local farmers organisation (IVA, Mbarara) who are already building RWH systems. The training was for a period of 6 weeks and was primarily 'on-the-job' training, with instruction being given by the project technician and with a classroom component included at the end of the period to re-cap on the work undertaken during the training. Feedback from the masons on the practical implications of the designs was absorbed and often changes implemented directly as a result of suggestions.

Figure 7.5: Classroom sessions for the masons gave opportunity to reinforce the techniques being taught as well as allowing the masons to discuss concerns and make suggestions



A series of Photo Manuals for the construction of these small RWH systems have been developed based on the work carried out at Kyera Farm. They can be found on the DTU Web Site at http://www.eng.warwick.ac.uk/DTU/rainwaterharvesting/workingpapers.htm or obtained directly in hard copy from the DTU.

A pump training course was arranged as a result of high levels of interest shown by people attending the programme seminar. This course in Low-cost Handpump Manufacture was run over a two-day period on the 22nd and 23rd August 2000 by Vince Whitehead, a Warwick mature student (and experienced machinist) who is working at Kyera Farm voluntarily during his summer break.

Table 7.8 Cost comparison between the jars constructed at Kyera

Type of jar	Size (litres)	Cost (USh)	Cost per litre storage capacity (USh)
Brick jar	750	83,000	110
Ferro-cement jar	500	70,000	140
Plastic tube jar	600	51,500	86

8. TECHNOLOGY – OTHER DRWH SYSTEM COMPONENTS

8.1. Roofs

For domestic rainwater harvesting the most common surface for collection of water is the roof of the dwelling. Many other surfaces can be, and are, used: courtyards, threshing areas, paved walking areas, plastic sheeting, trees, etc. In some cases, as in Gibraltar and Zimbabwe for example, large rock surfaces are used to collect water which is then stored in large tanks at the base of the rock slopes.

Most dwellings, however, have a roof. The style, construction and material of the roof affect its suitability as a collection surface for water. Typical materials for roofing include corrugated, galvanised, iron sheet (GI Sheet), asbestos sheet; tiles (a wide variety is found), slate, and thatch (from a variety of organic materials). Most are suitable for collection of roofwater, but only certain types of grasses e.g. coconut and anahaw palm (Gould And Nissen Peterson, 1999), thatched tightly, provide a surface adequate for high quality water collection. The rapid move towards the use of GI sheets in many developing countries favours the promotion of RWH (despite the other negative attributes of this material).

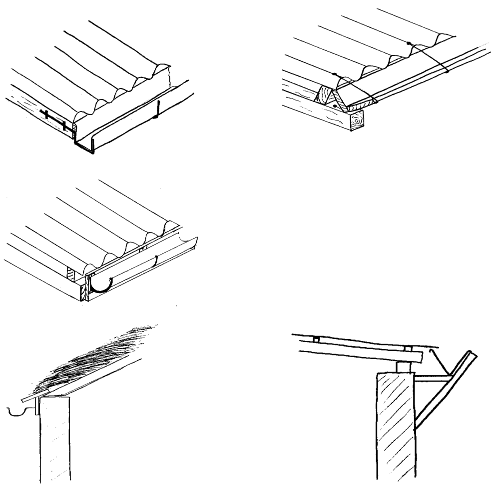
Some work was carried out during the study to investigate the possibilities of using grass roofing for DRWH. Guttering was installed on one grass roof that had been constructed with a plastic membrane beneath it – this helps to prevent UV degradation of the plastic. The grass was loosely thatched and found locally. The plastic sheet guttering that was installed is shown in Figure 8.1. It is designed to capture all the water falling on the thatch and passing through to the plastic sheet. It is fixed using two long poles, one suspended below the eaves and one on top of the thatch. It can also be designed to be demountable from the upper surface, such that it can be 'put away' under the eaves when there is no rain. Again this helps prevent degradation due to sunlight. A follow up survey will look at the longevity of the plastic guttering and the taste, odour, colour and palatability of the water captured. The survey will run until February 2001.

Figure 8.1: Plastic sheet guttering

8.2. Gutters and downpipes

Guttering is used to transport rainwater from the roof to the storage vessel. Guttering comes in a wide variety of shapes and forms, ranging from the factory made PVC type to home made guttering using bamboo or folded metal sheet. In fact, the lack of standards in guttering shape and size makes it difficult for designers to develop standard solutions to, say, filtration and first flush devices. Guttering is usually fixed to the building just below the roof and catches the water as it falls from the roof. Some common gutter shapes and fixing methods are shown in Figure 8.2.

Figure 8.2: Gutters and fixings



8.3. Water filtration

Again, there are a wide variety of systems available for treating water before, during and after storage. The level of sophistication also varies, from extremely high-tech to very rudimentary. The simple trash rack has been used but this type of filter has a number of associated problems: firstly it only removes large debris; and secondly the rack can become clogged easily and requires regular cleaning.

The sand-charcoal-stone filter is often used for filtering rainwater entering a tank. This type of filter is only suitable, however, where the inflow is low to moderate, and will soon overflow if the inflow exceeds the rate at which the water can percolate through the sand. Settling tanks and partitions can be used to remove silt and other suspended solids from the water. These are usually effective where used, but add significant additional cost if elaborate techniques are used. Many systems found in the field rely simply on a piece of cloth or fine mosquito mesh to act as the filter (and to prevent mosquitoes entering the tank).

Post storage filtration include such systems as the upflow sand filter or the twin compartment candle filters commonly found in LDC's. Many other systems exist and can be found in the appropriate water literature.

8.4. First/ foul flush systems

Debris, dirt, dust and droppings will collect on the roof of a building or other collection area. When the first rains arrive, this unwanted matter will be washed into the tank. This will cause contamination of the water and the quality will be reduced. Many RWH systems therefore incorporate a system for diverting this 'first flush' water so that it does not enter the tank.

There are a number of simple systems that are commonly used and also a number of other, slightly more complex, arrangements. The simpler ideas are based on a manually operated arrangement whereby the inlet pipe is moved away from the tank inlet and then replaced again once the initial first flush has been diverted. This method has obvious drawbacks in that there has to be a person present who will remember to move the pipe.

Other systems use tipping gutters to achieve the same purpose. The most common system uses a bucket that accepts the first flush and the weight of this water off-balances a tipping gutter which then diverts the water back into the tank. The bucket then empties slowly through a small-bore pipe and automatically resets. The quantity of water that is flushed is dependent on the force required to lift the guttering. This can be adjusted to suit the needs of the user.

Another system that is used relies on a floating ball that forms a seal once sufficient water has been diverted. The seal is usually made as the ball rises into the apex of an inverted cone. The ball seals the top of the 'waste' water chamber and the diverted water is slowly released, as with the bucket system above, through a small bore pipe.

Although the more sophisticated methods provide a much more elegant means of rejecting the first flush water, practitioners often recommend that very simple, easily maintained systems be used, as these are more likely to be repaired if failure occurs.

8.5. Water extraction devices (handpumps for sub surface tanks)

There are a number of designs of handpumps currently being investigated at Warwick and in Uganda for the extraction of water from below ground tanks. The findings are published as a DTU Technical Release (TR-RWH09 The Manufacture of Direct Action Handpumps for use with Domestic Rainwater Harvest Tanks)

8.6. Treatment of rainwater for potable supply

A number of post storage treatment techniques are recommended. Boiling water is the most commonly recommended, but it is seldom practised. Ceramic candle filters are easily purchased in most major towns in Africa, but their cost is often prohibitive. A technique that is being widely recommended currently is solar disinfection or SODIS. This technique requires only clear glass or plastic bottles that are filled and then placed in the sun for one day. More detail is given in Appendix VII. It is also worth remembering that in many cases contamination takes place during secondary storage or during water handling, due to unsanitary conditions.

9. MECHANISMS OF DISSEMINATION

9.1. Candidate mechanisms for dissemination

Any programme of disseminating a new developmental technique should be built upon evidence that the technique

- has a reasonable chance of meeting important and basic needs in a sustainable way, *and*
- requires an input of 'outside' effort to overcome specific obstacles to its adoption, *or*
- deserves to have its adoption accelerated.

Many useful innovations in the Target Area in recent years have occurred without any non-commercial promotion – the widespread uptake of burnt bricks and iron roofing sheets for housing, bicycles for transport, and plastic jerrycans for water are good examples.

Domestic roofwater harvesting is increasingly practised in the Target area by a minority of better-off householders, and during the last 5 years metal gutters have appeared for sale in many hardware shops. However it has been perceived as too expensive for poor rural households. Recent research has however demonstrated that, properly installed, DRWH *can* be economically viable for all but the very poorest households – in part because of the increasing commonness of hard roofing mentioned above. The payback time of a VLC system is around 1 year. Because RWH is relatively 'capital-intensive', it may need linking to micro-credit programmes or other means of spreading its cost over between 6 and 24 months.

Large (institutional) RWH systems require considerable care in construction and have been the subject of governmental or NGO training programmes for some years [FAKT & KRA, 1997]. Smaller domestic RWH systems have also been promoted, on the basis of their potential to reduce water-collection drudgery, their prospect of providing a focus for women's groups and the absence of familiarity and relevant skills.

Any promotional programme, once it has established that there are benefits in a technology being taken up more widely, can select its activities from the following list:

1. provide a continuing subsidy to make the technology affordable to the target group;
2. provide a temporary subsidy to accelerate take up and cover learning risks;
3. supply micro-credit to facilitate adoption of viable but capital-intensive techniques;
4. improve, adapt and test the technology;
5. train suppliers (especially where these are local artisans);
6. organise the provision of key tools or materials not yet available locally;
7. inform potential users (and producers) of the existence of the technology;
8. provide an independent source of quality control;
9. influence governmental or aid policies to facilitate take-up.

Moreover the promotion may be a primary objective or a secondary one – many technologies are disseminated as part of developmental programmes whose primary purpose is to generate employment, redress gender discrimination, empower citizens, improve health, protect the environment and so on. This can lead to considerable conflict in priorities.

Finally the promotion may be intensive or extensive. *Intensive* programmes have a high level of interaction with a small group, usually defined by socio-economic and geographical criteria, and a low 'multiplication factor'. *Extensive* programmes aim to reach a large group by means that have a large multiplication factor. They are necessarily more narrowly focussed and may employ such techniques as politics and advertising to benefit or reach its large and widely-located target group.

9.2. Regional experience of promoting DRWH

There is a surprisingly large number of organisations active in promoting *domestic* RWH in the Target Area, some of whom have been active for over a decade. Moreover other sources indicate a steady growth in 'commercial' DRWH – e.g. the attaching of tanks to middle-class houses by their owners. The formation of the Uganda Rain Water Association in 1998 and the recognition of the technology by Government are further pointers to its growing popularity. It would seem that the proportion of households operating DRWH systems is rising steadily, perhaps by as much as 1% per annum.

However the Study Area as shown on the map in the Introduction of this report, namely S Uganda, NW Tanzania and E Rwanda has a total population of over 6 million and therefore contains about 1 million homes. Against this figure, the perhaps 2000 DRWH systems installed by the combined efforts of the organisations in table 15 above represents but a tiny impact – say 0.2% penetration. Even if the seeds planted by training were to multiply vigorously, it seems unlikely that the number of DRWH systems disseminated by NGOs etc will catch up those in the private sector. In those same households the (purely commercial) penetration of ($100) iron roofing probably exceeds 60% and of ($50) bicycles perhaps 25%.

It is clear from the reports from active RW organisations in Appendix VI, and from the discussions at the Mbarara Seminar which concluded the Study, that non-commercial propagation of DRWH is extremely constrained. Not only is RWH generally treated as part of intensive rather than extensive interventions, but it also reported to be highly and permanently dependent upon availability of external subsidy. Repeatedly it was reported that it is too expensive for most of the population to purchase. The fraction of system hardware costs (i.e. *excluding* training and promotional costs) provided by beneficiaries in the interventions reported varied from about 10% to 70%. Thus there appears to be no sense that these interventions are for 'kick starting' a technology that is ready to run on its own.

One conclusion could be that DRWH, even in its very-low-cost (say $40) form is not affordably by the great bulk of its potential recipients. It is only a technique in a long-term programme of government-funded or Agency-funded provision of better water supplies for those too poor to do other than collect from 'free' natural sources. It may often cost less per household than such alternatives as protecting more springs and wells, or extending gravity-flow schemes, but it is not sustainably affordable. Thus either (a) DRWH should continue to be extended under substantial subsidy – perhaps over 50% of the organisational and material costs of each installation – at the rate that the availability of such subsidy affords or (b) it should be deemed to have failed to affordably meet water needs and aid should be devoted to other more deserving ends.

An alternative view – hardly a conclusion – is that DRWH should be just left to commercially trickle down from higher-income to middle-income households in locations where other water sources are inadequate.

A further option is to strive to reduce both the cost of individual systems and the cost of promoting/supporting them, so hat far more poor households can be reached with a given financial resource. Moreover the threshold of affordability could also thereby be changed so that all but the poorest say 30% of households could access water in this way without outside subsidy.

This Feasibility Study indicated that VLC systems are often financially justified where the 'opportunity cost' of householder time is over say $1 per day.

Table 9.1: Organisations in the region working in RWH

Name of Organisation	Area of Operation Country / *District* county / town U=Uganda T=Tanzania R=Rwanda	Type of org'n Commercial Gov't Internat'n NGO Local NGO	RWH type Domestic Institution'l VLC	Tech-niques Credit Demonst'n Insallation R & D Subsidy Training	Start with RWH (approx. year)
ACORD Uganda	U/*Mbarara,* Oruchinga Valley	I	D,I,V	C,T,S,R,D	1995
ARUCED	U / *Kabarole* Mwenge / Kyenjojo	L	D,V	T,R,I,C	1997
BRATIS	T / *Biharumulo*	L	D,I	T,D,S	1996
JUDEA	T/ *Bukoba*	L	D,V	T	2000
KARADEA	T / *Karagwe*	L	D,V	T,S,I	1987
Lutheran World Federation	R / *Kibungo* & *Gitarama*	I	D,V	C,S	1996
N Kigezi Diocese WATSAN Progr'm	U / *Rukungiri*	L	D,V	I,T,S,D	1991 ?
Rakai Dep't Water Dev't (Gov't Uganda)	U / *Rakai*	G	D,V	T,S,D	1995 ?
Uganda Nat'n Farmers Association UNIFA / IVA	U/ *Mbarara*	L / I	D,V	T,S	1998 ?
Uganda Rain Water Association	U	L	D,I	D	1998
Uganda Rural Dev't & Training Org (URDT)	U / *Kibaale*/ Kagadi	L	D,V	C,T,R,D,I	1996
Kigezi Diocese Water Project	U / *Kigezi*	L	D,V	I,T,S	1994 ?

10. CONCLUSIONS: PROSPECTS FOR EXTENSIVE TAKE-UP OF DRWH IN THE GREAT LAKES AREA

This Feasibility Study was undertaken to assess whether 'very-low-cost' domestic roofwater harvesting has the potential for extensive take up in the Study Area. It found that 'informal' DRWH, using bowls or sometimes 200 litre drums, is already widely practised and that rainwater collection goods like gutters and GI tanks are on sale in many trading centres. Moreover a large (>65%) and growing fraction of households have at least one roof of GI sheets suitable for RWH.

A number of NGOs have been active in promoting DRWH amongst the rural poor in the Area.

However it seems unlikely that their efforts have yet affected more than about 0.2% of the region's households, far fewer than the number practising DRWH because they can afford its full cost. These NGOs generally hold that for poorer households DRWH can only be accessible if it is strongly subsidised. The activities of NGOs and local government have made the DRWH option far better known and acceptable than a few years ago. Moreover DRWH has been used in a few cases to carry interventions aimed at enabling women to achieve greater status, skills and confidence.

Modelling using rainfall data from various locations in the area indicated that a water store of capacity about 600 litres would enable most households to draw 65-75% of their annual water from such stores, but would still need to collect the remainder from point sources like wells. This relief from water-carrying would be almost total in the wet seasons (about 2/3 of each year) but rather slight in the driest months when carrying distances and queuing times are at their longest. The average annual saving in distance walked corresponds (in this hilly terrain) to a mean time saving of about 700 hours per household per year, worth perhaps $70 at a realistic 'opportunity cost' for time. For households having to pay cash for their water carriage – at typically $0.1 per 20 litres – the benefits would be somewhat greater but at the expense of the income of professional carriers.

At present water consumption per capita is about 13 litres per day. With DRWH this is likely to rise significantly, but only in the wet seasons.

Six hundred litres of storage, including a domestic handpump in the case of underground storage, was shown to be achievable at a cost of around $50 to which might be added $10 for guttering and a downpipe. Three designs of small tank and four designs of simple handpump were developed, tested, demonstrated and used for training. However the cheapest of the stores (a partly underground polythene-lined store) has yet to satisfactorily pass endurance trials.

From this data, and from the various assumptions made, it appears that counting reduced walking time as the sole benefit, the 'payback time' for investment in VLC DRWH is about 1 year. This is an acceptable but not outstanding figure, indicating broad economic viability provided that promotion costs do not exceed say $10 per household. The availability of suitable credit would make take-up much easier for the poorest households. For households that are particularly poorly located with respect to point sources the payback period may be as low as 6 months. The benefits accrue more to women and school age children than to men.

No conclusive data was collected in this Study concerning the health or safety impacts of greater DRWH usage, although studies elsewhere indicate that they should broadly be more positive than negative.

Overall it was found that DRWH could bring considerable benefits to the majority of households in the Study Area, is likely to continue to expand into higher-income households and has considerable potential for adoption by low-income households if artisan training and either credit or modest subsidy can be provided.

BIBLIOGRAPHY

Ariyabandu R. de D., 1999, "Water Security through Rainwater Harvesting", Integrated Development for Water Supply and Sanitation (Proc 25th WEDC Conf, Addis Ababa), WEDC, Loughborough University, pp 366 – 368.

Bradford B., July 1992, "Focus on Thailand" Raindrop, Rainwater Harvesting Bulletin, Volume 7, pp 1 – 11.

Cairncross S & Feacham R, 1993, Environmental health engineering in the tropics, Wiley

Development Technology Unit, University of Warwick, http://www.eng.warwick.ac.uk/DTU/rainwaterharvesting

FAKT & Kenya Rainwater Association, 1997, Mvua ni maji – Rainwater harvesting by women's groups in Kenya, Video, FAKT, Stuttgart

FAO, 1987, Africa: Mean Annual Rainfall Map, (Scale 1:1,000,000), Rome Pearce EA & Smith CG, 1998, The Hutchinson World Weather Guide, 4th Ed, Helicon

Gould J & Nissen-Petersen E, 1999, Rainwater catchment systems for domestic supply, IT Pubs

International Rainwater Catchment Systems Association (IRCSA), http://www.ircsa.org

Rees D.G and Thomas T.H., 1999, "Low-cost Roofwater Harvesting in the Humid Tropics" Integrated Development for Water Supply and Sanitation (Proc 25th WEDC Conf, Addis Ababa), WEDC, Loughborough University, pp 418 - 421.

Rugasira, J., Rakai Women Groups involved in Rainwater Harvesting Activities (Women leading the development process in the new millennium), Uganda Rainwater Harvesting Association and SIDA.

WHO/UNICEF, 1996, Water Supply and Sanitation Sector Monitoring Report 1996 (Sector status as of 31.12.1994) Publication WHO/EOS/96.15; see also http://www.africanwater.org/csscoverage.htm - Africaand for forthcoming Global Assessment 2000 see http://www.unicef.org/prog/wes/

APPENDICES

APPENDIX I
Participants at the RWH Seminar, 19th – 21st July 2000

APPENDIX II
Design drawings (VLC RWH systems)

APPENDIX III
Small tank costs

APPENDIX IV
1. Map of the Target Area showing average annual rainfall

2. Map of the Target Area showing the location of the organisations participating in the Study

APPENDIX V
Minutes of seminar held 19th & 20th July 2000 at Mbarara

APPENDIX VI
Partners in DRWH – organisational profiles

APPENDIX 1: PARTICIPANTS AT THE RWH SEMINAR, MBARARA, 19TH – 21ST JULY 2000

List of participants by name

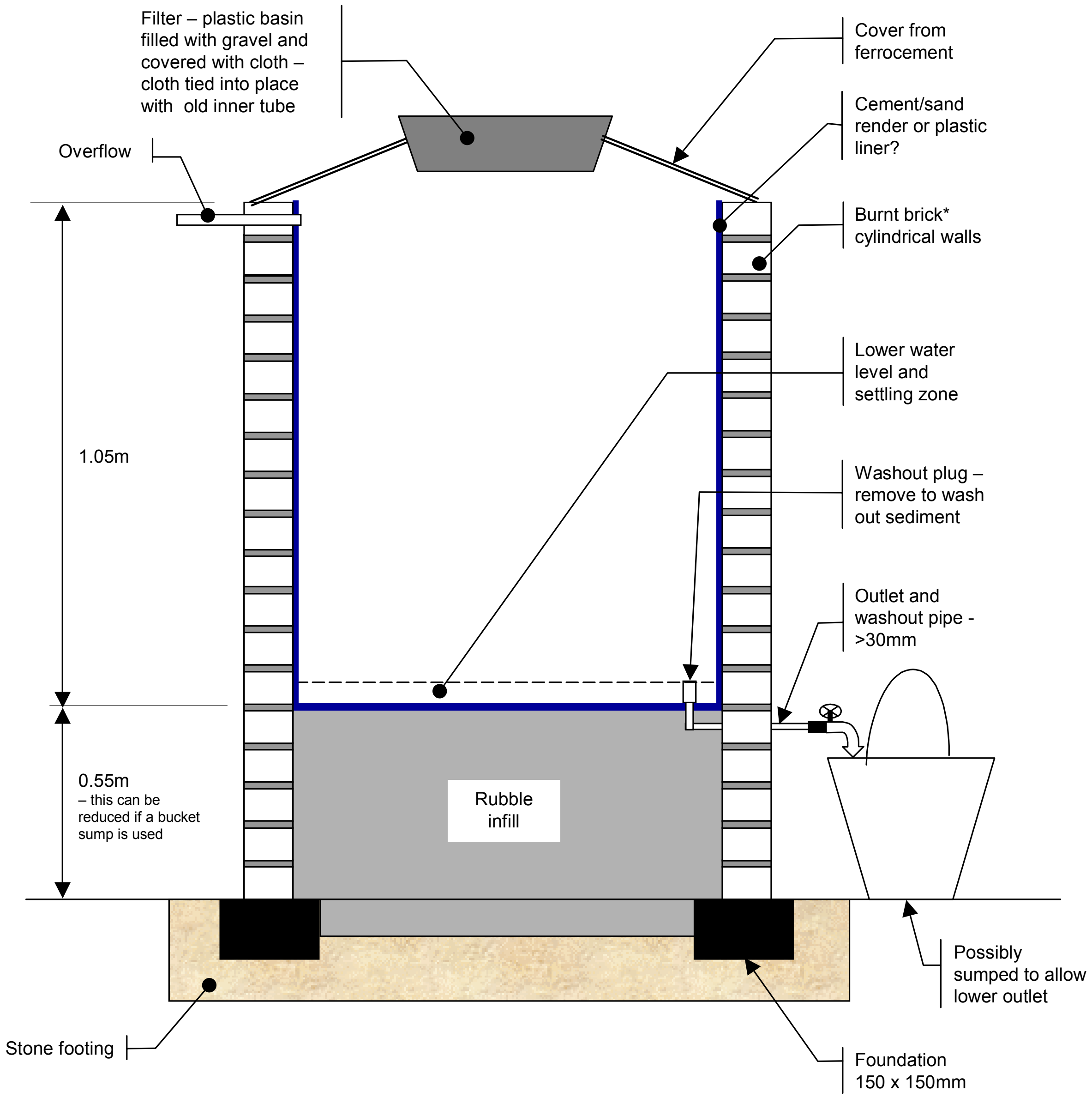
	Name	Organisation
1	Mr Angelo Nzigye	Biharamulo Rural Appropriate Technology and Innovations Society (BRATIS)
2	Mr Byaruhanga Moses	Uganda Rural Development and Training Programme (URDT)
3	Mr Nirere Sam	Uganda Rural Development and Training Programme (URDT)
4	Mr Victor Turyamureba	Uganda Rural Development and Training Programme (URDT)
5	Mr Oswald Kasaizi	KARADEA
6	Mr Mugisa Kimarakwija	ARUCED
7	Mr Charles Rwabambari	ACORD
8	Mr Edward Ahimbisibwe	Kyera Farm
9	Mr Timothy Tibaijuka	JUDEA
10	Ms. Caritas Mukankusi	Lutheran World Federation (LWF)
11	Mr Ssemanda Edward	Rakai District Water Development Department
12	Mr Bariyo Rogers	Mbarara University of Science and Technology (MUST)
13	Mr Turyaramya Moses	IVA Mbarara
14	Rev Eric Kamutera	North Kigezi Diocese
15	Mr Kaleega William	DWD, Mbarara / Uganda Rainwater Association (URA)
16	Rev George Bagamahunda	Kigezi Diocese Watsan Programme
17	Mr Swithen Nyakaana	DWD Mbarara
18	Dr Terry Thomas	Development Technology Unit
19	Mr Dai Rees	Development Technology Unit

List of organisations participating in seminar

Organisation	email	physical address	Contact person	Tel / Fax
Uganda				
ACORD (Oruchinga Valley Project - Mbarara)	acordug@uol.co.ug	PO Box 1394, Mbarara, Uganda	Charles Rwambambari (Water)	T 256 41 256 41 267667 / 267668 F 267669 (Kampala)
URDT	urdt@swiftuganda.com	PO Box 24, Kagadi	Mr Byarushanga Moses Mr Victor Turyamureba Mr Sam Nirere	T 256 0483 22820/1
ARUCED		PO Box 1028, Kyenjojo, Kaberole district	Mr Kimarakwija Mugisa	T 256 (0) 483 22756 F 256 (0) 483 22636 (Fort portal post office)
Department for Water Development – Rakai		PO Box 1, Rakai, Uganda	Mr Ssemanda Edward	
Uganda Rainwater Association (URA)	wesw2.dwd@imul.com	DWD Kampala	Kimanzi Gilbert (DWD Kampala) Kaleega William (DWO Mbarara)	T 077 500602 (Mob) 220374 / 220560 (Office)
North Kigezi Diocese WATSAN		PO Box 23, Rukungiri	Rev Kamuteera Eric	T 256 486 42230
Kigezi Diocese WATSAN	kdwd@imul.com	PO Box 3, Kabale	Rev George Bagamuhunda	T 0486 23940
Kyera Farm		PO Box 1577, Mbarara	Mr Edward Ahimbisibwe	
DWD Mbarara		PO Box 704, Mbarara	Mr Swithen Nyakaana	T 256 485 21011 Mobile 256 077 594518
MUST	highland@imul.com	PO Box 1410, Mbarara	Mr Barigi Rogers	T 0485 20642
IvA (Mbarara)	rottiers@africaonline.co.ug	PO Box 1592, Mbarara, Uganda	Mr Moses Turyaramya	
Tanzania				
BRATIS	drdp-biharamulo@twiga.com	PO Box 79 Biharamulo, Kagera, Tanzania	Mr. Nzigi Angelo	T 028 255 222634 F: 255 66 23222 (via post office)
Karagwe Development Association (KARADEA)	ndeki@unhcr.ch	PO Box 299, Karagwe, Kagera	Exec Sec. Oswald E. Kasaizi	T 255 66 22541/22971 F 255 66 22541
JUDEA – partner org of IvA (Bukoba)	mayawa@twiga.com	Bukoba	Mr. Timothy Tibaijuka	
Rwanda				
Lutheran World Federation (LWF)	lwf-rw@rwandatel1.rwanda1.com	B. P. 2831, Kigali	Ms. Caritas Mukankusi	
UK				
Development Technology Unit	dtu@eng.warwick.ac.uk	DTU, School of Engineering, University of Warwick, Coventry, CV4 7AL, UK	Dr Terry Thomas (Director) Mr Dai Rees (Researcher)	T 44 24 76522339 F 44 24 76418922

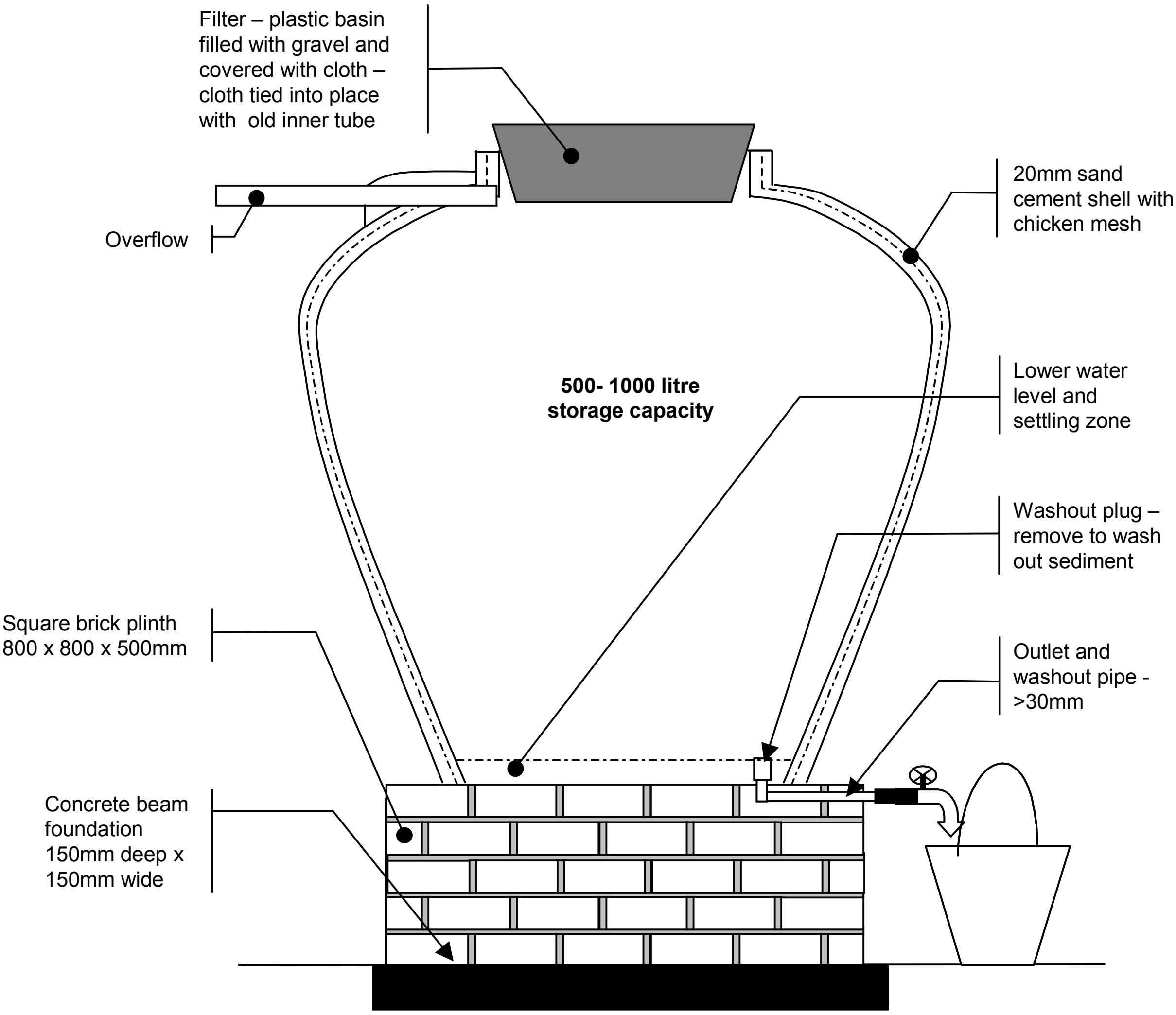
APPENDIX II: DESIGN DRAWINGS (VLC RWH SYSTEMS)

Brick jar – 750ltr



Notes:
- Number of courses – 21* * Burnt brick size – 225 x 100 x 75mm
- Number of bricks per course - 14
- Total number of bricks - 294
- Shaping of bricks or using half bricks
- Cover - conical from ferrocement
- Good stone footing under foundations – 100mm deep
- Size of tank will be about 750 litres useful storage capacity
- If drainable bucket sump is possible then infill section can be reduced, reducing number of bricks for construction

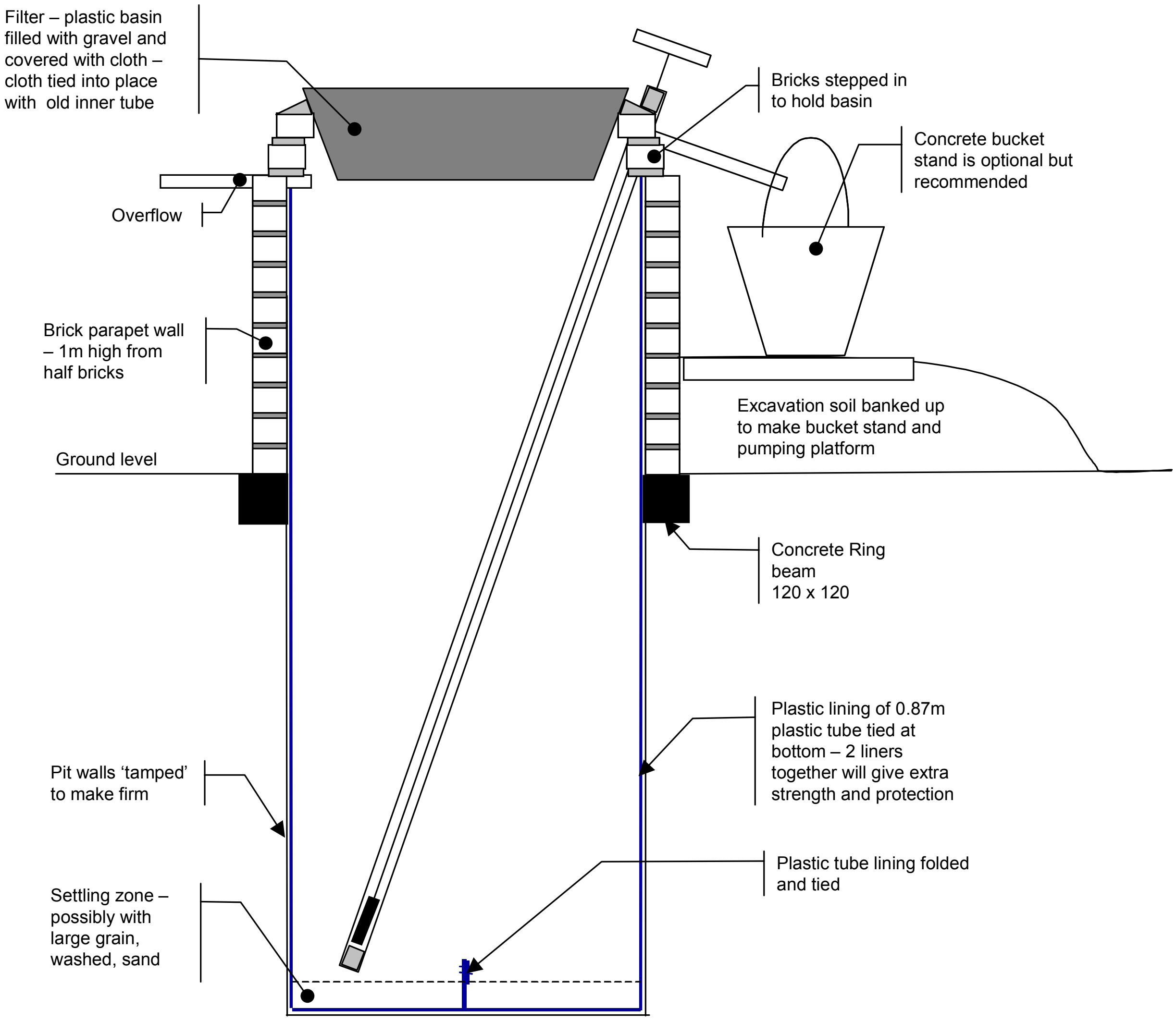
Ferro-cement jar – 500ltr



Notes:

- This version of the ferrocement jar uses a polypropylene sack mould that is filled with sawdust / rice or coffee husks and is then rendered. It uses an optional single layer of chicken wire. The level of skill required (based on recent experience in Kyenjojo) is quite high. Size is about 500 – 1000 litres depending on size of mould (dimensions below for 500 litre mould).
- There are a number of alternatives: using block moulds, wooden moulds, making cylindrical tanks using a number of different mould types,
- We could possibly simplify the design by using a collapsible cylindrical or octagonal mould. This would be easier to handle and transport

Plastic tube tank –600ltr



Notes:
- This a small version of the tube tank and could be developed to give a storage of say 600 litres or so.
- A handpump developed for use with this tank is very low cost and very low maintenance.
- This version uses a tube of plastic sheet which is turned up and tied (0.87m flat tube is available on the market). Two layers of plastic offer extra protection against leakage. The 0.87m (flat) plastic tube gives a diameter of 0.55m. This is the internal diameter of the tank.
- The capacity for each metre of depth is 230litres. Digging inside the pit can be difficult, but is aided by using a long bar and a long handled hoe. Depths of 2 metres can be reached this way, and with the 1m of parapet wall this gives a capacity of 600litres.
- The sand in the bottom prevents damage of the liner should someone enter the tank and aids cleaning (the settled matter can be scraped from the sand.
- The tank will be monitored for the following: damage, durability, cleaning, repair, replacement.

APPENDIX III: SMALL JAR COSTS

Ferrocement jar costs

Item	Unit	Number required	Unit cost	Total cost	Total US$	Total £
Cement	kg	68.5	300	20,550	13.70	9.26
Sand	kg	245	20	4,900	3.27	2.21
Aggregate <50mm	kg	40	25	1,000	0.67	0.45
Bricks	no	100	42	4,200	2.80	1.89
Rubble	kg	75	0	0	0.00	0.00
Chicken mesh 0.5"	m	3.2	1,667	5,334.4	3.56	2.40
GI Pipe 1"	m	0.5	4,200	2,100	1.40	0.95
GI Elbow 1"	no	1	1,500	1,500	1.00	0.68
PVC Pipe 1.25"	m	0.5	1,667	833.5	0.56	0.38
Tap 0.5"	no	1	5,000	5,000	3.33	2.25
Reducer 1" – 0.5"	no	1	1,500	1,500	1.00	0.68
Basin	no	1	1,000	1,000	0.67	0.45
Labour (skilled)	days	2	5,000	10,000	6.67	4.50
Labour (unskilled)	days	4	3,000	12,000	8.00	5.41
	Material costs			47917.9	31.95	21.58
	Total cost (including labour)			69917.9	46.61	31.49
	Cost per litre stoarge			95.84	0.06	0.04
	Cost per litre storage (incl labour)			139.84	0.09	0.06

Notes

1. Mould cost not included - cost of mould is approximately 6000 Ush and may last for up to 10 or 15 jars depending on care taken during manufacture
2. Larger sizes of jar - say up to 1500 litres - can be achieved by experimenting with the mould size
3. Wall thickness is approximately 20mm but varies due to bag shape. Keeping the wall thickness of the f/c to the suggested thickness helps keep cement costs down.
4. Sawdust is obtained from local sawmills (sometimes at a small cost)
5. The volume of the jar is obtained by using a bucket of known volume and counting the appropriate number of buckets of sawdust
6. The jar can be made without the plinth, but water extraction is then difficult - dipping with a container can then be practised. This would significantly reduce the cost of the jar
7. Some transport costs included (i.e. for sand, aggregates and bricks)
8. Cost of bucket slab not included
9. Omitting the tap reduces cost but contamination is more likely to occur

Brick jar costs

Item	Unit	Number required	Unit cost	Total	Total US$	Total £
Cement	kg	92.5	300	27750	18.50	12.50
Sand	kg	390	20	7,800	5.20	3.51
Aggregate <50mm	kg	40	25	1000	0.67	0.45
Bricks	no	300	42	12,600	8.40	5.68
GI Pipe 1"	m	0.5	4,200	2,100	1.40	0.95
GI Elbow 1"	no	1	1,500	1,500	1.00	0.68
PVC Pipe 1.25"	m	0.5	1,667	833.5	0.56	0.38
Tap 0.5"	no	1	5,000	5,000	3.33	2.25
Reducer 1" – 0.5"	no	1	1,500	1,500	1.00	0.68
Basin	no	1	1,000	1,000	0.67	0.45
Labour (skilled)	days	2	5,000	10,000	6.67	4.50
Labour (unskilled)	days	4	3,000	12,000	8.00	5.41
	Material costs			61,083.5	40.72	27.52
	Total cost (incl labour)			83,083.5	55.39	37.43
	Cost per litre storage			81.44	0.05	0.04
	Cost per litre storage (incl labour)			110.78	0.07	0.05

Notes

1. Some transport costs included (i.e. for sand, aggregates and bricks)
2. Cost of bucket slab not included
3. No mould required
4. The tank size can be increased slightly by increasing diameter, but tests should be carried out to determine the strength of the jar.
5. Omitting the tap reduces cost but contamination is more likely to occur
6. Where the ground slopes suitably, the plinth height can be reduced, thus reducing cost

Plastic tube tank costs

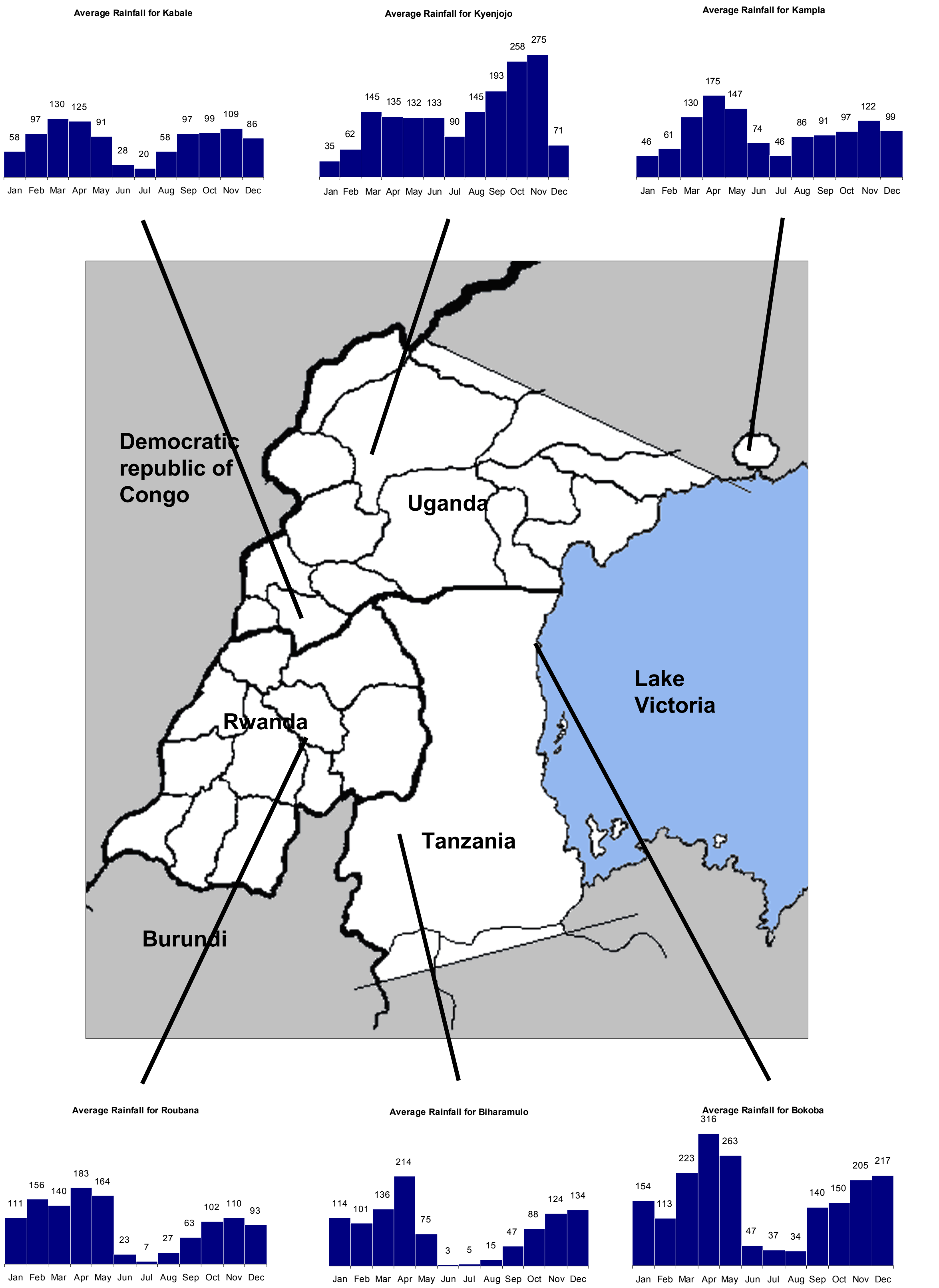
Item	Unit	Number required	Unit cost	Total	Total US$	Total £
Cement	kg	33	300	9,900	6.60	4.46
Sand	kg	136	20	2,720	1.81	1.23
Aggregate <50mm	kg	20	25	500	0.33	0.23
Bricks	no	115	42	4,830	3.22	2.18
Harold pump	no	1	15,000	15,000	10.00	6.76
PVC Pipe 1.25"	m	0.3	1,667	500.1	0.33	0.23
0.87 flat plastic tube	m	6	1,000	6,000	4.00	2.70
Basin	no	1	1,000	1,000	0.67	0.45
Labour (skilled)	days	1	5,000	5,000	3.33	2.25
Labour (unskilled)	days	2	3,000	6,000	4.00	2.70
	Material costs			40,450.1	26.97	18.22
	Total costs (incl. labour)			51,450.1	34.30	23.18
	Cost per litre storage			67.42	0.04	0.03
	Cost per litre storage (incl. labour)			85.75	0.06	0.04

Notes:

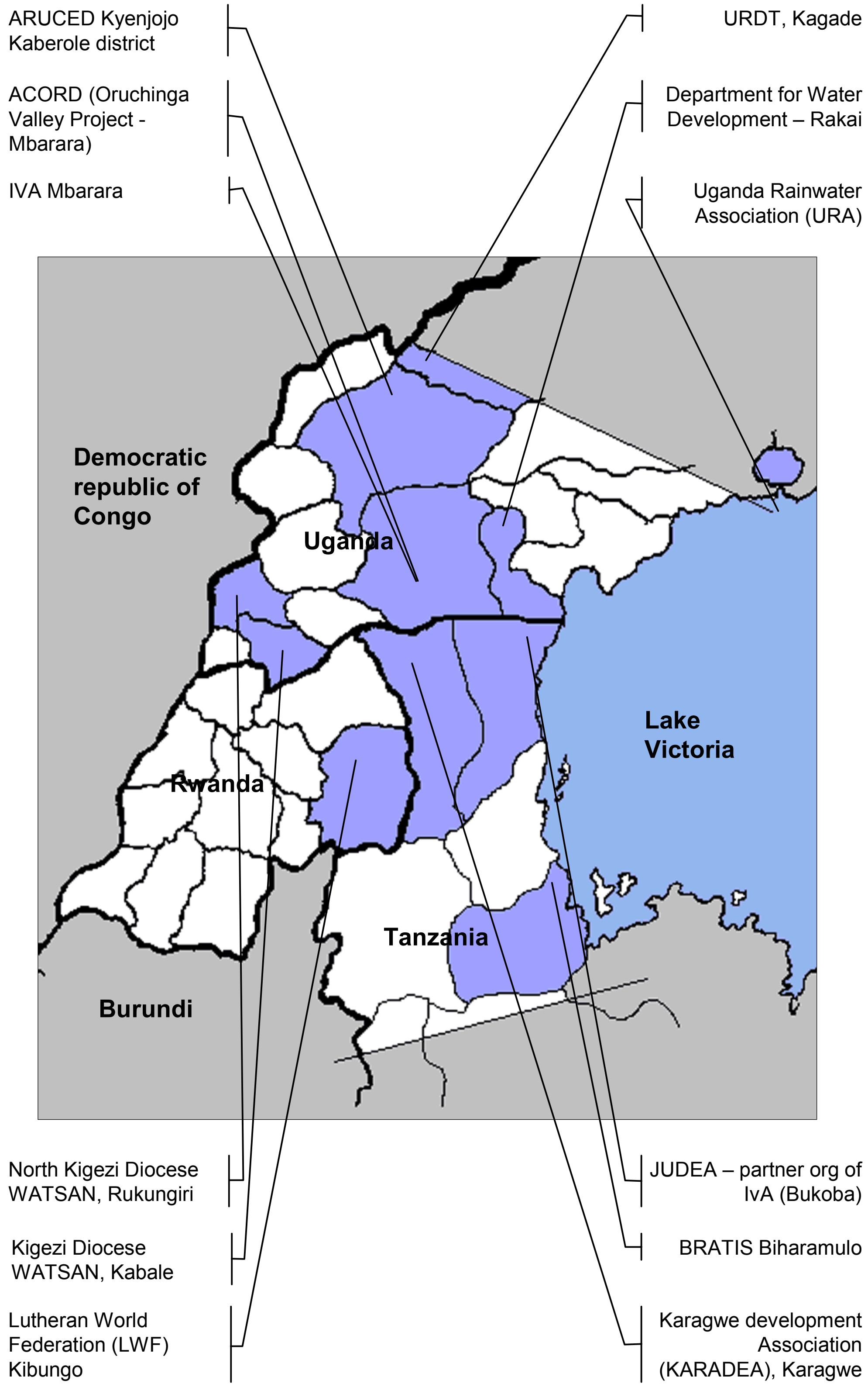
1. Diameter of Jar is 54cms
2. Diameter of polythene sheet is 55cms
3. The ring is cast at 54cms
4. Some transport costs included (i.e. for sand, aggregates and bricks)
5. Cost of bucket slab not included
6. The handpump can be omitted but water is then more prone to contamination and the lining is more likely to be damaged

APPENDIX IV: MAPS

1. Map of the Target Area showing average annual rainfall



2. Distribution of organisations



APPENDIX V – MINUTES OF SEMINAR HELD 19TH & 20TH JULY 2000 AT MBARARA

The two-day Seminar was held at the Pelikan Hotel, Mbarara, Uganda

Timetable for the seminar

DAY 1

9.00 a.m. - Opening address - Kimanzi Gilbert - DWD

9.05 a.m. - Self-introductions by participants

9.15 a.m. - Dr. Terry Thomas - Introduction to the Seminar and its objectives

9.45 a.m. - Mr. Swithen Nyakaana - the findings of the Feasibility Study - a brief overview

10.15 a.m. - Break for coffee

11.00 a.m. - Presentation - the experience in Rukungiri (North Kigezi Diocese) - followed by discussion

12.00 a.m. - Presentation - the experience in Karagwe (Karadea) - followed by discussion

1.00 p.m. - Break for lunch

2.00 p.m. - Presentation - the experience in Rakai (DWD) - followed by discussion

3.00 p.m. - Break for coffee

3.30 p.m. - Presentation - the experience in Rwanda (LWF) - followed by discussion

4.30 p.m. - End of Day 1 Activities

DAY 2

7.30 a.m. - Breakfast

8.30 a.m - Field visit 1 - **UNIFA** (Tarpaulin Tank)

10.00 a.m. - Field visit 2 - **Kyera Farm** - (Small jars, Partially below ground tank, Ferrocement tanks, Experimental Rammed Earth tanks, Grass roofs). *Tea will be served at Kyera Farm*

12.00 p.m. - Return to Mbarara

1.00 p.m. - Lunch at Pelikan Hotel

2.30 p.m. - Mr Dai Rees - technical issues related to small-scale RWH - followed by discussion

3.00 p.m. - Discussion - small-scale RWH Technology

3.30 p.m. - Break for coffee

4.00 p.m. - Time for reflection and comments

5.00 p.m. - End of Seminar

1. Mr Kimanzi was unable to attend the Seminar and the opening address was made by Dr Terry Thomas, who warmly welcomed all participants.

2. Dr. Terry Thomas, Director, DTU, University of Warwick, UK. Introduction to the seminar - background, aims and objectives.

Contents of the presentation:

- Types of Roofwater Harvesting and our focus today
- Small systems
- DTU Roofwater Harvesting Programme
- Key Questions

Dr Thomas discussed the following issues:

- Institutional vs Domestic RWH – and how the bias often been toward large institutional RWH by the funders
- ASAL vs Humid zones – Dr Thomas pointed out that the focus of past RWH programmes has tended to be in arid regions and for full coverage through RWH
- Total, partial, seasonal or casual. Dr Thomas outlined these 4 ‘styles’ of RWH and briefly discussed the relative merits of each.
- Water for people, cattle or gardens? During this study we are concerned mainly with water for people, possibly with a small surplus for animals or the garden.

- Our focus today. Dr Thomas pointed out that we are focusing on domestic, humid zone, partial or seasonal, for poorer people
- "The fifty thousand shilling system". Is this system achievable, and is it affordable and attractive to the poor of the region?

Dr Thomas then spoke about the law of diminishing returns and the relatively good return from a small scale system (e.g. a five day supply of water can give 70% of total water needs, wheras a 100 day supply may only give 95% coverage).

Dr Thomas spoke briefly about the DTU roofwater harvesting programme.

- Dr Thomas then presented some key questions to the participant, questions that he hoped would be answered during the course of the Seminar:
- How does RWH compare with other sources in terms of…….. cost and effort?, ...reliability?,.... water quality - (actual and perceived)?.... ease of installation?….. general image?….. permanence?
- How much can "most" people afford to spend on water supply?
- The practicalities of multi water sourcing?
- Who are best suppliers of domestic RWH systems - householders, fundis, self-help groups, industry, local ngo's, local government?
- What are major constraints to the use of RWH?

Mr. Swithen Nyakaana, DWD, Mbarara (and Social Scientist employed to carry out the Feasibility Study). An overview of the findings of the Feasibility Study to date.

Mr Nyakaana presented the findings of the feasibility study to the participants. He emphasised that there is a need for RWH in the region and that the technology is feasible, both technically and economically. Some of the factors for choice of RWH system include:

- Relief and topography
- Soil texture
- Existing water facilities (e.g. ponds, pans, valley tanks, wells, boreholes, etc.)
- Quality of water
- Demand
- The availability of hard catchment surfaces (usually in the form of corrugated iron (mabate) roofing (figures quoted were Uganda 60 – 80%, Tanzania 40 – 60%, Rwanda – no figures available)
- Maintenance costs are low
- Time saving

He pointed out that RWH is:

- practised locally and the skills required are readily available
- known to local people as a viable alternative water supply
- suited to the area in terms of climate
- suitable to individuals or community groups

The identified constraints were noted:

- Transport and communication are poor and expensive
- Some skills need improving
- There is insufficient choice of technology available
- Information is not readily available in the area – there is inadequate extension service

The Rev Eric Kamutera of North Kigezi Diosece (NKD, Rukungiri, Uganda) then shared the experiences of his organisation with the group.

Rukungiri is situated between 2000 and 3000 above sea level. It is a densely populated area with 200 people per km^2 and an average annual rainfall of 1500mm, falling between March and June and then between September and December. Recently the climate has been erratic and changeable. Rev Kamutera stated that RWH has great potential in the region and already 5000 homes, of the 100,000 or so homes, has DRWH. Water collection in the area is very difficult as the terrain is hilly and very muddy in the wet season. Most of the homes in the area have hard roofs. Since 1992, NKD have been promoting cement jars, ferrocement jars and ferrocement tanks in the region. They have worked closely with Water Aid (who funded the RWH programme) to promote the 250 litre jars, working closely with women's co-operative and women's groups. The aims of the water programme have been threefold:

- to provide wholesome water for drinking (the beneficiaries were advised to conserve the water for drinking purposes only)
- to reduce the burden of women and children
- to supplement existing sources

Some of the systems built have been communal, some for individual homes. There have been some management problems with the communal systems. The average family size is 6 – 8 people.

The constraints faced by the programme were noted as:

- limited funds
- limited storage capacity of the tanks
- problems with management of communal systems
- poor design or lack of choice (particularly covers, inlets and construction quality – some tanks failed due to poor construction)

Local initiative resulted in larger jars – up to 5000 litres) being constructed. These were reinforced with chicken wire and steel bars.

Rev Kamuteera recognises the limitation of their top down approach to RWH dissemination and recently the Uganda Rainwater Association (URWA) has been training women's groups in the area. NKD are convinced that RWH is a serious alternative for rural water supply in their area.

Mr Ssemanda Edward from Rakai District Department for Water Development (Uganda) presented the experiences of their work in RWH.

The rainwater programme in Rakai started in 1997 when a group of women from Kenya trained two groups form Rakai district in the skills of water jar construction and also in self expression and confidence building techniques. The two groups then built a tank for each of the group members using funds from a revolving credit scheme. The Twekembe group now builds tanks for profit as well as being involved in other profit making activities such as jam making and baking. They have trained a further 17 women's groups. The Katuntu group have trained groups from 3 other Districts and some are going to Zambia in September to train women there.

The water situation in Rakia District is difficult. It is a dry area (compared to neighbouring Districts) and the groundwater is highly mineralised. The main water sources in the area are ponds and swamps.

The appraoch taken by Rakai DWD was to 'give' only what was essential – i.e. cement on occasions, but to encourage people to rely on their own resources to supply other materials and labour. To date 300 jars (500 litre) and tanks (2500 litre) have been built in one county, with as much as 30% coverage in some areas. The cost of the jars is approximately 160,000 Ugandan Shillings (£72) and the tank 220,000 Ugandan Shillings (£100).

Please see the report titled 'Rakai Women's Groups Involved in Rainwater Harvesting Activities – Women Leading the Development Process in the New Millenium' (distributed during Seminar).

Next Mr Charles Rwabambari took advantage of some spare time to present the work of ACORD, based in the Oruchinga Valley, south of Mbarara.

The ACORD programme started in the Oruchinga Valley in 1987 with the objective of improving the standard of living of the people and improving household income. The programme identified water as one of the key problems in the area and has developed a programme based on hand augered wells and RWH (Rock catchment, tanks and dams).

However, donors were not keen to support RWH activities. After a visit to the 1993 IRCSA Conference in Nairobi, ACORD staff arranged a visit for a local women's group to the Laikipia region of Kenya. The women in this region had earlier received training in tank construction and were willing to train their Ugandan sisters. The experience is documented in a video titled 'Mvua ni Maji' – Rain is Water.

The implementation method used by ACORD is that for every domestic tank constructed by the group, ACORD provides funding for another tank to be built.

Problems faced by the organisation include:

- inflation
- drought

- no clear government policy on household water security
- poor communication
- dependency on imported materials
- low priority on rural water supply by the authorities

The opportunities include:

- economic improvement in recent years
- benefits of RWH easily seen
- skills are readily available in the region
- RWH provides a better quality of water
- simple O & M
- reduction of burden on women and children

Future work includes:

- continued support for community initiatives
- further improvements in household water security
- further networking with other organisations in RWH

Mr Oswald Kasaizi of KARADEA was next to share his experiences of RWH in the Karagwe District of NW Tanzania.

KARADEA was started in 1987 with the objective of solving community problems. Their location is the remote area of NW Tanzania and they serve an area 6700 km^2 where there is an annual average rainfall of 1000mm (bimodal). They started with 10 integrated projects, provision of water being one of these. The terrain is hilly and water is a serious problem. In 1990 their RWH programme got underway with the construction of 1400 litre jars. RWH was seen as being affordable and managable. A British VSO volunteer who was working with KARADEA at the time helped to start the programme. A 50% subsidy for tank construction was provided by VSO initially but later the tanks were sold at cost price. Meetings in the communities were arranged and women were targeted. Water committees were established with a 75% female balance. Initially things were slow and attendance at meetings was poor, but eventually word got around about the benefits of RWH and things took off. Fundis (male and female) from the villages were trained. Eventually 5 workshops were set up for semi-centralised manufacture (the finished jars [500kg] were transported to site from the workshop on donkey carts for up to 10kms). Selling jars was difficult in this low-income region due to the 100,000 TSh cost, and so a rotating fund was established to help promote the technology. People paid an initial 25,000 and the remainder was then repaid into the fund over a one year period.

Problems faced:

- transport
- very high cement costs (almost 3 times the price of cement in Dar es Salaam)
- competition with other organisations
- low income in the region
- lack of hard roofs in the region

Solutions

- rotating fund established
- subsidised sale of mabate sheets
- external funding sought from Government and NGO's

In 1996 / 97, after the return of the Rwandan refugees from the area, an international NGO copies the KARADEA example but jars were given to beneficiaries. KARADEA could no longer sell jars and so started producing larger ferrocement tanks (10 – 25m^3). These have also been successful.

To date it is estimated that about 600 individuals households have RWH jars. AT the KARADEA HQ water from their numerous tanks is sold for income generation! RWH is seen as being a very promising water supply option but pressure is needed on Government to promote RWH and to alter policies favourably.

The final organisational presentation was given by Caritas Mukankushi of the Lutheran World Federation (LWF), Kibungo, Rwanda.

Rwanda is a country of 8 million people, and 23,000 kms^2 (pop density of 350 / km^2). LWF works with refugees and also in the resettlement camps, where the returned refugees (returnees) are now living. Generally Rwanda is well endowed with springs, and so spring protection and gravity systems are common. But the area where LWF is based do not have these springs. Danida has funded the drilling of boreholes in the area. RWH is limited to the returnee settlements in the arid Kagera National Park. The technology used is a 1200 – 1800 litre ferrocement jar which is built onto a bamboo basket mould. The tank is for individual household supply, uses 200kg of cement, has a wooden cover and costs between US$220 and US$250. LWF realise that the cost is high.

Identified constraints

- High cost – the community cannot afford to purchase the tanks themselves
- Limited rainfall
- Limited skills
- Poor information dissemination

After field visits to both the premises of the Uganda national Farmers Association (UNIFA) and Kyera Farm Training Centre, there was a presentation by Mr Dai Rees of the DTU, titled 'Technical issues related to small-scale Domestic RWH'. The content of the presentation is outlined below:

- What is small-scale RWH?
- Why small-scale RWH?
- The benefits of small-scale RWH
- Reverse economies of scale
- Actual benefits to the user - a study in Kabarole District
- Some low-cost technologies for small-scale RWH - experiences at Kyera Farm

Firstly Mr Rees described what he sees as being small scale RWH

- Storage capacity of 500 - 1000 litres
- Provides several days storage for a typical household (but not inter-seasonal storage)
- Suitable for partial water supply
- Used in conjunction with other water sources e.g. distant spring or borehole
- Suitable for humid climates with evenly distributed rainfall
- It is low-cost and therefore affordable by poorer families

He then looked at why small-scale RWH is well suited to the region:

- Low-cost and therefore more affordable
- Suitable for local construction by artisans, women's groups, farmers groups, etc.
- Suitable for regions with evenly distributed rainfall pattern e.g. SW Uganda, Rwanda, NW Tanzania
- Adoption can be incremental – people buy more capacity as and when it can be afforded
- A significant supplement to existing sources - say providing 50 - 80% of total domestic water supply

He then looked at the sensitivity of annual coverage to roof area, daily demand and tank size, using rainfall figures for Mbarara to demonstrate the point and also using figures form a recent study carried out in Kabarole District (Uganda) where a number of small systems have been built by the DTU and ARUCED.

Mr Rees then went over the costs and designs of for the small RWH systems that had been seen at Kyera Farm earlier in the day (and are shown elsewhere in the Report document).

Finally Dr. Thomas led a discussion as to the future of the DTU's involvement in RWH in the region. He particularly tried to ascertain what the participants saw as being the most suitable dissemination strategy for the region for the proposed dissemination programme for which the DTU hope to secure funding. The discussion homed in around two differing strategies, namely a product based, market orientated approach or a more traditional developmental approach as exemplified by the Rakai experience. The participants were divided as to which may be the 'best' option and different organisations felt drawn to one or other (or both) of the strategies. It was generally agreed that it would be interesting to proceed with both approaches and to monitor the outcome.

APPENDEX VI - PARTNER ORGANISATIONS IN DRWH

North Kigezi Dioceses:

Organisation location: Rukungiri district in SW Uganda

Visited by Swithen Nyakaana May – July 2000

Population: Rukungiri district = 390,780

The organisations main activities:

The North Kigezi Diocese WATSAN is a water and sanitation organisation. It is funded through its religious background and aims to improve people's development. People apply for support from the church in various fields: Water and sanitation, primary health care, farming and tree planting. The organisation has majored in water supply improvement and advancement of H/H water technology. The organisation is funded by the church and has worked in partnership with Water Aid U.K.

Administrative structure:

The organisation is divided into two sections, the management and the extension team. The management is composed of the co-ordinator, spring supervisor and the hygiene and sanitation group leader, and these are supported by the Water Aid representative. The extension team is composed of the water-jar officer, senior fundi (mason/craftsperson), sanitation senior fundi, sanitation/hygiene educator, driver/mobiliser and the contractors (fundi's). There are 10 people in the team; 3 of these are women and they deal with the software within the programme. The target group is organised by the local councils, chiefs, religious leaders, opinion leaders, CBO's and NGO's.

Topography and physical features:

The area is hilly with some very steep hills (around 250) separated by narrow valleys. The valleys have seasonal rivers but few springs. The people live in these areas because the soils are very fertile; crops grow well though there is a problem of lack of markets. This is attributed to bad roads and steep terrain. The problems faced due to these features are shortage of water, bad roads, little income due to poor markets, a burden of carrying water loads on the steep terrain. The water that is available from seasonal rivers is often dirty. The soils are loamy/clay, stony and volcanic soils. The hill slopes have stones of varying sizes. The grounds are very stable, pits are unlikely to collapse, so unde ground tanks can be constructed. In the valleys, there are some sandy pits, the sand is mixture of clay and silt. The sand is not currently used for tank/jar construction.

Rainfall: 800-1000mm

The area experiences rainfall mainly in two phases March to May and Sept to Dec. There are some trace rains in late July and August.

Economic activities and major crops:

Rukungiri is mainly a subsistence farming area, the crops grown include, banana (grown for food and beer brewing) maize, rice, beans, millet and potatoes. Most of the crops are eaten by the H/H.

Major H/H expenditures:

The main expenses for H/H's include, school fees, health services, agricultural implements, paying for labour charges, building houses, water jars, clothes, drinks, food and dowry payments.

Purpose for intervention in RWH

The purpose of the organisation's intervention with RWH is to focus on areas with people in the Rift Valley & other hilly areas with either no springs or only unprotected springs. The organisation intends to promote RWH as a technology, because there is a lot of potential due to widespread hard roof coverage (70% - 85%) and sufficient rainfall (800-1000mm). RWH will reduce water shortage and amounts of water loads being carried by women & children from the valleys.

Achievements of the organisation

The organisation focused on training women groups in jar construction in the sub-counties of Bwambara, Kihiihi, Kambuga and Nyarushanje. The total number of group members trained has been forty but most of the group members have failed to use the skills due to lack of money. It takes a very long time for the group to collect money among themselves, due to low incomes. The women groups are constructing 800 l tr jars costing USh.120,000/-.

The organisations approach is basically training and demonstrations. The groups being assisted are those that have been in existence for some time. The group selects a site for the demonstration and the community takes responsibility for managing the facilities. The organisation has a sanitation component, so group member are required to ensure that latrines are available.

Agency's opinion about effectiveness:

- The organisation supports the promotion of small water tanks/jars. The beneficiaries have expressed the need for training in the construction of Ferro- cement tanks.
- The organisation lacks external support as Water Aid no longer work in the district.
- The organisation has learnt that technology development requires a strong economic base from the beneficiaries.
- The organisation has trained groups in jar construction but it feels there is need to make improvements in making jar covers also guttering is a problem.
- RWH is not only an alternative water source but can also be an income generating activity, especially to women groups…

  Example: Mrs. Mungereza was asked what she has benefited from her membership of a water group.

  "I call upon fellow women to join the groups, at first people thought it was time-wasting. But now, after constructing my 3000 ltr tank in addition to 350 ltr water jar. In the previous rain season, I sold water & got USh.80,000/-; I was about to sell all the water but I reserved some for my H/H. From the money, I have bought home utensils. My husband and has promised to support me in constructing another tank".
- The organisation, looks at the people as poor, so there is need for subsidies to these people so that they can get started.
- The organisation is seeking support from donors, because the water problem is still on.

Observation:

- The groups could benefit from being informed on ways of disseminating existing technologies, e.g. phased construction can help the beneficiaries rather than advocating big tanks, as they claim to being poor.
- The organisation has emphasised the promotion of above-ground tanks/jars and underground tanks have not been demonstrated, however the inclusion of the latter could have given the beneficiaries another possibility to choose from.

ACORD

Organisation Location: Orichinga valley, Mbarara district

Visited by Swithen Nyakaana in May–July 2000

Population: Mbarara District = 930,772

ACORD started in southern Sudan in the early 1950's in war torn areas by inter-agencies response to people's problems. The organisation was initiated by Oxfam to solve people's problems by relief and community rehabilitation by initiating income-generating activities. The other partners in funding include Britain, France, Europe, Canada & Belgium. ACORD has many branches in Africa and one of which is in Mbarara.

ACORD Mbarara operates in the southern part of Mbarara District bordering Tanzania. The project offices are in the Oruchinga Valley. The organisation operates in the three sub-counties of Kabingo, Ngarama and Kikagati.

Administrative structure:

ACORD Mbarara has two main sections, management and extension team and employs 20 staff. They work in different sections, i.e. management, field staff & support staff, and cover credit/micro-finance, agriculture, water & Sanitation, HIV (STD) and gender issues. The communities they work with are organised under the local council (L.C) system; other leaders include Chiefs, elders, religious leaders and community based organisations (CBO).

Topography and physical features:

The area is quite hilly with very few natural water sources. The hill slopes are covered with stones and rocky outcrops. The area experiences water shortage, poor communication, limited health facilities, lack of markets due to bad terrain and limited extension services.

Rainfall

890mm

Economic activities in the project area:

The basic economic activity is subsistence agriculture. Banana growing is the main H/H income earner. Other activities include keeping animals, growing beans, & groundnuts, brick making and stone sales Some people in the area salary earners.

Purpose of intervention and opportunities for RWH

The organisation started by assisting refugees & distributing seeds but this was gradually phased out. A self-reliance participatory methodology took root and people started requiring other services such as support for income-generating activities. The purpose for the organisation's intervention in RWH was to improve the quality and quantity of water for H/H and sanitation and hygiene.

The target group was the population with water stress in the three sub-counties of Kabingo, Ngarama & Kikagati. These are the areas where H/H experience water shortage most of the time because the area has limited natural water sources. A limited number of individuals have tried out their own water ponds. There has been an increase in the population as many people have been migrating to the areas but the water sources remain scarce, this is one of the reasons behind the organisations intervention.

The availability of hard roofs in the area is 90% the catchment area the average H/H is 54m2. People have started income generating activities. They are ranked rich = 17%, medium 65%, poor =15% and destitute = 3% so the money can be channelled to technology development

Impact:-

- Some women have started home gardens and zero grazing.
- H/H have built individual tanks
- The trained artisans have got employment
- RWH has been considered as a supplementary water source.
- H/H with tanks collect less water from more distant sources

Achievements of the organisation

ACORD supported the communities by training interested groups in demonstration tank construction, which includes providing transport for all materials. The capacity of the demonstration tanks are generally 6m3. A ferrocement design of about 4 m3 and a much cheaper, partly-underground, tarpaulin design have been developed and promoted. In the case of the former, a group of 9 women build and share tanks until there is one at each household. A few much larger community systems have been built in plateau areas. Where ACORD has trained, the skills are now available and people have initiated low cost tarpaulin tanks, and the people have a spirit of working together. The average H/H tank cost USh.400,000/- to USh.600,000/-, 48 tanks have been constructed after the demonstrations.

Specific problems of water collection and availability of water

Women and children mainly carry out water collection, though the children have been noted as the ones that collect the most water for H/H. During serious droughts men are hired to transport water from more distant permanent sources (springs, gravity fed systems).

The people expressed problems with accessing water, lack of finance, high charges for skilled labour. The cost of a 6 m3 tank is USh.150,000/-. There is a high transport charge for the sand: 7 tons of sand cost USh.20,000/- but with transport this becomes Ush.100,000/-

Agency's opinion of effectiveness

Training: Identified groups should be trained by use of demonstrations at the beginning of the project, and tours conducted to view new technologies. This is seen as a valuable method of awareness creation. It creates economic empowerment, reduces costs for the project, the multiplier effect is high and the skills remain within the community.

Subsidy: Quick project implementation and enhances group participation which creates a sense of ownership.

Private water provision (RWH): To benefit the poor the provision should be through group formation by selecting a feasible technology, then the poor can plan for the tank size according to their economic base.

Organisations Future plans: Continuity and sustainability:

To train artisans & pump mechanics, involve government structures (e.g. LC5s) & District extension staff, train groups in accountability and group dynamics and give the groups a chance to originate the choice of the technology. As a multiplier mechanism, locked up information can be disseminated via seminars and inter-agency forums to form a common front.

Observations

- For a project to succeed there is need to first initiate and sensitise the communities/target group to initiate income-generating activities, then invest in the technology development.
- Training, demonstration and subsidising costs makes the technology advocated far simpler and awareness takes root in a short time.
- The organisation has mainly one type of technology, this being the ferro-cement tank. The community may still miss a more favourable option, which could have a higher adoption rate.

The organisation has put a limit in terms of tank size which it can subsidise (i.e. 6m3). This may have been a problem to the target group by not giving them chance to choose the type and size of tanks to build.

Rakai District Water Development

Organisation Location: Rakai district

Visited by Swithen Nyakaana, May – July 2000

Population: Rakai District = 383,501

The organisations aims and activities:

The organisation deals basically with rural water development and is under local government control. The partners in funding are SIDA and Rakai District Administration.

Administrative structure:

The organisation is composed of water staff, health assistants, community development assistants, drivers and a secretary. As a means of dividing out the extension activities, the department employs a district water officer, county water inspectors, sub-county health assistants, and sub county community development assistant as well as drivers and secretaries.

Due to the nature of the work there are few women involved in the hardware section, though one women is in the field of community development. The communities are organised under the leadership of LCs, religious leaders, opinion leaders, cultural leaders, NGOs and CBOs.

Topography and physical features:

The area has gentle and steep hills (15°-25°) with no rivers and few swamps. The valleys have seasonal water bodies due to ground run off. These ponds provide water for both domestic and animal use. The main problems faced due to the physical features are poor road networks because of the hilly terrain and a lack of clean and soft water. A notable feature of the District is that its soils are highly mineralised. Any water in contact with soils is either salty or changes colour owing to this mineral contents: thus little groundwater can be used.

The soil texture varies depending on the area, i.e. silt in valleys, hilltops have murram with graded stones and sandy /clay in other places. Some areas have under-laying rocks at 3m deep. The ground and soils are stable which favours underground tank construction. The sand from the valleys is used for the mud rendering of house walls but is regarded as of poor quality.

Rainfall: 800-1000mm

The area experiences two rainy seasons, one in March-May and the second in August-December.

Economic activities in the project area:

Mainly subsistence farming with banana plantations (grown for both food and beer), coffee, potatoes (Irish and sweet), maize, beans, animals (hens, goats) and brick making.

Purpose for intervention in RWH

The purpose for the organisation's intervention in relation to RWH was to promote an alternative source of water for H/H's and reduce walking distance to collect water. It also aimed at tapping water before it touches the ground as this would prevent it becoming so highly mineralised. The target group for the organisation's intervention is H/Hs and women in particular.

The organisation basically deals with training. The women are required to form groups, and then apply to the Chief Administrative Officer; then the District Water Officer assesses what the group requires. Demonstration training for the group is conducted on site with selected group members. The philosophy in all the steps is participatory and the technology being promoted is on jars and Ferro–cement tank construction. The groups requested specific training skills for constructing bigger tanks of 3000-6000 ltr. Underground tanks have not been tried out though it is hoped that the cost may be much less. However, there is need to analyse the effect of cement on the highly mineralised soils.

Achievements of the organisation

The training in tanks of bigger capacity, using welded mesh as a mould has already been done. The welded mesh is provided by the organisation after which the training is left with the community. Most of the jars constructed by the women groups are of size 1500 ltr, they cost USh.250,000/-. The bigger ferro-cement tanks of 7000 ltr cost Ush.510,000/-. The communities note that the ferro-cement tanks are are cheaper per litre than jars.

Women groups have been empowered and they are constructing RWH facilities on their own. The three women groups in Rakai have constructed tanks on the H/H and have been given a chance to organise and plan for what they want, other than being guided by the support agency. Women groups have become more group oriented after seeing that group formation makes things easy.

Specific details:

Rolf Winberg a representative for SIDA's E Africa office in Nairobi responded to the people's call in relation to the mineralised water problem. Through the District Water Office, womens group representatives and some staff members travelled to Kenya to train in rainwater jar construction. After the training, the women formed three womens groups and started constructing water jars.

Agency's opinions for effectiveness observations

- Communities should be supported and guided to identify their needs, then they should be given a chance to organise themselves.
- There is need to integrated sector in development in order to achieve high adoption rate in the technology being advocated for.
- In development, there is no need to form parallel bodies when working in the same area. There is need for integration and support to each other.
- Dissemination of skills and training is the best tool to be handed over to communities if they are to work on their own with minimum external support.
- Women are the basic elements to faster change. So to every intervention there is need to involve the most affected persons, they know their problems better.
- Let the groups start from where they are with their limited resources. As their income base stabilises, they will demand more complex

facilities which will suit them at that particular time.

- Private water provision is more feasible when done at H/H level, this develops a sense of ownership

Observations

- Demonstrations make participants more involved and they can see their potential benefit; this leads to strong groups, and it encourages other new members. The people tend to demand more services, which leads to building capacity in the target group (planning and implementation).
- Empowerment starts with skill dissemination, which leads to self-reliance in both decision-making and a demand for more needs or support.
- Credit schemes were found more effective when beneficiaries raised the funds for the scheme themselves
- Subsidies to target groups activates group formation and a sense of ownership is developed. This further creates competition among the group members and the adoption rate is high.
- Provision of key components helps the group to access non-locally available materials commodities. The organisation's presence is felt.
- Construction in phases: communities should be encouraged to start where they are by building small tanks. The demand for others can be met by continuous construction of other small tanks with in the limits of their economic base.
- Sustainability: the basic strategy for this involves training, involving local leaders on ground and the beneficiaries, should use their own resources to multiply the technology.
- Technology development: for the technology to develop there is need for other income-generation activities by the potential adopters of it. RWH as a programme on its own is difficult to adopt or develop when the income base is very low. Technology development can be hindered by several factors, lack of extension services and/or lack of markets/income. There is need to work in an integrated way by involving other agencies that deal in those particular fields.
- Many people have abandoned tank construction due to fear that the sand is bad and the structures would crack. So, there is need to support the communities with transport facilitates so that cost of this critical raw material can be lowered.
- Most people are negative when looking at communal water structures, because such structures often end up in the possession of individuals.

  Case Study: "A communal water tank of 30m3 was constructed at the chiefs' house because it had the biggest catchment in the village. After one year the chief made a fence around his home and bought dogs, the people then abandoned the tank". This showed that the group lacked any laws to guide them in case one of them lacked trust from fellow members.

KARADEA (Karagwe Development Association).

Organisation Location: Kagera Region of NW Tanzania, bordering Uganda & Rwanda

Visited by Swithen Nyakaana, May – July 2000

Population: Not known

The organisations aims and activities

The association was initiated by both the community and the donors, A British volunteer Alice Morris strengthened the project with a grant from Britain. The grant at the start was used in conducting sensitisation workshops, and for training fundis in water jar construction. Sweden gave aid to the association with the intention of improving peoples living standards by providing clean water.

KARADEA's objectives include "the emancipation of women" and it contains 31 women groups. Its activities, as reflected by its organisational sections as listed below, include administering three loan funds, seminars for women and help to orphans RW jar construction has been undertaken for 13 years. Although for 2 years high cement costs and the temporary intervention of a new (refugee-support) NGO providing free jars interrupted this activity.

Administrative structure:

KARADEA has the following sections: Water, Health & Nutrition, Solar, Finance, Appropriate Technology, Education & Research, Women's Development, Afforestation and Youth Development. The water section is sub divided with technicians and a water contractor who work on a part-time basis. They have an annual general meeting with the board of directors, the executive secretary, and heads of department. There are more women than men on KARADEA's staff because the programme is intended to assist women in easing their problems.

Communities are organised under village leaders, opinion leaders, religious leaders chief and NGO's. The most active person in mobilisation for the development programmes is the Village Executive Officer.

Topography and physical features

Generally the whole region is quite hilly.

Rainfall = 1000 mm

The region receives two rains, one in February–May and the other in September-December.

Economic activities in the project area:

The main income generating activities is subsistence farming. Most of the products are consumed at H/H level; these are beans and maize. Ground nuts, bananas, tomatoes and coffee are the main cash crops.

Purpose for intervention in RWH

The organisation recognised that access to water was very limited and in 1987, with assistance from Sweden and Britain (under Alice Morris), training artisans in jar construction was initiated.

Achievements of the Organisation

They introduced plastic bags of 1000 ltr, after some time it was discovered that they were prone to damage by termites. The bags were finally rejected by the communities. After the training the jars were bought by the individual beneficiaries. There is still a demand for services from the project by the community. Beneficiaries then started applying for assistance, the organisation would make costs for them depending on the distance from the head officer or nearby workshop, the beneficiaries would prepare the platform (the seat for the jar). The organisation provides transport for the jar from the workshop and makes/provides gutters. People are allowed to pay in instalments: contracts are made specifying when to fulfil the payments.

The common technology is the sand/cement (mortar) jar of 1400 ltr costing TSh.5,000/-. Some people have started constructing ferro–cement tanks and underground stone masonry tanks.

The quality-control advantages of manufacturing jars in specialised workshops have been offset by the difficulty of then transporting them (fragile and heavy) across difficult terrain. At one stage donkey-cart transport was introduced into Karagwe solely for this task. As a means of reducing transport distances, five workshops branches have been established. These are managed by the beneficiaries some of whom are employed in the workshops. Branches make reports to the heads of departments and the executive secretary. Every department makes reports on the activities conducted.

The organisation promotes training for both staff and beneficiaries. Two approaches are used to train staff, an expert is called in to train specific sections or staff are taken to selected institutions.

Agency's opinions about effectiveness:

- Training: the beneficiaries, artisans and staff involved in new technologies are a means of creating more awareness.
- Demonstration: the organisation does not use H/H demonstrations, but shows models in public places or jars are taken to exhibitions/open shows.
- Establishment of branch workshops: to reduce contact distance for the beneficiaries and organisation.
- Formation of groups and water committees: as a means of empowering the community. Phased jar construction by individual H/Hs ownership, as a means to meet people's water needs, in

small bits, by building jars in sequences with time.

NB: Very few H/H can construct tanks large enough to meet the whole water needs of a H/H. The common and affordable tank size for a H/H, depending on funds, is 5.6m3 = (4 jars of 1400L). These can be built one after the other. Beneficiaries need assistance in obtaining materials that are not available locally. The organisation success is partly dependent on providing other inputs i.e. transport, skills and supervision).

Observations

- For a programme to succeed there is a need to integrate technology development with other income-generating activities. Very few technologies can be adopted when the income levels for the beneficiaries are very low. Improvements to extension services and markets could lead to technology development
- Many households have more than one water jar. It has been noted the beneficiaries do not feel the total cost when jars are bought at different times.
- Children collect more water in the H/H than the men or women. Both men and women control the funds in the H/H, yet it is the children that suffer from collecting water. Children have no say on the funds to be invested on RWH.
- Jars and tanks may take a long time to buy when the H/H has many children as income is directed to higher priorities i.e. school fees.
- Portable water jars may be a problem when they are carried to areas that are very hilly.
- Jars are not made by individual H/Hs due to concerns that:

  1. It can reduce the quality of the jar constructed
  2. The cost for transporting sand is very high.
  3. There is a need to subsidise the jars to the community and the people's income is very low.

BRATIS (Biharamulo Rural Appropriate Technology Innovation Society)

Organisation Location: NW Tanzania.

The organisation is located in Biharamulo district N W Tanzania. Five other districts, Ngara, Muleba, Geita, Kahama and Kibondo surrounds Biharamulo. The organisations area of operation is Busenga, Husahunga and Nyakanazi. The head quarters are in Katooke and the other office in Biharamulo town.

Visited by Swithen Nyakaana, May – July 2000

Population: not known

In the initiation of the organisation the target group was not involved, they only benefit from the organisation through awareness creation and training. The organisation started as a voluntary youth group under the Caritas Church organisation (Diocese of Rulenge). After 3 years it become independent and registered in 1995. Mr. Angelo Nzigiye and the youth group initiated the organisation. The partners in funding are comprised of the Tanzania team, the District Council, and subscription by members. 50% of the funds are generated by executing tasks in construction and carpentry workshop

The organisations aims and activities

The broad objective is to animate communities to participate fully in their own social and economic development and aims to establish an organisational infrastructure among disadvantaged people.

Administrative structure

The organisation has a board of six directors, management committees and co-ordinator departments of carpentry, agriculture, entrepreneurship and technology.

Educated women often go to the bigger towns in search for better pay, as gender issues are not viewed as important topic at present. The organisation employs people of low cadre and trains them on the job.

The organisation's activity co-ordination is done through reporting and meetings are intended for planning and ironing out differences. The executive meetings are conducted every Monday while the Board and General Assembly meet every year. Each department reports independently and the reports are then co-ordinated to formulate policy on how the organisation is run.

The communities where the organisation operates are organised by village leaders, religious leaders, opinion leaders, traditional leaders and NGO's

Topography and physical features.

The organisation's area of operation has undulating gentle hills covered with stones and boulders; there are valleys and swamps, which serve as water sources for the area. The soils are fertile with natural vegetation. Firewood is scarce in the area and there are water shortages in the dry periods. Swamp water is dirty, some of the sources are on hills and in the valleys. Soil varies depending on the location: in the valleys there are clay/sandy, on the hills there are the red laterite soils and red loamy soils. The area has large rocky outcrops and the soil stability varies with the location. Sand in the area is available and is used for building constructions.

Rainfall

Not known but likely to be over 1000 mm per year, with 1 long and 1 short rainy season..

Economic activities in the project area:

The major activities are agriculture and the main crops are maize, beans, like coffee, sweet potatoes, keeping goats and cows. Other sources of income are charcoal manufacture, brick making, salary earners, carpentry and tailoring.

Purpose for intervention in RWH

The purpose for intervention in RWH was that there was a limited number of water sources available. The low amount of 10L/ per person/day was seen as another valuable reason for RWH intervention.

The organisation's aim is to try out RWH technologies for the beneficiaries to appreciate, adopt and practice (multiply). The main problems are that few people have hard roofs, 50% of 40-50m2 and the technologies appear to be costly. The target groups for the intervention were the villagers and areas with acute water shortage. This would help women suffering from the drudgery of water collection. It is the institutions that have taken up the technology by contracting the organisation to put up large tanks of 43m3 and some institutional VIP latrines.

The entrepreneurship department conducts the economic assessment for the area and determines the beneficiaries' contribution. The organisations approach is in sensitising the community on RWH. Identifying and training the artisans on the selected affordable technologies (jar/tanks). However, the adoption rate is still very low as the target group claims the technologies are very expensive.

Agency's opinions about effectiveness:

- The organisation has no intention of ending its services because the problems in water are still on and the youth groups still need support, so the programme should continue trying to resolve the problem, lack of support and funding are limiting factors.
- The organisation aims at assisting H/H by constructing small jar/tanks of ferro-cement and under ground tanks. Due to low costs.
- As a means of multiplying the technology. The beneficiaries, should is to participate in training to become artisans, the organisation is to give technical advice.
- On information services, the organisation has set up training centres where the youth go to seek advice and new technologies. The organisation has formed a networking forum for local NGOs.
- The organisation supports the idea of public demonstration, but if the first structure fails the idea being demonstrated does not take root. Unfortunately most demonstration systems in public places lack ownership and therefore good maintenance.
- Training the users in system construction may not be economical, when the trainees do not use the skills attained. The better approach is the training of artisans, as skills and employment are created and the multiplier effect is noted when there is the availability of funds. This is further developed if the group's activities are

subsidised, this creates a higher chance hat the technology in question is adopted.

- Most beneficiaries have failed due to lack of key components for RWH. There is a need for the provision of key components & services to be brought nearer, giving a reduction in cost/litre. For example tools and equipment such as moulds could be hired.
- As a means of reducing cost/litre at the time of construction, water provision for the poor should be implemented though group formation and subsidy
- The organisation has started income generating activities, promotion of improved bananas, making tiles – as a means of uplifting H/H income.

Observations:

- According to the organisations approach, if the target group is to benefit and adopt the intended technologies there is need for group formation, subsidy provision and credit scheme association as a means of empowering the individual H/H to construct their own water structures.
- The organisation may have good intentions and approaches in reaching the target group, but with limited funds and support this is difficult. The intended objectives may take long to be realised. Like wise, if the target group has financial constraints, the message and ideas sound beneficial but implementation remains a problems.
- HESAWA (Health through sanitation and water) had different approach in the same area, its approach is more towards donation. (The beneficiaries are required to contribute local materials for a 20m3 structure). This has been a hindering force for BRATIS to succeed. The beneficiaries look at BRATIS's requirements as expensive in comparison to HESAWA.

  CASE STUDY: NGO's with very similar activities may complement each other or compete, to the extent of displacing each other from either the area of operation or the technology".

- According to the beneficiaries' claims, the best tank size to meet the H/H water needs, may be constructed in the phased approach (one jar after another) up to the capacity of (4 to 6 m3).
- The organisation has started on projects intending to raise people's income, (introduction of new banana species, making roofing tiles, carpentry. Many technologies fail due to low income in the target group. No new technology can develop before the target groups' brains are trained and changed towards the new approaches. (Specific training).
- The organisation intends to network with other NGO's but there has cropped up a problem of extra payment to the focal people that run day-to-day activities. This has led to the need for the formation of a secretariat for NGO's.

  NB: (In collaboration there is need to avoid too demanding partners).

JUDEA. (JUhudi DEvelopment Association)

Organisation Location: North west of Tanzania bordering Uganda.

Visited by Swithen Nyakaana, May – July 2000

Population: Not known

JUDEA is a rural development NGO in Bukoba, Tanzania, it focuses on poverty alleviation and improving the living conditions in the rural communities. The organisation was initiated by Mzee Ishengoma and registered under co-operative organisations involving individual partners. The organisation was started on the basis of the individual annual subscription of 5000 TZS, and an entry fee of 2000TZS as the basic funds for the organisation. The Belgians (IVA and VIC) have given support to the organisation. IVA supports agriculture and VIC support JUDEA in under taking drilling. The administrative structure for the organisation is the chairman, the executive committee and the general meeting.

Administrative structure

JUDEA is an organisation initiated and run by retired people, mainly professionals.

Topography and physical features/soils:

The area where the organisation operates has gentle undulating hills, valleys, swamps and rivers. The soils are mainly loamy lateritic and murram. The ground is stable and favours underground tanks.

Rainfall: 1900 mm per annum

Economic activities in the project area:

The economic activities in the area are mainly agriculture or subsistence farming. The main cash crop in the area is coffee. Other crops include beans, bananas & maize etc.

Purpose for intervention in RWH

JUDEA wants to promote RWH as a source of water supply and assist elderly people of Sawata & Sadiawazee Tanzania, by constructing H/H water jars tanks to reduce water stress. The target group for the organisation is the mostly the elderly H/H,s but women and children are a focus as well. The criteria used in choosing areas to benefit from the RWH programmes are those areas that have water stress, no alternative water sources, those with hard roofs and where the people are willing to form groups and contribute.

Agencies opinions about effectiveness

- As the organisation has not fully participated in RWH system building, many of their ideas are waiting to be turned to reality.
- The organisation intends to introduce and promote water jars of 500 ltr to 1000 ltr, and encourage underground tank technology using tanks of capacity 2000 to 7000 ltr.
- The organisation does not intend to end the programme, it wishes to continue but the organisation lacks founders and financial/technical support.
- The organisation has a limited number of staff, due to lack of funds. It intends to establish a training scheme and open up a workshop for water jars, and public demonstration on low cost technologies as a means of awareness creation.
- Though the organisation has not implemented RWH activities, suggestions on possible means on approach & dissemination of RWH technology were noted.
- Group formation, mobilisation and sensitisation for RWH and create a structure, which creates awareness and strengthens RWH as a technology.
- The communities need to be introduced to simple techniques, affordable and with in their financial reach.
- As a tool to promote RWH, there is need to form a collaborative NGO body and implementing agency in RWH to form a common front in promoting the technology.
- Government and community based organisations need to have documented policies regarding the approach & extension to support the dissemination of RWH technology.

Observations

- The organisation has a vision to support the elderly and the poor who have no capacity to contribute towards the services they require. The group would require a grant for the organisation to achieve its goal.
- The other population if mobilised and sensitised would take up the organisations idea because water is a problem to most H/Hs.
- HESAWA (HEalth through SAnitation and WAter) has been operating in the region, so the sensitisation required is not new, and the organisation needs to introduce the feasible technologies and train the target group on implementation.
- The composition of the organisation may cause a rejection by potential groups, as there may be a bias towards elderly people only. There is need to integrate other players in the organisation. The youth and women can play a good role in community mobilisation and sensitisation.
- There is need for both the organisation and target group to train or visit other areas implementing RWH so that the organisation can gain awareness of available technologies. Skilled labour and raw materials could be considered.

IVA + Uganda Nat Farmers Association, Mbarara District

Organisation Location: IVA has its office in Mbarara town, S Uganda.

Visited by Swithen Nyakaana, May – July 2000

Population: Mbarara district = 930,772

IVA is a Belgium farmers alliance service similar to UNFA. The organisation provides funding to support farmers in Mbarara. The organisation works through UNFA members, UNFA members contact the office for technical support /skills development.

The organisations aims and activities

IVA operates throughout the whole of Mbarara district. They use a bottom-up approach when executing the extension, IVA has its representative up to parish/cell level and deals with the grass root farmers.

Administrative structure:

The organisation has a project management team composed of two Europeans and two Africans. The support staff comprise of agricultural teams and a secretary. The target groups are organised through committees, groups and the leaders include LC chiefs, opinion leaders, religious leaders and NGO's.

Topography and physical features:

The organisation's area of operation varies considerably, it therefore deals with hilly areas, valleys without or limited natural water source, valleys with a flat basin, steep hills with rocky out crops like those in Rugaaga S county. The hillsides are used for both crop production and as grazing land. Water sources are few and far between in the hilly regions.

The soils are of a variety, black soils, sand loamy clay soils are found in the valleys, while stony, murram, soils found on the hillsides are very compact and difficult to dig, but may be suitable for underground tanks. IVA operates in one of the areas with the best sand pits (Nyeihanga). This sand is good in jar/tank construction.

Rainfall: 890mm

The rainfall in Mbarara district occurs in February-May and Sept-December with some light occasional rains in July.

Economic activities in the project area:

The income generating activities include banana plantations, coffee growing, animal keeping (cattle, goats, hens and rabbits) sand extraction, charcoal etc.

Purpose for intervention in RWH

The organisation conducting a needs assessment survey in the area which showed a water shortage. Many areas have no or limited natural sources of water. In the survey it was noted that most of the water sources are located in the valleys, with ponds and springs being used for both animal and human use. From this survey IVA felt that they could meet people's needs by introducing RWH systems.

Achievements of the Organisation

In the course of one year 10 groups have been formed and 8 ferro-cement tanks have been constructed. The organisation carries its activities out through UNFA members, at the parish level people are mobilised to form water groups, when groups are formed and contribution accepted two agreements are made one for the group and the other between the group and the organisation. Then group member selects local builders and a demonstration training session is conducted.

Agencies opinions about effectiveness

- IVA promotes training the beneficiary groups as a means to create skills in the target groups. This is done on public demonstration.
- Prior to technology development there is need to create income for the target group so, that the funds from their produce is again put into the new technology development.
- As an entry point there is need for group formation, this encourages the group to access financial support and funds are easily generated for an activity.
- The organisation encourages the idea of inter-agency collaboration, this increase information

flow and there is a gain from sharing the experience and/or technical assistance.

- The communities should not be limited on a particular tank size or technology, the people know their problem best and how to solve them e.g. the organisation demonstrates; on 3000 ltr tank costing USh.400,000/- but the communities now are constructing 5000 ltr tanks.

Observations

- As IVA deals with UNFA members and those that are not members do not have access to the organisations services.
- If UNFA fail to deliver then IVA will also be regarded as having failed.
- H/H with a regular (but low) income could improve themselves through a RWH programme. It has been noted that most of the H/H with rainwater tanks are those that who have benefited form UNFA programmes. Therefore technology development requires external financial support, this can be generated locally or through subsidising.

LWF (Lutheran World Federation) in Rwanda

Organisation Location: The organisation has its country office in Kigale.

Visited by Vince Whitehead, July 2000

Population: Not known, but population or Rwanda is currently 7.3 million and population densities are generally high

LWF is a worldwide religious organisation and deals with humanitarian assistance. It started in 1946 as a world service provider by the federation of Lutheran churches. The partners in funding include the Federation of Lutheran churches of Europe, North American and Australia. The organisation operates especially in areas of Rwanda to which former refugees and IDPs have recently returned, including in the Ndego II sector of Kibungo, Kigarama Departement and in Gitarama Departement. The population served is between 6000-7000 people but the number is increasing, as there are more people returning home.

Administrative structure

LWF consists of a director, a programme co-ordinator, and three project managers there are thirty-two staff in total. Nine out of the thirty-two staff are women.

Topography and physical features:

Much of this area of Rwanda is characterised by high hilly regions and isolated steep valleys. Many of the peaks are rock-strewn, with little vegetation. Lower down the slopes are a mixture of rocky outcrops and lateritic soils. Erosion from the rains regularly causes deep ruts in the unmetalled roads. The occasional hurricane has been strong enough to take the CI sheets off houses. The soils are very stable and favour underground tanks; in the area there are some rocky outcrops.

Rainfall: About 1000 mm

The area experiences rain from February-May and October-December. The Kibungo district was suffering a serious drought during the time of the visit.

Economic activities in the project area:

The main economic activities in the area are the selling of agricultural produce and animals (goats, hens).

Specific details

In Ndego II, people were walking 6 km twice day collect water; the round trip took about two hours on foot and one hour on a bicycle. The water sources in the area include lakes and shallow wells (the wells were protected by NCA, Norwegian Church Aid). Water collection is done by women and children. In drought seasons some people hire others for water collection. In the communes hard roof coverage is 100%, each with a catchment area of only 15-20 m2.

Purpose for intervention in RWH

In Rwanda as a whole, gravity piped water schemes are common. However they are not easily applicable in the specific target areas of LWF. Rainwater harvesting has not been developed much in Rwanda, but the potential of assisting returnees is high. All the roofs in the communes are new and made from corrugated iron, which is an ideal solid catchment area for RWH. Also because of the six month long drought season there is a desperate need to alleviate some of the problems faced with extreme water shortages. Many of the tanks currently built are very costly and have limited capacity, which seems insufficient to assist them through the dry season.

Achievements of the Organisation

Some of the work in building tanks has been built on experience of the returnees from Tanzania seeing the Umutara tanks. The tanks were made from a basket of reeds covered with cement mortar and placed on a stone foundation. The inside of the tank is rendered with waterproof cement. Communities are required to organise them selves via meetings and to provide sand and stones. LWF provides cement, guttering and skilled labour. The organisation has provided credit not specifically for RWH, but to micro-financing and agricultural schemes. There has been very little capacity building through training. One training session for masons was conducted with twelve men and one female being trained in house construction.

Agency's opinion about effectiveness

- The organisation provide materials as a gift to returnees at the beginning, this is now being scaled down and will probably be on a oan basis in future.
- The involvement of CBOs in spring protection, shallow well and gravity flow scheme development. Water committees are formed and representatives are selected. Representatives come together and build up larger bodies for all water system management.
- The communities in Mugesera district were shown video's on RWH by women's groups from Kenya and other realisation developmental communications.
- On the means for collaboration, high demands from the returnees have initiated collaboration from UNHCR/ECHO. This has been used as a methodology to all developmental programmes. The beneficiaries put forward a request for the services they require rather than donors giving them what the donors wish to give.
- The organisation suggests starting with water tanks of small capacities (1.5 or 2 m3) for H/Hs; the more prosperous H/Hs can construct 5m3 water tanks.
- The organisation intends to create local capacities to take over the responsibilities after five years. The idea is still on trial to see if CBO's can look after themselves.

Observations:

- The organisation has had little dealing with RWH technology in the past, but it has a strong need to initiate the advancement of RWH in its region of operation.
- Some tanks in the community for returnees that were observed in the National Parks cost around RFr.20,000 (about $50) - the high cost was because of the high cement content, transport cost and for a technicians services.
- The area where the organisation is operating is still faced with new returnees and their resettlement, so the technology to develop at the beginning may require a push from the implementing organisation. The returnees have very little to offer and have very little material possessions, they lack any real land security during early resettlement and their contribution may be hard to mobilise.
- Depending on the distance from the existing permanent water sources (lake, gravity flow scheme) the idea of RWH can still be promoted as a jar or tank as an asset for a particular H/H giving water 'on the step'.
- The organisation needs to sensitise communities & train artisans from their area of operation on the new, available & affordable technologies in the region. Tours visits, and demonstrations could be the basic tools in disseminating the information about RWH technology.

Kigezi Diocese: Kabale

Organisation Location: Kabale

Visited by Swithen Nyakaana, May – July 2000

Population: 417,218

Kigezi Diocese Water Project is a church initiated project and aims to ease people's water shortage problems. The partners in funding at present are Tear Fund, Irish Embassy, DFID and the beneficiary- communities. The role of the beneficiaries' community is supplying locally available materials.

Administrative structure

Water Project staff comprises a programme co-ordinator (water engineer), an assistant water engineer charged with construction and an accountant. The organisation has a health component. There are four female co-ordinators responsible for monitoring/ functionality. The construction section consists of masons who build water jars/tanks, spring protections, san plats, and other constructions. The technicians are involved with the plumbing, pipe laying and tap-stand fixing. Higher cadre officers do the supportive supervision. Within the communities are organised through the LCs, opinion leaders, religious leaders and NGOs.

Topography and physical features:

Kabale District is high, cool and very hilly. Population density is very high and for many years Bakiga have been emigrating to other parts of Uganda, There are no significant rivers; springs are located down in the valleys and near to Kabale town there is the picturesque Lake Bunyonyi. The soils are very fertile for crop production, the though the area suffers soil erosion. Cultivatable land is covered with many stones, which makes tillage difficult. The soil is composition of inteharaea(?), murram, sandy/silty soils, and sandy loamy. Many stony grounds are porous, the ground is stable and could favour the use of ground tanks. Sandpits are located in the valleys; the sand is used in building construction.

Economic activities in the project area:

Economic activities in the area are basically subsistence farming, with crops of sorghum, beans, Irish potatoes, sweet potatoes, & millet.

Rainfall: 1000-1200 mm.

The area experiences two rainy periods of March to May and September to December.

Specific details

Most H/H experience water shortage, as springs are usually quite distant. There are long dry spells and small containers are used for RWH. A jar of 350 ltr costs USh.90,000/-. The water shortage is overcome by collecting water from distant places: lakes and permanent springs. RWH is practised by few H/H. Women and children collecting water from springs may be queuing up to an hour during the peak times, walking time for the round trip may be 1.5 hours, which is over steep terrain. Missionaries introduced gravity flow schemes as a means of reduce the amount of time and walking for water. Though gravity flow water routes are determined by the terrain, people who live uphill quite a distance away from a source have their water problems not solved but reduced. Those that have tried out RWH have faced such difficulties as scarcity of money buying sand and transportation of materials to site, skilled labour.

Purpose for intervention in RWH

The purpose for the organisation's intervention is to minimise water-borne diseases, to reduce time spent in water collection and to introduce new low-cost technologies for H/H water supply. The target group is the rural community at both institutional and H/H level, but focussed on relieving the water burden of women and children.

Achievements of the Organisation

The organisations entry point is at the parish level. The mobilisation and sensitisation is done by software on how the contributions are shared. Benefitting H/Hs are required to provide only 'local materials' i.e. earth mortar, while the project provides skilled labour, cement, moulds and gutters. Beneficiaries pay only USh.7000/- (out of a total cost of USh.90,000/-) for a 350 ltr jar. An average H/H can make 350 ltr last for 6 days. The organisation requests a payment for training artisans.

As a means of sharing their experience the organisation collaborates with the District Water Office (Health), LCs, churches, EU, DFID, Irish Embassy, women groups, Tear Fund, Rotary clubs and the government. This has helped the organisation in sharing experiences on new technologies, identified resourceful people and avoided the double counting of RWH structures.

Agency's opinion about effectiveness.

- The communities in the area of operation are poor so there is need to identify low cost technologies within the reach of the low-income base. Even a USh.50,000/- jar seems too expensive for the communities.
- Subsiding should be looked as the only means of encouraging people to start RWH.

Observations:

- The project contribution is very high, though some H/H do not seem to have picked up the donations. With more sensitisation many H/H are now welcoming the RWH technology.
- The organisation has not trained many artisans in the area. For all the works done, the project staff provides the skilled labour. For one to be trained he is required to pay for the training. The community solely depends on the project's skilled labour.
- The organisation has promoted water jars only where people have been looking at small structures. There is need to introduce new technologies, i.e. underground tanks, which are potentially lower cost per litre.

Domestic Roofwater Harvesting
Research Programme

THE OPTIMUM SIZING OF GUTTERS FOR DOMESTIC ROOFWATER HARVESTING

by Gwilym T. Still and Terry Thomas

December 2002

Development Technology Unit

School of Engineering, University of Warwick

ABSTRACT

Guttering, in a roofwater harvesting system, has the purpose of intercepting the roof run-off and conveying it to a downpipe (which in turn carries it to a store). The two phases of 'interception' and 'conveyance' make different and sometimes conflicting demands upon a guttering design. Their respective failures (overshoot and overflow) occur under similar circumstances, namely intense rain, and for most analytic purposes it is appropriate to consider as total water loss just the higher of the overshoot loss and the overflow loss, rather than their sum. A good gutter design must satisfy many criteria including durability, cheapness and ease of fixing. In this paper on gutter sizing, the primary approach is to find that gutter size and shape that maximises the ratio of water benefit to system cost. The work was motivated by field observation of evidently over-sized gutters and the absence of any published 'informed' guidance on sizing. The study entailed theoretical analysis, laboratory experimentation and some field studies. The findings are that:

(i) 'U' or trapezoidal-section gutters give the best economy,

(ii) roof area is the primary determinant of gutter size and

(iii) a 'U'-shaped or trapezoidal section gutter of width only 70 mm will be sufficient for most house roofs in the tropics.

The optimum gutter location and fixing trajectory are also explored to produce recommendations.

CONTENTS

PREFACE

This Working Paper summarises research performed over a period of 5 years by Warwick University's Development Technology Unit. Some of it is mathematically analytical or entails computer simulations, however all the more complex maths has been removed to the Appendices of the Paper. Readers who only seek the findings, rather than the route by which they were reached, are recommended to read just the Introduction and the Conclusions.

Several people have been involved in the work besides the two named authors. Inputs have come from engineering students at Warwick University, from members of Palm Foundation (Nuwara Eliya, Sri Lanka) and Uganda Rural Development & Training (Kagadi) and (under the aegis of a research contract from DFID, UK Govt) from a Research Associate at Warwick and subcontractors in Sri Lanka (LRWHF) and Uganda (ACORD). The authors wish to acknowledge financial support from DFID, the European Union and the Nuffield Foundation.

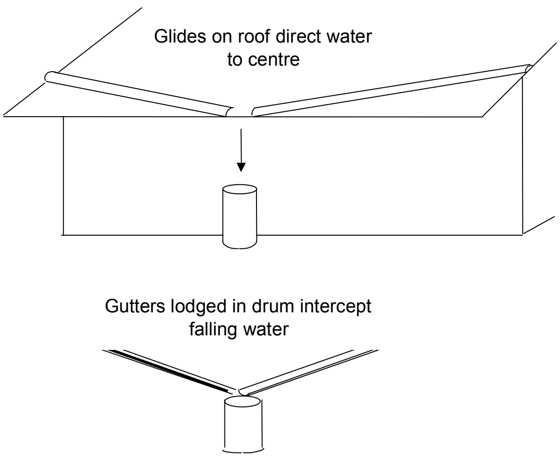
Gutter behaviour is very complex to analyse, yet RWH guttering design is not of such great economic importance that it justifies huge efforts to codify. The Paper exposes that complexity but then employs 'reasonable approximation' to reduce it to the point where simple design rules can be generated.

1 INTRODUCTION

Gutters are an almost essential but relatively cheap part of a roof-water harvesting (RWH) system. It *is* possible to collect roofwater without them by using instead glides or ground level troughs (Gould & Nissen-Petersen, 1999; Qiang & Fuxue, 1995) and some house geometries concentrate run-off from adjacent roofs into gulleys/valleys.

However the great majority of roof-water collection systems contain gutters to intercept run-off from the roof edge and convey it laterally to a downpipe leading to a tank

Figure 1.1: Alternatives to Roof-edge guttering



In temperate countries at latitudes above about 40^{o}, roof overhangs are often very small, so gutters are commonly installed to prevent rainwater running down the house walls and damaging them. In countries closer to the equator, the higher temperatures and higher sun angles make it attractive to project roofs 60cm or more beyond house walls to provide shading. Such large overhangs throw most roof run-off water clear of the walls, especially in single-storey buildings, and therefore guttering is not needed for wall protection. Any guttering that is installed must therefore be financially justified wholly by its contribution to water harvesting.

In addition to the prevalence of big roof overhangs, and sometimes of poorly aligned roof edges, the *humid* tropics are characterised by three features that impinge upon gutter design, namely:

- rainfall can be intense,
- the dry season is not long,
- household incomes are low.

Intense rainfall requires guttering of relatively high flow capacity: a design norm equivalent to rainfall intensities of say 1.5 to 2 mm per minute. The short dry season permits use of relatively small and cheap tanks: this in turn raises the fraction of RWH system cost attributable to guttering. Finally, low household incomes require low-cost RWH systems. These three factors result in the need to pay more attention to minimising guttering costs in the humid tropics than elsewhere.

Any observation of existing tropical RWH systems reveals many inadequacies in guttering (Gould & Nissen-Petersen, 1999), often resulting in the loss of over 50% of potential water yield. Gutter slopes may be inadequate or even negative; joints may leak; serious blockage by debris is common, as is twisting of gutter sections (resulting in spillage over their sides). In institutional systems, lack of clear management responsibility results in damaged or stolen guttering not being replaced, often leading to total failure of water delivery.

A 'failure' of a different kind is that both gutters and downpipes are often seriously oversized in domestic RWH systems. This may be due to the unrealistic choice of 100% as the design criterion for run-off capture. To intercept *all* run-off from the poorly aligned edge of a small house's roof may indeed require 100mm or even 150mm gutters, whereas to collect 96% of the run-off can be achieved with guttering of half the cost. Furthermore it is uncommon to find guttering economy given any attention when decisions are being made about tank location or tank height, so often gutters have to be designed for unnecessarily difficult conditions.

In some cultures it seems that aesthetics favour the selection of particularly conspicuous (large) gutters and pipes: in other cultures every attempt is made to hide them.

Finally guttering has a ‘health’ dimension, in that gutters can be a breeding ground for mosquitoes and in that gutters full of debris impart extra bacteria into the water they convey.

Ideally, guttering should be cheap to produce, efficient in capturing run-off water, easy to align and install, resistant to damage and (if not completely self-cleaning) simple to clean. The worldwide trend towards multi-storey housing increases the difficulty and even danger of installing and cleaning gutters, so that designs and attitudes adequate for RWH in rural homesteads in the past are unlikely to suffice for urban or rural housing in the future.

There has of course been some past research into guttering, but there is rather little one can refer to with respect to either the theory of its performance or to its performance in practice. Gutter manufacturers have undertaken tests of their own products and sponsored some research by others (for example into ‘downpipes for very large buildings). There are some standards (BS6367, 1983) and web pages. However very little of the published literature refers to conditions in poor tropical communities, to the specific context of roof-water collection or matches the information needs of promoters of RWH. The research reported in this Paper was therefore undertaken to extend existing guttering knowledge to better cover domestic tropical RWH, to fill gaps in that knowledge and to give certain aspects of gutter design a more scientific basis. Finally we have tried to translate our findings into ‘rules of thumb’ simple enough to be transmitted to RWH practitioners.

In the sections that follow we have concentrated on gutters proper, giving only brief attention to downpipes and avoiding any consideration of first-flush diverters or other devices sometimes incorporated into downpipes. Moreover our limited resources have led us to concentrate research upon gutter performance and sizing, rather than the equally important issues of gutter manufacture, design and durability.

The format of the paper is as follows. Sizing gutters for firstly water conveyance and then run-off interception are considered in isolation, giving some bounds on gutter size. Following this, modelling of gutter performance is conducted considering both these mechanisms acting together. Conclusions are given following analysis of the results.

2 SIZING A GUTTER TO CONVEY WATER

2.1 Capacity to convey water

Before examining the interaction of the conveyance and interception functions, we first fairly crudely examine conveyance on its own. We do this in order;

(a) to short-list configurations that are efficient at conveyance,

(b) to develop the conveyance model later to be used in combination with an interception model to produce an overall performance forecast and

(c) to be able to optimise gutters in situations where interception can be assumed to be near complete (e.g. where roofs are not corrugated and winds are generally gentle).

In this section we will consider sizing a gutter just to convey water from its entry into the gutter to the downpipe. The findings from the work are summarised in Appendix 1, which gives a more detailed explanation of the data used and analysis performed.

Sizing a gutter for conveyance involves modelling the flow within the gutter. The flow changes along the gutter, extra water coming in from the roof; so we are dealing with an example of spatially varying flow. There are difficulties with modelling this form of flow. Obtaining a full algebraic solution is not practical. So we developed instead a numerical model of this situation, and validated it via an experimental test rig. Further work yielded an asymptotic non-dimensional solution to the flow equation. [This was however only a second order solution, i.e. it was not exactly the solution of the original equation, but a very close approximation.] In this solution, the most significant term was equivalent to applying the long-established Manning formula:

$$Q = \frac{1}{n} a r_h^{2/3} \sqrt{S} \qquad \text{Equation 2.1}$$

Flow (Q) thus depends on the roughness of the gutter material (n), gutter cross-section area (a), hydraulic radius (r_h) and gutter slope (S). The roof area efficiently drainable by a gutter is proportional to Q.

However to this Manning term has to be applied a correction based on the aspect ratio of the gutter; namely the ratio of the gutter's length to its width. This correction gets smaller as the aspect ratio increases: i.e. as the gutter increases in length, the flow within it approximates more and more closely to that predicted by the Manning equation. As gutters in domestic applications nearly always have an aspect ratio higher than 100 (they are very narrow relative to their length), the correction to the Manning formula becomes sufficiently small that it can be neglected. The Manning formula alone may then be used to size gutters for water conveyance. Moreover, provided the roof area is not extremely squat (i.e. gutter length is not less than the slope-length of the roof perpendicular to the gutter), the gutter can be sized just according to the roof area, as is in section 4.7.

So we start by assuming all water falling on the roof also enters the gutter. If we now wish to minimise the system cost per litre captured, it can be shown (see Appendix 1) that a tropical gutter should be sized to match a rainfall intensity of around 2 mm/min of rainfall. (This intensity gives a gutter outlet flow of around 0.03 x A litres per second, where A is roof plan area.) As shown in Table 2.1, quite small gutters are adequate for rainwater conveyance from domestic roofs of representative areas.

Gutter trajectories where the slope varies along the gutter are also considered, leading to the conclusion that a gutter laid at a slope of α % at the outlet could be laid at $\alpha/2$ % for the first 2/3 of its length with no loss in conveyance capacity. This is covered in Section 2.3.

As we can achieve almost any flow capacity from a given gutter by making it steep enough, there is no meaning to 'optimum size' unless we first constrain the gutter slope. Moreover from just a conveyance point of view we would choose a deep gutter shape such as a square section or a deep-drawn 'U'. (Later we will find that for economy, interception requires a shallow shape.) Table 2.1 lists the roof areas that different sizes of square gutter could optimally drain at various given slopes.

As a further point of interest, it would appear that many downpipes are oversized: calculations given in A1.5. These suggest that pipes with bores around 40mm should give adequate capacity for the flowrates of interest.

Table 2.1: Optimum roof areas drainable by square gutters (considering only conveyance)

Square gutters	**Slope (%)**			
	0.5	1	2	4
Gutter width	**Optimum roof area gutter will serve (m^2)**			
33 mm	10	14	20	28
50 mm	29	42	60	85
75 mm	88	125	177	250
100 mm	190	269	380	538

Note that (sheet) material width is 3 times gutter width for this square section.

2.2 Gutter shape and conveyance

The effect of gutter shape on conveyance can be examined assuming the Manning formula for capacity is valid. Where it is, capacity is proportional to:

$$A_g^{5/3}\ p_g^{-2/3} \qquad \text{Equation 2.2}$$

Where A_g is gutter cross-sectional area and p_g is the length of the wetted perimeter of the cross-section.

Changing from square section (as in Table 2.1) to other common shapes (semi-circular, trapezoidal and vee) requires us to decide what is to be fixed. Two important strategies are: -

(i) holding the amount of material in the gutter constant – in effect we hold the gutter perimeter p_g constant. – so that gutter cost is kept approximately constant

(ii) holding constant the gutter width (its aperture and therefore its ability to intercept roof run-off).

Routine calculations then give us Table 2.2

Table 2.2: Roof areas drainable by gutters of different shapes (assuming square gutter is sized to drain 100 m^2)

Shape	square	semicircular	trapezoidal	vee (90°)
w / p_r	0.33	0.64	0.67	0.71
Strategy	**Roof area gutter drains (m^2)**			
(i) const p_r material	100	182	250	122
(ii) const *w* width	100	33	39	16

Thus on the criterion of flow capacity (and therefore roof area drainable) alone, for a given gutter *width* a square section is much superior to other shapes, while for a given gutter *cost* the square section is inferior. This switch reflects the low width-to-perimeter (w/p_r) ratio of the square shape. The trapezoidal shape is probably the best shape overall – the one used in these calculations has 'wings' set at 30° to the vertical and of the same size as the base.

2.3 Gutter trajectory (varying the gutter slope)

Hanging a gutter at a constant slope is easier than trying to follow a more complex trajectory. However this is not an efficient way of using a gutter of constant section: at rainfall intensities causing the lower (i.e. discharge) end of the gutter to overflow, the upper part of the gutter does not run full and is therefore 'oversized'. One option would be to steadily increase a gutter's *size* as one progressed from its top to its bottom end. This is not usually practical and a constant gutter section must be accepted. A second option would be to steadily increase the gutter's *slope* along its length. Here we explore the theoretical benefits of such a procedure to see if they might justify the extra complexity it introduces into gutter hanging.

At *constant slope*:

local slope is: $S(x) = S_o$, where x is distance from the top end and S_o is maximum slope,

local drop is

$y(x) = S_o\, x$ Equation 2.3

maximum drop is

$y_o = L\, S_o$ Equation 2.4

mean drop is

$y_m = L\, S_o /2$ Equation 2.5

However the most efficient profile for a gutter would be the one whereby (at the design rainfall intensity) the gutter was running just full at every point along its length. Manning's flow formula indicates that the local slope of such an 'ideal' gutter would need to be not constant but according to

$S(x) = S_o\ (x/L)^2$, where L is the gutter length.

At this *ideal slope*:-

local drop is

$y(x) = x^3\, S_o / 3\, L^2$ Equation 2.6

maximum drop is

$y_o = L\, S_o /3$ Equation 2.7

mean drop is

$y_m = L\, S_o /12$ Equation 2.8

Thus compared with constant slope, by using an ideal profile we have achieved a 6-fold reduction in the mean drop, giving thereby a probably substantial improvement in run-off interception. Alternatively we could have kept the same interception (same mean drop y_m) and have increased the maximum slope S_o by a factor of 6, thereby increasing gutter flow capacity by factor 2.45. A further option would be to commute this extra capacity into a smaller gutter size, thereby allowing use of a 29% smaller gutter (as $2.45^{-3/8} = 0.71$). These are significant benefits.

Hanging a gutter to such an ideal cubic trajectory is unlikely to become common practice. Gutters are not made to bend easily in a vertical plane. Moreover to avoid mosquito breeding we require all parts of a gutter to drain down after a storm has ended. So we avoid flat gutter sections and can contemplate changes in slope only at the joints between gutter sections. One simple compromise is the 'dual-slope' gutter whose upper end (length L_u) is laid at a low slope S_u and whose lower end (length L-L_u) is laid at full slope S_o. Using the Manning formula one can find by inspection that to minimise mean drop y_m the *best dual slope* arrangement is for

$L_u/L = 0.5$ and $S_u/S_o = 0.25$ giving a mean drop of $y_m = 0.22\ L\ S_0$

Comparing with the *uniform slope* gutter, this *best dual slope* represents, for constant mean drop, a flow capacity gain factor of 1.51 (which is significant) or a size reduction factor of 0.86 (not very significant).

In recommending dual slope to practitioners, we have to describe it in a simple and easily memorable form. It may be that

> *"Use a quarter slope for the upper half of the gutter length and full slope for the rest"*

is too complex, or requires too fine a level of slope measurement. For that reason we have also investigated a less efficient but more memorable dual-slope configuration, which uses "half slope" ($S_u = 0.5\ S_o$) and "full slope". A near-optimum and readily memorable form of this we may call *simple dual slope*

> *"Use a half slope for the first 2/3 of the gutter length and full slope for the rest"*

The numerical results of comparing *constant slope, ideal slope, best dual slope* and *simple dual-slope* gutters are given in Table 2.3. There we see that substituting *best dual-slope* or *simple dual slope* for constant-slope guttering is likely to yield significant improvement in run-off interception (via halving the mean gutter drop) and that for very long large gutters it may even be worth adopting a complex trajectory close to *ideal slope* to reduce gutter size.

Table 2.3: Comparison of gutter-slope options

Gutter slope		Max drop	Mean drop	Capacity* enhanced by
		y_o/S_oL	$y_m/2S_oL$	$(S_mL/y_o)^{0.5}$
Constant	$S=S_o$	1.00	1.00	1.00
Ideal	$S_o(x/L)^2$	0.33	0.17	2.45
Best Dual	$S=S_o/4$ & $S=S_o$	0.63	0.44	1.51
Simple Dual	$S=S_o/2$ & $S=S_o$	0.67	0.56	1.36

* Flow capacity subject to a fixed mean gutter drop y_m

2.4 Conclusions concerning conveyance alone

Because gutter length-to-width aspect ratio is so high, gutter flow can be calculated using the steady flow Manning's formula. This formula shows that for a given amount of material and given slope, square gutters convey less water than other common shapes. A trapezoidal shape is recommended.

Using such a shape, laid at a slope of 1% at its discharge end, a gutter width of 67mm (material width of 100 mm) should suffice to carry the water running off one side of a domestic roof. The gutter size giving best economy is one that overflows when rainfall intensity reaches about 2 mm/minute. A dual-slope (0.5% & 1.0%) trajectory will not compromise capacity but will approximately halve the mean drop along the gutter compared with a constant slope trajectory. its use is therefore recommended. Even bigger advantages would accrue if gutters could be laid to a $y = k x^3$ trajectory.

3 SIZING A GUTTER TO INTERCEPT RUN-OFF FROM A ROOF

We now move to consider the interception aspect of guttering, which in this section is treated in isolation, i.e. assuming that all water that is intercepted will be conveyed without spillage. In this case the only gutter dimension of interest is its 'aperture' *w*, namely the width of the opening at the top of the gutter.

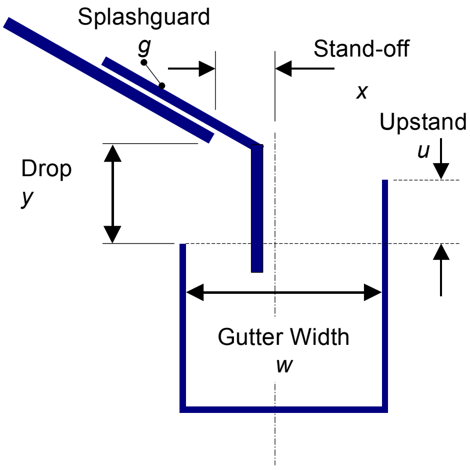
3.1 Characterising roof run-off

Water leaving a roof may collect at the edge and then fall in droplets, or it may leave in such a form as to give directed jets of water. This latter is particularly the case with corrugated roofs, where the corrugations act to concentrate the flow into channels, thereby increasing their flow velocity. By contrast, with tiled roofs the water is spread to such a fine layer that it rarely leaves the roof with a significant horizontal component to its velocity.

The performance of the gutter will vary from minute to minute, due to the ever-varying rainfall intensity. However the gutter designer needs an overall figure such as mean annual performance.

The interception performance of a gutter will depend primarily upon the dimensions labeled in Figure 3.1

Figure 3.1: Gutter Dimensions affecting Run-off Interception



These dimensions are, in declining order of influence,

i. Gutter aperture size (width *w*). The width of the opening at the top of the gutter will clearly affect its performance at collecting water leaving the roof. From the conveyance work in the previous section, we have in mind values around *w*=50 mm to *w*=100 mm; (2" to 4" gutters).

ii. Gutter drop *y*, measured from the discharge point on the roof to the top surface (level of water at overflow) of the gutter; spot values of 10mm and 100mm should cover the range of normal use.

iii. Gutter stand-off *x*, whose value is zero when the *centerline* of the gutter is directly below the edge of the roof.

iv. Gutter upstand *u* above its overflow surface. For symmetrical gutters this will be zero. For gutters with a raised outside edge this will be positive.

v. The presence or absence of a splash-guard on the roof edge directing water into the gutter: *g*=0 *or g*=1.

However besides the gutter dimensions many other factors affect interception performance, including:

i. Rainfall intensity.

ii. Wind strength.

iii. Backing – i.e. the presence or absence of a fascia board preventing wind passing across the gutter. Note that to be effective in controlling wind, such a board needs to be very close to the back of the gutter.

iv. Roof type – furrowed roofing gives a more strongly bunched and directed runoff than tile roofing.

v. Roof length *l* perpendicular to the gutter. For poor housing we might use *l*=2500 mm and *l*=3000mm, the latter being the length of a full GI sheet.

vi. Roof area draining into the gutter.

vii. Roof slope, which in the tropics typically lies in the band 15^o (S=0.26) to 30^o (S=0.5).

viii. Straightness of the roof edge (in plan) and the roof environment (e.g. overhanging trees).

It may be that a gutter sized to give sufficient flow capacity would not intercept sufficient water leaving the roof to make use of this capacity. Alternatively, sizing a gutter for interception alone would lead to an extremely wide, shallow gutter, which might lack the flow capacity to convey the water intercepted.

From observations by several fieldworkers, it would appear that except during the most intense rainfalls, water leaving the roof collects at the roof edge and then drips vertically, i.e. with negligible horizontal velocity. Such drips are strongly affected by wind around the gutter. Wind in turn is moderated by the presence of a fascia board. However it is intense rain that interests us most, because it generates such strong flows in the furrows of the roof that small jets are formed at the roof edge. These jets are more resistant to wind deviation but their velocity has a horizontal component that may carry them past the gutter's outside edge, resulting in loss through overshoot.

3.2 Gutter positioning in the absence of wind

Experimental work was conducted under laboratory conditions by two Warwick students, Boswell & Vispond, in 1997, simulating a range of rainfall intensities on corrugated roofs of varying length and slope. They found that the outward 'throw' of the run-off jets increases with roof slope, roof length and rainfall intensity, but that below certain levels of these parameters there was negligible throw.

For a representative roof (slope of 22^o), at rainfall intensities of 2 mm/min, the throw was found to be around 100mm at 100mm drop (slightly more for longer roofs). There is pulsation as the water leaves the roof, giving some variation in throw. The drop of 100mm was chosen, considering the previous work on conveyance, to be consistent with: a 1% gutter slope along a representative 10m roof edge.

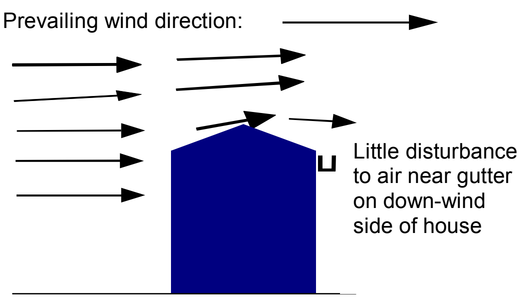
These results suggest that for likely gutter drops, to capture all the rainfall at intensities up to 2 mm/min requires a gutter stretching 100 mm outwards from the roof edge (requiring at least a *w* = 100 mm gutter aperture).

3.3 Effects of wind

Having obtained some data concerning the behaviour of water leaving the roof in calm conditions, it becomes necessary to consider the effects of wind. Wind is particularly important in the case of water dripping from the roof, as it will then be the only factor causing the water to deviate from a vertical path, and hence require a larger gutter.. However it is only wind perpendicular to the gutter that need concern us, since wind along the gutter can only generate a small end-loss.

One can argue that the wind is unlikely to be strong, and hence to have a significant effect on roof run-off, on the down-wind side of a house. Indeed wind below the roof edge on this lee side is probably an inwards-directed eddy rather than directed outwards. So we are primarily interested in wind influence on run-off on the *up*-wind side of a building. Here the effect of the wind is to reduce the outward throw, or in the case of 'dripping' run-off to impart a negative throw. This in turn requires the inside edge of the gutter to be set some distance *inside* the roof edge. Field observation suggests that provided the gutter inner edge is set about 20mm inside the roof edge, we need not worry unduly about wind effects.

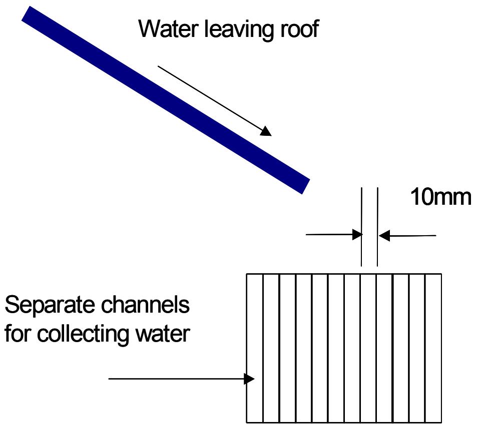
Figure 3.2: Wind flow past a building



3.4 Experimental evidence of gutter performance at intercepting run-off

Experimental work was conducted on GI roofing in Sri Lanka. Two roofs were used, one of 22^o slope and one of only 6^o slope. Both roofs were equipped with an experimental test rig that gave a distribution for the water leaving the roof to a resolution of 10mm:

Figure 3.3: Test rig for investigating roof run-off 'throw'



The distribution experiments were designed to allow the selection of an optimal gutter aperture, and give some indication of the trade-off, when increasing gutter slope, between increased flow capacity and reduced interception. To obtain an overall indication/measure of gutter performance, a number of rainfall events were sampled (more than 10).

Following collection of this data, it was observed that the rainfall leaving the 6^o roof fell within a narrow spread, and was not significantly affected by drop (y in Figure 3.1). This, suggesting the water did not acquire a significant velocity whilst on the roof, and that dispersal was caused by wind action.

An analysis similar to that described in the Section 3 on conveyance was performed, in which the gutter size was optimised to give the lowest possible system cost per unit of water collected. This is described briefly in 6A6.1.

This indicated that for a drop of 100mm, the economical gutter width w would be about 115mm, whereas for a drop of 10mm the size would be only *ca* 45mm. These figures apply to a 22^o roof angle and a 4 m long roof-slope. However the optima are not very sensitive to slope length. Both sets of results (6^o and 22^o slopes) indicated that water leaving the roof experienced a surface tension effect, causing it to curl back under the roof lip.

As the gutter itself will be at a slope, and hence the gutter drop y varies along its length, these values for respectively 100mm and 10mm drop may be taken as effectively maximum and minimum gutter sizes. Thus the optimum gutter width will fall within this range of 45mm to 105mm.

3.5 Summary of interception modelling

We have then performed two analyses, both assuming given values for roof-to-gutter drop. The first used rainfall simulations under laboratory conditions, and thereby showed the gutter aperture needed to intercept the same intensity of rainfall (2mm/min) as that already assumed for conveyance analysis. The second used data from fieldwork to optimise the aperture against a cost/unit water criterion.

Neither approach however generated an overall optimum gutter size, since neither determines the best slope of the gutter. Yet gutter slope generates the varying drop that causes the variation in performance along a gutter's length. We can however say, after placing an upper bound of 100mm on gutter drop y, that:

- the laboratory experiments place an upper bound of 120 mm on domestic gutter width,
- the fieldwork data places the optimum gutter width in the band 45 to 115 mm.

4 OPTIMISING SIZE & CONFIGURATION OF GUTTERS

4.1 Introduction to overall size optimisation

The previous sections have considered sizing gutters for interception or conveyance in isolation. This has been useful in giving some bounds to the size of gutter being considered, but cannot adequately simulate the interaction of the two in generating recommendations for gutter size and configuration.

Configuration is taken here to cover both the vertical trajectory of the gutter along the roof (the slope(s) it is hung at), and its horizontal position relative to the edge of the roof. To optimise guttering requires manipulation of these parameters in addition to gutter size. As mentioned before, changing the gutter slope will have opposite effects on the conveyance and interception performance. Both of these losses generally occur at the lower end of the gutter,[1] and so the smaller of the two will generally be subsumed in the larger when considering overall performance. Given the complexity of the interaction of these two phenomena, deriving a simple analytical model is not feasible.

So to obtain useful results, numerical simulation modelling was applied, which divided the gutter length into a series of small sections, and applied interception and conveyance criteria to determine the flow from one section to the next. Simulation was repeated over a full range of rainfall intensities, and, using information on actual tropical rainfall probabilities, the mean performance of the gutter was then calculated. Further details on the simulation are given in 6Appendix 6. Two simplifications are inherent in the model:

[1] The lower end of the gutter is the first place for interception losses to occur, and also for overspill from the gutter. With increasing rainfall intensity, overshoot will gradually occur further up the gutter, as will overflow. Overflow may also occur at the change in gradient if a dual-slope gutter has been used.

Firstly, the trajectory of the rainfall leaving the roof is assumed constant for a given rainfall intensity: pulsation effects are neglected. The losses from pulsation in one section would be opposed by gains in a separate section, so there is some justification for this.

Secondly, in the model each section as treated as having constant properties along its length. Maintaining a large number of section reduces inaccuracies arising from the use of such a numerical technique.

4.2 Gutter losses and overall losses

Gutters are part of a system that captures the rain falling on a roof and transfers it to a store. Not all the rainfall can be captured because of water-loss mechanisms. The four most important of these are:

i. Roof loss (due to splashing or absorption followed by evaporation): the latter is most prevalent during very light rainfall

ii. Gutter overshoot, due to inadequate gutter ‘aperture’, during intense rain or very strong winds: this is concentrated at the lower end of any gutter

iii. Gutter overflow during intense rain due to inadequate gutter capacity (itself a function of size, shape and slope): this normally takes place only at the lower end of each gutter

iv. Tank overflow occurring mainly when the rainfall in the say last 24-hours has been very high

These loss mechanisms interact, in that the same water cannot be lost more than once. Thus any process of calculating each loss independently and then adding them will seriously over-estimate the total loss. In the case of gutter losses (ii and iii), we should therefore calculate, for each rainfall intensity, both the overshoot fraction and an overflow fraction (assuming no overshoot), and then take the higher of the two to represent the loss due to the two mechanisms acting together. This is equivalent to the operation performed by the numerical model.

It is well established that it is uneconomic to provide a tank so large that there is never any tank overflow (loss mechanism). Indeed an economically optimum tank design is likely to give tank overflow in the range 10% to 30% of annual roof run-off. Because gutters are cheaper than tanks, when considered in isolation an 'economically optimum gutter' is likely to lose only 3% to 6% of annual run-off. A question to be answered in any analysis of optimum gutter-size is therefore: "what fraction of gutter losses may be discounted because they have already been 'counted' in tank overflow?"

A full analysis of the interaction between gutter-loss and tank overflow is very complex and also requires extensive meteorological data (concerning the correlation between intense rainfall and high daily rainfall). Ideally one would use ten years' of rainfall data sampled at 1 minute intervals. Unfortunately such data is almost unobtainable even for temperate climate sites let alone for tropical ones. We might however crudely test three propositions (approximations). In order to illustrate the discussions we will take the particular case where gutter loss is around 5% and tank overflow loss (assuming perfect gutters) would be 10%; thus apparent gutter and tank efficiencies are 0.95 and 0.9 respectively.

(Proposition A) Gutter loss is *uncorrelated* with tank overflow; so gutter optimisation can be performed without consideration of tank losses other than to multiply the calculated raw gutter loss by tank efficiency (here = 0.9) to estimate water actually lost to the consumer due to gutter losses. The total system loss would therefore be 14.5%, namely 4.5% attributed to gutters + 10% attributed to tank overflow.

(Proposition B) Gutter losses are *totally correlated* with tank overflows so that we may simply take the higher of the gutter's annual loss fraction and the tank's annual loss fraction as representing their combined effect. In our example total loss is 10% and the gutter loss, being the lower of the two, doesn't matter.

(Proposition C) Intense rain giving gutter losses *occurs on different days from* tank overflow, so that there is *negative correlation* between the two. Total losses are now simply the sum of gutter and tank losses –e.g. 5% + 10% = 15%.

Although the total losses are not much different under the three assumptions, the gutter-loss component changes considerably and it is on this component alone that we choose gutter size.

Under Proposition A, a reduction in gutter losses (by *z*% of annual rainfall) would cause a reduction of 0.9 x *z*% in overall losses, corresponding to an increase of 0.9 x *z*% in the percentage of rain captured. Under (B), gutter losses don't contribute to overall losses, and could be increased up to 10% without any loss in system performance: in this case the gutter is oversized. Under (C), any reduction in gutter losses will have an equal effect on system losses.

Gutter losses depend only on very recent rainfall – it takes on average about 10 seconds between a raindrop striking a roof and its water content reaching the discharge end of that roof's gutter. By contrast tank overflow is likely to take place during the wet season (because then the tank is usually already partly full) when at least 20 mm of rain has fallen within the last two days. During the dry season the condition changes to 'when at least 30 mm has recently fallen'. With the large tanks used in semi-arid zones, all tank overflow, like almost all rainfall, will be in the wet season. However with the smaller tanks appropriate to a humid zone, a fraction of both tank overflow and rainfall occurs in the drier season, when captured water is particularly scarce and valuable.

We therefore have a number of scenarios, given in Table 4.1:

Table 4.1: RWH Scenarios

Tank Size	Climate & Conditions	Comments
Large	Semi-arid, dry year	No overflow, so assumption (C) holds, and all gutter losses should be counted
Large	Semi-arid, wet year	Some overflow, hence assumption (A) or (B) holds, and around say 50% of gutter losses should be counted.
Small	Humid zone, wet season	Considerable overflow, so assumption (B) holds, and only an (unlikely) excess of gutter loss over tank loss should be counted.
Small	Humid zone, dryer season	Some overflow, so assumption (A) holds, and around say 90% of gutter losses should be counted.
Small	Humid zone, whole year	Mostly assumption (B) holds, but (A) is representative for some of the time, so count say 60% of losses.

For a semi-arid zone and a large tank, we worry most about 'drier than average' years so assumption (A) can be accepted and as a 'rule of thumb' 90% to 100% of gutter losses should be counted. For a humid-zone small-tank system we might count only 60% of gutter losses in realistically accounting for the impact of gutter overshoot or overflow, the other 40% being included within the usually larger tank losses.

4.3 Overspill/overshoot tradeoffs

As we have seen from Section 3, sizing for interception gives a larger optimum gutter size as the drop from the roof increases. Gutter performance would be improved if the mean drop along the gutter length could be reduced whilst maintaining the gutter's flow capacity. We therefore need to choose a trajectory that best trades off overshoot losses with overspill ones.

Other trade-offs include those involved with gutter shape. Some of these relate to the gutter performance in use, and others to manufacture. The optimum shape for interception is a flat plate, which will clearly have a useless conveyance performance. There is a difference in ease of manufacture as well. If the gutter is being made from sheet metal, each fold will require additional forming, while folding metal will be simpler than trying to produce a specified curvature.

4.4 Typical roof situation

Given the large number of possible roof configurations, an exhaustive search for optimum guttering in every case would be time-consuming. The approach taken was to consider some typical values and ranges, and see which variables or combinations of variables could be eliminated or simplified. Some parameters were fixed, namely:

Roof Material Corrugated GI roofing was chosen, for several reasons: It is a common roofing material in developing countries, and is becoming more widespread in its use. The corrugations act to concentrate the flow, increasing the velocity at which it leaves the roof. This then acts as a worst-case material; a predicted performance from corrugated guttering should be equalled or exceeded by other roof materials.

Roof Slope A slope of 22° was chosen as representative. Roof slope variation is dealt with further in Section 4.8.

4.5 Gutter shape

We may take gutter cost as being mainly dependent on the quantity of material used to produce it, which for sheet material is proportional to the perimeter p_g of the gutter cross-section. In the preceding sections it was observe that 'deep' shapes are desirable for water conveyance, wide (and therefore shallow) shapes are good for runoff interception. Here then is another conveyance *v* interception trade-off. Figure 4.1 shows the results of simulations for different gutter shapes placed at the dual slope (0.5% - 1.0%) already found to be optimum.

Figure 4.1: Overall performance of differently-shaped gutters

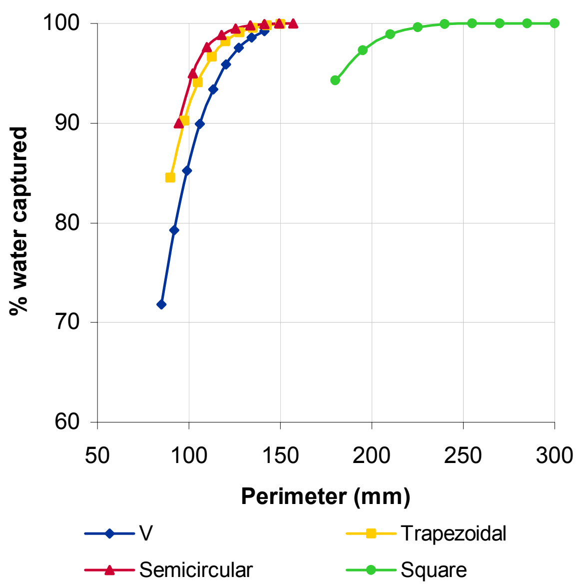


Figure 4.1 is illustrative of the situation for a range of gutter sizes. There is a distinct extra cost to using a square cross-section, because it has only a small aperture. There is however little difference between the remaining three gutter cross-sections, semi-circular, trapezoidal and vee: all have similar performance for equal material quantities. This is not surprising since both the aperture-to-perimeter ratios (w/p) of the three sections and their √area-to-perimeter ratios are similar as the following table shows.

Table 4.2: Properties of different gutter shapes

Cross-section	Ratio (√area)/p	Ratio *w/p*
Semicircular	0.40	0.64
Trapezoidal	0.41	0.67
Vee	0.35	0.71
Square	**0.33**	**0.33**

At this point the analysis simplifies, as from now on we may consider only one section, We reject the square section and choose as our norm for analysis the trapezoidal section (with wings set at 30° to the vertical). We know that substituting either semi-circular or vee shaped gutters would give very similar economic performance.

4.6 Gutter slope and trajectory

In the context just of conveyance, different gutter trajectories were discussed in Section 2.3. That discussion identified an 'ideal' but impractical trajectory of constantly-varying slope and a more practical 'simple dual-slope' trajectory that performed better than a fixed slope one. We can now examine the overall (interception plus conveyance) performance of a few promising trajectories. We have selected two candidates for the shape of the trajectory, namely uniform slope and simple dual slope., and we combine these with different mean slopes to give six rival configurations. These six, listed in Table 4.3, were tested in the overall model and their gutter efficiencies plotted against gutter width for a variety of roof lengths. (A rainfall intensity distribution representative of the humid tropics was used in combination with a trapezoidal gutter shape – see Appendix 6 – and the mean annual gutter losses were calculated.).

Table 4.3: Six gutter-slope configurations

Config No	Description	Slope of gutter (%)		
		First 2/3	Last 1/3	Mean
C1	low dual slope	0.25	0.50	0.33
C2	**medium dual slope**	**0.50**	**1.00**	**0.66**
C3	high dual slope	1.00	2.00	1.33
C4	low constant slope	0.50	0.50	0.50
C5	medium, constant slope	1.00	1.00	1.00
C6	high constant slope	2.00	2.00	2.00

The most efficient configuration (highest water capture for given roof and gutter material) was found to be C2, shown bold in the table above (Dual-slope: 0.5% & 1%). The two high slope configurations, C3 and C6 performed poorly. The remaining configurations, C1, C4 and C5, were a little inferior to C2. We therefore carry C2 forward as a recommended norm.

4.7 Roof area and shape

For a given roof area, increasing the gutter length will reduce the roof-slope length perpendicular to the gutter. The lower roof length will reduce the exit velocity of the water, but the longer gutter will develop a larger drop at its discharge end. So there are opposing effects acting on the interception performance. To test the proposition that roof area determines gutter performance independent of the shape of that area, several configurations were considered in which the aspect ratio of the roof area (gutter length / roof-slope length)was varied from 1 to 4:

Table 4.4: Various roofs used in modelling

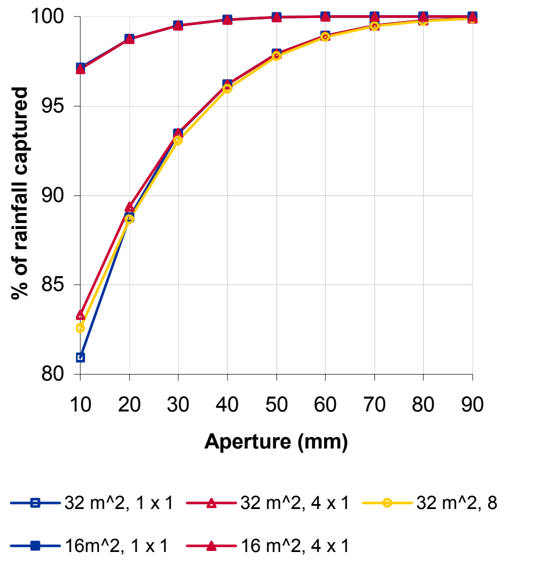
Roof-plan area* (m^2)	Gutter length (m)	Roof-slope length (m)	Aspect ratio
16	4.2	4.2	1
16	5.9	3.0	2
16	8.4	2.1	4
32	5.9	5.9	1
32	8.3	4.2	2
32	11.8	3.0	4

Note that roof plan area is smaller than roof length times depth, as the roof is not horizontal.

The performance of these configurations is shown in Figure 4.2.

As would be expected, for a given gutter size a higher fraction of yearly rainfall is captured on the smaller roof area. The more interesting feature of the efficiency plot however is the similarity in performance for the different roof shapes (aspect ratios from 1 to 4). This suggests that the effects of reduced mean drop and increased exit velocity (from using a low roof aspect ratio and hence a short gutter) largely cancel within our region of interest. Thus *we may size gutters according to the roof area*, rather than according to gutter length and roof-slope length considered separately

Figure 4.2: Gutter efficiency v roof size and shape



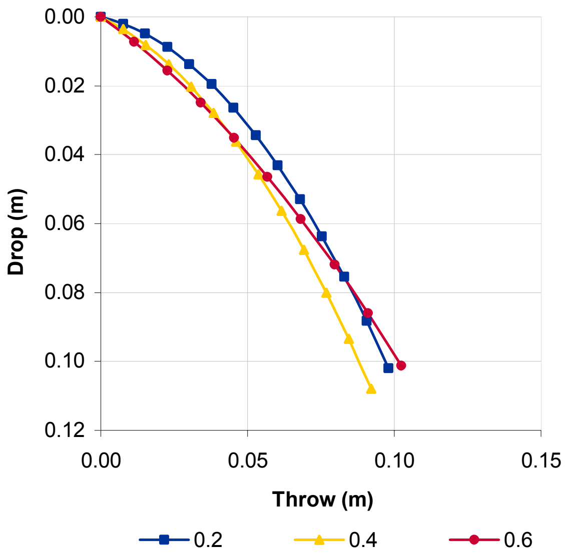
4.8 Roof slope

We might expect variation in the roof slope to alter the trajectory of the water leaving it. Analysis of the laboratory work conducted certainly shows that an increase in roof slope also increases the exit velocity of the water. Figure 4.3 shows jet trajectories leaving 4.2m-long roofs of 11°, 22° and 31° for rainfall intensities of 1 & 2 mm/min. In the region of interest (drops from 0 to 10cm), the throw from the 22 and 31° roofs are very similar. In addition to this, the throw from the 22° roof is larger initially, and there is then a cross-over point. For the lower rainfall intensity, the trajectory from the roof is greater for the lower slope.

The small variation with slope, and crossover within the area of interest indicate that gutter sizing is not very sensitive to variation in roof slope within the region of interest. For the normally-found roof slopes we may safely size gutters without taking actual slope into account.

The data from Sri Lanka and experimental work suggest that at untypically low roof slopes (less than 10°) the water leaving the roof has little velocity, so at this point there is some risk of gutter over-sizing.

Figure 4.3: Effect of roof slope on run-off trajectories



4.9 Reducing the number of variables

In Section 3.1, fourteen variables were listed as affecting 'interception', itself only one aspect of gutter performance. Such complexity is not acceptable in selecting an appropriate gutter size. However in the preceding sections we have considered many of these variables. For each we have identified a suitable 'good' value or we have shown that its influence on gutter performance is negligible. Thus we have reached the point of recommending:

i. a dual-slope gutter (with a slope in the region of 0.5% for 2/3 of its length, 1.0% for the remaining 1/3 of its length);

ii. a trapezoidal or semi-circular gutter shape;

iii. that the inside edge of the gutter be 20mm inside the roof edge;

iv. that roof *shape* can be ignored and only roof area considered;

v. that gutters correctly sized for a roof slope of 22° will also be good for common roof slopes from 15° to over 30°, thus we do not need to know roof slope;

vi. that within the humid tropics exact location and climate are not critical and we may assume a representative rainfall intensity distribution;

vii. that (common) corrugated iron roofs represent a worst case – gutters sized for CI will be adequate for other roofing types.

We are therefore now ready to explore optimum gutter size with only one variable left, namely roof area.

4.10 Economic gutter optimisation

We can define the optimum size (w=w_0) of a gutter as being the value that maximises the ratio of annual water captured Q to system cost and readily show (see Appendix 7) by standard calculus that this condition is met when:

$$S_{Q,w} = \frac{dQ/dw}{Q/w} = a \,.\, \lambda \qquad \text{Equation 4.1}$$

Where

$S_{Q,,w}$ is the sensitivity of annual gutter discharge (Q) to gutter size (w)

a is the sensitivity of gutter cost to gutter size (typically 0.6) and

λ is the fraction of total system cost caused by the gutter cost (typically 0.15 in small systems).

The sensitivity $S_{Q,,w}$ varies not only with gutter size but also with the site's rainfall intensity pattern, roof size, shape and type, gutter shape and location relative to the roof edge. However we can choose single representative values for all these variables except roof area. A_g. For each value of A_g, that leaves $S_{Q,,w}$ as only a function of gutter size w.

For very small gutters, the annual captured flow Q will be low but the sensitivity S high. As we make the gutter steadily larger, Q will rise and S will drop, until at $w = w_0$ the equation above is satisfied. In practice therefore we plot S against w and note where the locus crosses the value aλ. Under some circumstances, discussed in section 4.2, only a fraction μ of the gutter losses should be counted, since some gutter losses only serve to reduce tank overflow rather than reduce water capture. For small-tank systems in the humid tropics it was argued that we might adopt the value μ=0.6. Under these circumstances our criterion for optimum gutter size changes to:

$$S_{Q,,w} = a\lambda/\mu \qquad \text{Equation 4.2}$$

typically 0.15.

Table 4.5 shows the application of this formula to find the optimum gutter width for a roof area of 16 square meters. This is a typical area for one side (and therefore one gutter) of the roof of a small rural home.

Table 4.5: Gutter optimisation for trapezoidal gutter, humid tropics, roof area A_g=30 m^2 and $a\lambda/\mu$=0.15

Gutter width *(w)* mm	Gutter annual efficiency (%)	$S_{Q,,w}$
60	84.5	0.81
65	90.3	0.55
70	94.1	0.38
75	96.6	0.24
80	**98.2**	**0.15**
85	99.1	0.08
90	99.6	0.04
95	99.8	0.02

Note: Optimum size ($w = w_0$, when $S_{Q,,w} = a\lambda/\mu$) is shown in bold.

The process used in this table was then repeated for other roof areas to generate Table 4.5, which therefore constitutes the basic output of this Working Paper. From the figure we can also deduce that the sensitivity of ideal gutter width to roof area is around 0.35.

Moreover we may use the economic optimisation model to confirm some of the assumptions listed in Section 4.9.

In particular it was found that the variation of optimum gutter width across the three locations (climates) Lae, Kampala and Surabaya was only ±3% about the value obtained using a rainfall intensity distribution averaged over all three sites. So it seems reasonable to suggest that the results found may be generalised across the humid tropics.

5 CONCLUSIONS

Having examined by field observations, laboratory experiments and computer simulations the situation of gutter size and position for roofwater harvesting in humid tropics, the following gutter widths are recommended

Table 5.1: Recommended gutter widths for use in the humid tropics

Gutter width (mm)	Roof area in (m^2) served by 1 gutter
55	13
60	17
65	21
70	25
75	29
80	34
85	40
90	46
95	54
100	66

These figures are smaller than common guttering sizes used in the tropics and indicate there are opportunities for cost-savings. Thus even a quite large house, roof 10m x 6m, requires only a 75 mm (3”) gutter on each side.

Variations in roof slope between around 20^o and 30^o have negligible effect on gutter size requirements. When the roof slope drops to around 10^o there is a significant drop in the velocity of water leaving the roof and slightly smaller gutters might be used.

For realistic roof shapes, gutters may be sized simply on the basis of the roof plan area each serves rather than via separate consideration of gutter length and roof-slope length.

Trapezoidal, semicircular and Vee-shaped gutters give somewhat similar economic performance in intercepting and conveying roof run-off water. Choice between them can therefore be made on the basis of ease of manufacture or self-cleaning properties (Vee shapes become blocked rather frequently). Rectangular gutters however do not make efficient use of material.

Gutter mean slopes should be small, normally under 1% - and there is a small advantage in giving a gutter a dual slope such as 0.5% for the first 2/3 of its length, 1.0% for the final 1/3 to the outlet point. No part of a gutter should however be laid flat in an area subject to mosquitoes. In cases where roofs are not accurately aligned, higher slopes may be needed or the gutter direction should be decided only after the roof-edge slope has been measured.

6 BIBLIOGRAPHY

Adimula, I.A. Bryant, G.H. & Riva, C. (1998),"Rain rate distribution for tropical regions",Proc. MTT-S European Wireless Conference, Amsterdam, Netherlands, 5-8 October 1998, ISBN 086213 1510; Microwave Engineering Europe, London, UK, pp. 134-138. [ECR-1 b4]

BS 6367: 1983, "Drainage of roofs and paved areas"

Gould, J. & Nissen-Petersen, E. (1999), "Rainwater Catchment Systems for Domestic Supply", Intermediate Technology Publications.

Rees, D., Nyakaana, S. & Thomas, T.H. (2000), "Very Low Cost Roofwater Harvesting in East Africa", DTU Working Paper 55:

Zhu, Q., & Wu, F. (Oct/1995) "A lifeblood transfusion: Gansu's new rainwater catchment system", *Waterlines*, Vol: 14 (2), 5-7.

APPENDICES

APPENDIX 1 CONVEYANCE ANALYSIS

Section A1.1 will examine the economics of sizing gutters for their capacity to convey the water entering them. This leads to the suggestion that gutters should be sized for rainfall intensities of around 2 mm/min.

In section A1.2, the theory and calculations for sizing gutters are covered. This is followed by brief details of experimental work conducted (section A1.4), and sizing tables for downpipes (section A1.5).

A1.1 Describing the flow distribution

A gutter that is 'too small' will, under very heavy rainfall, spill some of the water it is trying to carry. A gutter that is too big will cost too much.

The cost of spilling water may be just the value of the water not conveyed, or it may be higher (if overflow causes damage to the building) or lower (if the overflow would have been discarded anyway due to tank overflow or inadequate tank inflow capacity).

As mentioned earlier, in the context of rainwater harvesting in the humid tropics, most buildings will have a considerable roof overhang. In this case, the cost of letting water overflow is likely to be equal to or less than the value of the water not conveyed to a tank.

From the analysis that follows, it appears that it is generally economic to make gutters large - for example so large that only 1% of the roof runoff overflows the gutter system.

In any economically viable roofwater harvesting system, the amortised value of the water collected is greater than the capital cost of the system: in the ensuing analysis, however, we will assume a 'just-economic' RWH system in which the amortised value of the water *equals* the system cost. We will also assume:

(a) The value of water, per litre, is constant, so that a 1% increase in water harvested represents a 1% increase in user benefit.

(b) There is sufficient storage that increasing gutter output by 1% will increase water available for consumption by 1% too.

Clearly both these assumptions are debatable and could be altered: however that would introduce unwanted further complexity into the basic analysis below.

The cost of guttering can vary considerably from system to system, for example, in one $10m^3$ system the guttering was 12% of the system cost, whereas in a (600) system the figure was 30% (DTU Working paper 55). In this analysis a mid-range figure of 20% will be used. Therefore, under the assumptions just made, increasing gutter size in order to collect more water is always worthwhile, providing that the fractional increase in gutter cost does not exceed 5 times (100% / 20%) the fractional increase in water harvested. The analysis below refines this statement.

A1.2 Strategy for optimising gutter size for conveyance alone

A1.2.1 Formal modelling

At some representative sites, data can be obtained for the rainfall intensity i, giving a probability distribution $p_r(i)$. To find the gutter inflow Q_i we multiply the intensity by the roof area, a runoff coefficient C_r and an interception coefficient C_i. These may be treated as a constant, Ω, for a particular roof. Q_i is a flow rate, as is the gutter capacity Q_d. The conveyance coefficient (or 'efficiency'), C_c, is the fraction of Q_i that is conveyed by the gutter to the tank.

Knowing the gutter shape, and keeping other factors such as slope constant, increasing the gutter capacity Q_d by 1% will have a certain effect on the total cost of the system. From applying the Manning formula to the flow in gutters (the validity and specifics of this is discussed in section 3.3), the gutter size will require a 0.4% increase. This can then be adjusted to take account of the change in materials and manufacturing cost of the gutter, to give a say 0.36% increase in gutter cost. [2]

As previously mentioned, the cost of the gutter as a fraction of the system cost may be taken as 20%. This means that the 1% increase in gutter capacity will result in only a 0.072% increase in the system cost.

Increasing the gutter capacity by 1% will also increase the quantity of water captured by the system, as expressed as the capture coefficient C_c.

Within the context described, it is economically sound to continue mentally increasing the gutter size until the percentage increase in C_c falls to that of the increase in gutter cost, i.e. 0.072% for each 1% increase in gutter capacity. After this point, any further increase in gutter cost will further increase the quantity of water harvested, but will not give a superior performance measured by cost per litre of water captured.

For a particular rainfall intensity probability distribution $p_r(i)$, an exceedance function $P_r(i)$ may be defined, giving the fraction of time that the rainfall intensity exceeds any threshold i.

A given gutter arrangement will have a flow capacity Q_c at which it overflows. This can be expressed as a corresponding rainfall intensity I. It is also possible to calculate the fraction of water spilled by the gutter, $sp(I)$, and hence the gutter efficiency $\eta(I)$. These performances will be for an entire year of operation, not a single rainfall event. At this point we need to use the concept of sensitivity. For two variables A and B, $S_{A,B}$ is the sensitivity of A with respect to B. This may be thought of as the percentage change in A for a 1% change in B.

The conveyance coefficient (C_c) for a particular gutter is a complicated function of the gutter's flow capacity (Q_d), the rainfall intensity distribution $P_r(i)$, and Ω (the roof area multiplied by roof run-off coefficient). As the gutter inflow is proportional to the rainfall intensity (runoff and interception coefficients being known), the exceedance curves for gutter inflow Q_i and for rainfall intensity i are therefore simply related:

$$P_{Qi}(i\Omega) = P_r(i) \qquad \text{Equation A1.1}$$

From the theory included in Appendix 2, the sensitivity of conveyance coefficient to gutter capacity is given by:

$$S_{Cc,Qd} = \frac{I}{i_{mean}} \frac{P_r(I)}{\eta(I)} \qquad \text{Equation A1.2}$$

As the gutter efficiency tends towards 1, this simplifies to:

$$S_{Cc,Qd} = \frac{I}{i_{mean}} P_r(I) \qquad \text{Equation A1.3}$$

Where i_{mean} is the mean rainfall intensity.[3]

[2] The sensitivity of gutter depth D to gutter cost C_g is typically about 1.1 for gutters of fixed proportions cut from constant thickness sheet, although it would be as low as 0.6 if gutter thickness were kept proportional to its depth. In this analysis it is taken as 1.1, so the sensitivity of gutter cost C_g to gutter depth D is 1/1.1.

[3] In the calculations performed, the efficiency of the system has been included.

We are interested in the case where $S_{Cc,Qd} = 0.07$, so we require $\frac{I}{i_{mean}} \frac{P_r(I)}{\eta(I)} = 0.07$

For high rainfall intensities, the following formula may be used for the exceedance function:

$$P_r(i) = ae^{-bi} \qquad \text{Equation A1.1}$$

Our task now is to find what gutter size gives the required value to $S_{Cc,Qd}$. [4]

Data has been obtained for actual rainfall distributions for three locations in the humid tropics, namely Uganda (Kampala), New Guinea (Lae) and Indonesia (Surabaya), and is included in Appendix 3.

This leads to the suggestion that the gutter should be sized for rainfall intensities between 80 and 120mm/hour (1.3~2mm/min).[5]

A1.3 Theory of Flow in Gutters

Moving along the gutter towards the outlet at the downpipe junction, the amount of water flowing in the gutter increases, as more water is fed into it from the roof. This is an instance of what is termed spatially varying flow. In some cases, such as corrugated roofs, the water enters the gutter in discrete jets, one jet coming from each furrow. With other roofing materials the flow may approach a continuous sheet, although in most cases it will remain discrete.

Within the context of sizing the gutter, we will deal with the situation where the flow entering the gutter is known. It is relatively simple to calculate the quantity of water falling on the roof, taking account of roof dimensions and slope.

In this case, we wish to be able to choose gutter size, cross-section, slope and material (within limits set by the situation) to give a system that will convey all the water for a selected rainfall intensity from its entry to the gutter into the downpipe. The value of 2mm/min as a sensible maximum intensity is argued for in A1.2 and Appendix 3, and the design calculations have been performed assuming all water falling on the roof is intercepted. Although this will overestimate gutter size, there are arguments for the validity of making this assumption, including system degradation over time and probability of errors in configuring the gutter.

This system should be the cheapest possible, so quantities of materials should be reduced, as should complexity of manufacture. In some cases there may be considerable flexibility in gutter size possible, particularly if the gutter is being made locally by artisans. There will be other situations however where guttering is more readily available in standard sizes, from industrial manufacturers. The local situation will dictate which is more economically attractive-if an oversized extruded PVC gutter is cheaper than a smaller locally-made gutter, then clearly the former should be used.

A1.3.1 Parameters

The following factors will influence the flow in the gutter, and should therefore be considered when designing the guttering system:

---

[4] Interestingly, European standards for gutter capacities (where damage from overflow is the main concern, not rainwater harvesting) are set extremely high, corresponding to rainfall intensities I which are exceeded for less than 1 minute per year ($P_r(I) = 2\text{x}10^{-6}$)

[5] Logically the capacity Q_f of any tank inlet filters should be made to match the gutter capacity Q_d.

A1.3.1.1 Gutter material

This will affect the frictional force opposing the flow (quantified by the Manning roughness factor). The rougher the surface, the greater the resistance the water will experience, and hence the lower the gutter capacity.

This will be examined in greater depth later in section A1.3.2.1.

A1.3.1.2 Gutter cross-section

Different shapes and sizes of gutter will have different performances in otherwise identical situations. For example, a flat, wide gutter will have a greater resistance to flow when full than a semi-circular gutter of the same cross-sectional area.

There are likely to be local limitations on dimension (for example, standard sizes if mass-produced guttering is being bought/made from piping).

There are a series of common sizes of guttering used in industrialised countries. Obviously some of these shapes are more complex than can be reasonably treated by this paper, including the standard 'K' shape. It is hoped that the shapes considered will be best suited to the developing countries context. The most commonly used shapes are: semi-circular, rectangular, v-section and trapezoidal.

Standard sizes tend to be based upon gutter aperture: the width w of the top of the gutter. Common sizes range from 50 to 100mm.

A1.3.1.3 Roof dimensions

The edge-length, slope and roof-slope length (length of the roof perpendicular to the gutter) all influence the quantity of rainfall to be conveyed by the gutter. The edge-length clearly influences what gutter length will be required; the slope and roof-slope length will, (along with the rainfall intensity) fix the flow intensity leaving the roof.

A1.3.1.4 Maximum rainfall intensity to be accommodated

This has been discussed in section A1.2 and Appendix 3. Design and sizing recommendations produced are those to give a system that can convey all the water from a 2mm/min rainfall event, using coefficients for runoff and interception of 1.

A1.3.1.5 Gutter slope

Maximum permissible gutter drop will be influenced by both the throw from the roof (considered in section 1) and the gutter attachment methods to be used, both in terms of materials required, performance of fixings, and limits set by features such as fascia board dimensions. The maximum drop limits the mean slope of the gutter. For domestic roofwater harvesting, a slope of 4% is felt to be a reasonable maximum[6].

A1.3.1.6 Gutter outlet conditions

This will have some effect on the depth profile of the gutter flow. With a gutter unblocked at its lower end, the water flowing out is accelerating in the region of the outlet, hence reducing the depth of flow there for a given rainfall intensity.

An identical effect can be found when the flow from the gutter enters a downpipe that is not running full.

[6] Such a slope would give a drop of 100mm in 2.5m of guttering. Larger drops than this would cause difficulties in mounting the gutter (from limited fascia board depth etc), and would suffer from poor interception performance.

In both cases, this "draw-down" effect is beneficial, as it reduces the maximum depth of the water in the gutter for a given rainfall intensity. Hence a gutter experiencing this effect would have a greater capacity than one that was not.

However, experimental work suggests that this effect is not sufficiently significant to be worth considering in the design procedure.

In this case, it is sufficient to ensure that the outlet will not run full and increase the depth of flow within the gutter. The sizing of downpipes is covered briefly in section A1.5.

A1.3.2 The Manning Formula

The simplest theory to use within the context of flows in gutters is the Manning formula:

$$Q = A\frac{1}{n}R^{2/3}\sqrt{S}$$

Equation A. 1.4

Where Q= flow in channel (m^3/s)

A= cross-sectional area (m^2)

v= velocity of flow in channel (m/s)

n= Manning roughness coefficient

p_w= Wetted perimeter (m)

R= Hydraulic radius (m) ($R = \frac{A}{p_w}$)

S= Slope

There are some theoretical objections to the validity of applying this to the flow in gutters. The Manning formula is derived assuming a constant flow, and that the flow has reached an equilibrium. Neither of these is the case for the flow in gutters. The flow in gutters approaches the Manning solution as the gutter becomes longer, but does not reach the flow conditions described, hence the Manning formula will give an over-estimate of the performance of a set gutter.

A more appropriate area of fluid dynamics to apply to the flow in gutters is that of spatially varying flow. This was developed by Garcia (2000) and Still (2001) (in conjunction with Lucey). The details of the theory are unlikely to be of interest to all users, and so have not been included. Further information is available on the web at: http://www.eng.warwick.ac.uk/dtu/pubs/rn/rwh/ugp014_still.pdf

The spatially varying flow work allowed the generation of flow profiles-predicted depths of flow at varying points along the gutter. The approach taken to obtain these flow profiles for gutters was to use a non-dimensional asymptotic solution to the model developed. In essence this consisted of an initial solution which is equivalent to using the Manning formula, and corrective terms to account for the difference in the flow distribution. The corrective terms are based upon an aspect ratio, that is a ratio of gutter length to a cross-section dimension (width). As this increases, the deviation from the flow predicted by the Manning formula decreases. In the case of guttering, this aspect ratio is often of the order 100~1000, and so the corrections to the Manning formula become small enough to be neglected. In this case, analytic results can readily be obtained for different gutter cross-sections, slopes etc.

A1.3.2.1 Manning roughness coefficient

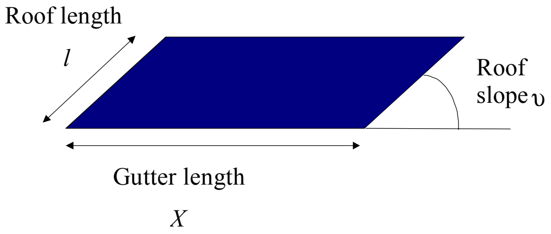
There are various possible values for this, according to the material being used in the gutter. For smooth PVC values of 0.009~0.011 are suggested. It is likely that in use some sediment will collect in the gutter, and that this will increase the frictional resistance to flow in the gutter.

One source suggests "for gutters with small slopes, where sediment may accumulate, increase [the Manning roughness coefficient] by 0.002." This would lead to a reduction in capacity of around 20%, which seems plausible. To take account of the effect of joints etc, a figure of 0.015 has been used in the sizing calculations.

A1.3.2.2 Using the Manning formula

Sizing calculations using the Manning formula are relatively simple (particularly compared to the spatially varying flow model).

Figure A. 1.1: Roof dimensions.



The assumption will be made that the gutter is being sized so that it is at the point of overflowing at the outlet end.

For this case, the flow within the gutter is expressed as a multiple of the rainfall intensity, *i* (mm/min), the gutter length, *X* (m), the roof length *l* (m), and the roof slope *θ*, as illustrated above in Figure A. 1.1. If the cross-sectional area and hydraulic radius are evaluated for the gutter running full, then the roof area at which the gutter will reach this stage can be calculated:

$$Roof\ area,\ A_{rg} = \frac{60\,000}{nl\cos\theta}\sqrt{S}R_H^{2/3}A$$
$$= \frac{60\,000}{nl\cos\theta}\sqrt{S}\,\frac{A^{5/3}}{\wp^{2/3}}$$

Some sensitivity analysis is possible from this:

$A_{rg} \propto \frac{1}{n}$: the roof area that can be covered by a gutter is inversely proportional to the manning roughness coefficient of the gutter material, so a 10% increase in the roughness value arising from gutter degradation over time, effects of joints etc will lead to a 10% decrease in the roof area the gutter can convey water from.

$A_{rg} \propto R_H^{2/3}A\,; R_H = \frac{A}{\wp}, R^{2/3}A = \frac{A^{5/3}}{\wp^{2/3}}$: generally, $A \propto Cross - section\ dimension^{\ 2}$, and $\wp \propto Cross\text{-}section\ dimension$, so

$\frac{A^{5/3}}{\wp^{2/3}} \propto \frac{(Dimension^2)^{5/3}}{Dimension^{2/3}} = Dimension^{8/3}$, so the roof area that can be covered is sensitive to a scaling in cross-section.

The hydraulic radius of cross-sections can be used as a measure of the efficiency of that shape for carrying water. The lower the hydraulic radius, the greater the wetted perimeter for a given cross-sectional area of water, and hence the poorer the performance for a given cross-sectional area and slope.

A1.4 Experimental evidence of gutter flow capacity

A test rig was to simulate water leaving a roof was constructed at Warwick University. This enabled measurement of the flow profile within the gutter for varying gutter cross-sections, slopes and simulated rainfall intensities. These profiles were compared with predicted profiles using the spatially varying flow theory and a numerical technique. A good match was found between the measured and the predicted profiles. This was taken as validation of the theory. Plots of the results may be seen on the website mentioned above.

A1.5 Downpipe Sizing

There are well-established procedures for estimating the maximum gravity-driven flow that will occur through a piping system - its so-called capacity Q_c. Unfortunately these procedures are too complex for general use in roofwater harvesting and need to be replaced by simpler ‘rules of thumb’ or design tables.

For a very long vertical pipe we can experimentally determine its capacity $Q_c = Q_m$. This will be the highest gravity flowrate we can get through a pipe of that size and is tabulated below. The term ‘very long’ indicates that secondary effects, like turbulence at entry or the presence of sharp bends, can be neglected and the flow obtainable only depends on the friction in the pipe itself. For any pipe, the *total head loss* h_L must equal the *actual drop* h_D (from the water surface at the top to the water surface at the bottom of the pipe run). For a vertical pipe this drop is the same as the pipe’s length L. For non-vertical pipes, h_L still equals h_D but $h_D = S \times L$ where S is the effective slope of the pipe. The capacity of a long sloping pipe is naturally less than for a vertical one. The table below shows the capacity of long smooth pipes of various internal diameters. Capacity is roughly proportional to the square root of the slope S. Thus at a slope of only 10% ($S = 0.1$) the capacity is only about 30% that Q_m of a vertical pipe.

Table A1.1: Ideal flow capacities of smooth downpipes running full

Internal Diameter	Vertical, S = drop/length =1.0			or Drop/length= 0.5			or Drop/length= 0.25		
	Capacity Q_m	Velocity	Equiv to h_v	Capacity Q_m	Velocity	Equiv to h_v	Capacity Q_m	Velocity	Equiv to h_v
mm	l/min	m/s	m	l/min	m/s	m	l/min	m/s	m
25	160	5.4	1.48	104	3.5	0.62	70	2.4	0.28
32	295	6.1	1.77	200	4.1	0.86	135	2.8	0.39
40	540	7.2	2.56	360	4.8	1.14	250	3.3	0.55
50	1000	8.5	3.60	680	5.8	1.66	450	3.8	0.73
63	1800	9.6	4.63	1250	6.9	2.23	840	4.5	1.00
75	2900	10.9	5.98	2000	7.5	2.85	1400	5.3	1.39

[In the table the water velocity in the pipe is expressed both as a speed (m/s) and as an equivalent head h_v. This head is the head thrown away if the kinetic energy in the pipe discharge is not recovered by suitable pipe tapering. It also equals the height through which the water would have to free fall to reach the velocity v.]

If a pipe is not ‘long’, the effect of the three factors discussed in Appendix 4 will no longer be negligible. All these factors reduce the pipe’s capacity.

For practical purposes we will be safe, even for quite short pipes, if we assume capacities of 50% of the tabulated figures.

In the analysis earlier, it was suggested that RWH systems be designed for a rainfall intensity of 2 mm/min (~120 mm/hour), which gives 200 litres/min for each 100 m^2 of roof area. Small domestic roofs are about 50 m^2 and therefore require a downpipe capacity of 100 litres/min. For this a 40 mm (internal diameter) downpipe would usually suffice and even a 32 or 25 mm pipe would often do. These are much smaller sizes than are commonly found in RWH systems. Even a large school building of 400 m^2 and an effective downpipe slope of only 0.25 does not need a pipe larger than 75 mm ID whereas 160 mm OD pipes are commonly used.

In some cases the downpipe may be replaced by a section of gutter running for the roof to the tank. In general, if this is made the same size as the gutter, and laid at a steeper slope, there should not be problems with overflow. Sudden changes in channel direction or constrictions should be avoided, as these will reduce the channel capacity, and may lead to overflow.

APPENDIX 2 RAINFALL INTENSITY FUNCTIONS

Records are available from which, for particular locations, rainfall intensity exceedence $P_r(i)$ or rainfall intensity probability density $p_r(i)$ can be derived. Rainfall intensity i is precipitation per unit time (e.g. mm per minute).

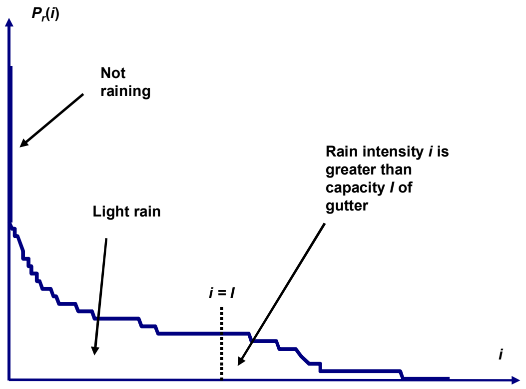
$P_r(i)$ and $p_r(i)$ are related by:

$$P_r(i) = \int_i^\infty p_r(i)di = 1 - \int_0^i p_r(i)di \text{ or } p_r(i) = -\frac{dP_r(i)}{di}.$$

We can note that:

1. The rainfall intensity is never infinite, so . $p_r(\infty)$ =0.
2. Hence the rainfall intensity can never exceed infinity $P_r(\infty) = 0$.
3. The rainfall intensity is never less than zero: "reverse raining" does not happen. $p_r(i) = 0$ for $i<0$

Figure A. 2.1: Rainfall intensity probability distribution



(There is some uncertainty about what happens at a rainfall intensity of i=0 since it only actually rains for about 1% of the time. Probability(i>0) is thus much less than Probability(i≥0) and therefore we might need to distinguish the former, $P_r(0^+)$=0.01 for example, from the latter $P_r(0^-)$=1.0. In practice this does not create a computational problem.)

The rainfall expected in unit time is $i_{mean} = \int_0^\infty i.p_r(i)di$

Substituting $p_r(i)di = -dP_r(i)$: $i_{mean} = -\int_0^\infty idP_r(i)$.

Integrating by parts gives $i_{mean} = -[i.P_r(i)]_0^\infty + \int_0^\infty P_r(i)di$

So $i_{mean} = \int_0^\infty P_r(i)di$, since $[i.P_r(i)]_0^\infty$ = 0. Equation A2.1

Thus i_{mean} is equal to the area under the whole $P_r(i)$ versus i curve.

If however, we restrict our interest to rainfall whose intensity exceeds a threshold intensity I then the quantity $e(I)$ expected in unit time will fall to:

$$e(I) = \int_I^\infty i.p_r(i)di = -\int_I^\infty idP_r(i)$$
$$= -[i.P_r(i)]_I^\infty + \int_I^\infty P_r(i)di = I.P_r(I) + \int_I^\infty P_r(i)di$$

Where $\int_I^\infty P_r(i)di$ is the area under just the tail of the $P_r(i)$ versus i curve

The *fraction* of the rain that falls at these high intensities ($i>I$) equals $e(I) / i_{mean}$.

A gutter designed for a maximum flowrate corresponding to rainfall intensity I will spill a fraction $sp(I)$ of the flow entering it. The mean spill flow, which is this spill fraction times the mean inflow, equals the integral, for all intensities greater than gutter capacity, of the difference between rain intensity and gutter capacity (expressed as equivalent intensity) weighted by the relevant probability of that intensity occurring, thus:

$$sp(I).i_{mean} = \int_I^\infty (i-I)\,p_r(i)di = -\int_I^\infty (i-I)dP_r(i)$$
$$= I\int_I^\infty dP_r(i) - \left[i.P_r(i)\right]_I^\infty + \int_I^\infty P_r(i)di$$

Therefore

$sp(I).i_{mean} = \int_I^\infty P_r(i)di$ Since the other 2 terms cancel out .

Note that the spill fraction $sp(I)$ is (much) less than the fraction ($e(I)/i_{mean}$) of rain falling faster than the gutter can carry, since even during intense rain the gutter captures some.

The gutter efficiency (fraction *not* spilled) is:

$$\eta_c(I) = 1 - sp(I) = 1 - \frac{\int_I^\infty P_r(i)di}{\int_0^\infty P_r(i)di} \qquad \text{Equation A2.2}$$

(Since $i_{mean} = \int_0^\infty P_r(i)di$)

$$\eta(I) = 1 - \frac{\textit{area under tail of distribution}}{\textit{area under whole distribution}}.$$

A2.1 Special case

In the region in the tail of the intensity exceedence curve (namely where $i>>i_{mean}$) we may often approximate $P_r(i)$ by

$$P_r(i) = ae^{-bi} \qquad \text{Equation A2.3}$$

This enables us to simplify Equation A2.2:

$$\eta(I) = 1 - \frac{\int_I^\infty ae^{-bi}di}{\int_0^\infty P_r(i)di},$$

and $\int_I^\infty ae^{-bi}di = \left[-\frac{a}{b}e^{-bi}\right]_I^\infty = \frac{1}{b}P_r(I)$

So $\eta(I) = 1 - \frac{P_r(I)}{b\int_0^\infty P_r(i)di}$ and hence the 'inefficiency' (fraction spilled) is

$$sp(I) = 1 - \eta(I) = P_r(I)/bi_{mean} \qquad \text{Equation A2.4}$$

This enables the sensitivity of efficiency $\eta(I)$ to gutter capacity (I) to be expressed as

$$S_{\eta(I),I} = \frac{\left(\frac{d\eta(I)}{dI}\right)}{\left(\frac{\eta(I)}{I}\right)} = \frac{-\left(\frac{d\,sp(I)}{dI}\right)}{\left(\frac{\eta(I)}{I}\right)} = \frac{\left(\frac{b\,P_r(I)}{b\,i_{mean}}\right)}{\left(\frac{\eta(I)}{I}\right)}$$

$$= \frac{I}{i_{mean}} \frac{P_r(I)}{\eta(I)}$$

As $\eta(I)$ approaches 1, we can say

$$S_{\eta(I),I} \approx \frac{I}{i_{mean}} P_r(I) \qquad \text{Equation A2.5}$$

Note that i_{mean} has the dimensions of rainfall intensity and b has the dimensions intensity^{-1}.

The sensitivity of *water harvested* to *gutter capacity* $S_{Qh,Qg}$ is equal to $S_{\eta(I),I}$

This quantity has been evaluated in Appendix 3 below (as a function of the rainfall intensity I that the gutters are just large enough to carry) for three locations in the humid tropics.

Setting $S_{Qh,Qg} = 0.03$, as suggested above for gutters alone, gives I values in the range 120 to 150 mm/hr.

Setting $S_{Qh,Qg} = 0.1$, as suggested above for gutters + filter, gives I in the lower range 80 to 90 mm/hr.

APPENDIX 3 CALCULATIONS FOR 3 LOCATIONS IN THE HUMID TROPICS

(At high rainfall intensities we use the approximation $P_r(i) = A\, e^{-bi}$)

Microwave telecommunications are interrupted by very heavy rainfall occurring along the path between transmitter and receiver. For this reason, detailed rainfall intensity data has been collected in many countries including some tropical ones. A telecomms engineer is primarily interested in the fraction of the time (in say minutes per average year) that rainfall is so intense that transmissions are affected. We can however use this same data to aid gutter design, provided we can convert it to express not the fraction of time but the fraction of total annual rainfall accounted for by very intense rain.

As we are primarily interested in *intense* rainfall, we can use a rainfall intensity model that is only valid at high intensities. Studies have shown that tropical rainfall derives from two main meteorological mechanisms and that the one that accounts for intense rain (say over 1 mm/minute) may be modelled by the probability density distribution

$$P_r(i) = A\, e^{-bi}$$

where i is rainfall intensity (e.g. in mm/min) and b is a constant (units of min/mm)

Moreover the annual rainfall R_a mm can also be expressed as a mean intensity

$$i_{mean} = R_a / (365 \times 24 \times 60)$$

Any particular gutter size, in the context of a particular roof and gutter drop, corresponds to a threshold I of rainfall intensity falling on that roof above which the gutter will overflow. At higher rainfall intensities $i>I$ the overflow fraction will be $(i-I)/i$.

By suitable algebraic manipulation of the quantities b, i_{mean} and I, we can estimate the fraction $sp(I)$ of annual rainfall that will spill and hence a conveyance efficiency. $\eta_c(I) = 1 - sp(I)$. Assuming for a moment that a constant fraction of water reaching the roof also reaches the gutter, annual water delivered by the gutter to the tank will be proportional to this conveyance efficiency η_c. Moreover we have shown earlier that the economically optimum size of gutter is that which gives a particular sensitivity of *water harvested* to *gutter size*. This criterion can be re-expressed as a particular value for the sensitivity of *conveyance efficiency* η_c to *design threshold rainfall intensity* I, namely to the measure $S_{\eta c,I.}$

We can also look directly at the values for spill fraction as a function of design intensity I to get an immediate feel for gutter sizing. For example we can decide that a conveyance efficiency $\eta_c = 0.95$ is appropriate and find the corresponding value for I.

If we extend this analysis to the *interception* of run-off by a gutter we must allow for important differences between 'overflow' and 'overshoot'. With conveyance a fraction $(i-I)/i$ of rainfall with intensity $i>I$ is spilled. With interception we assume ALL of the precipitation occurring at intensity $i>I$ is lost through overshoot. However for interception, unlike conveyance, the value of I corresponding to a particular gutter size varies along the gutter and cannot be defined for the gutter as a whole. One approach is to define I for midway along the gutter. If we define the fraction of annual rainfall intercepted as $\eta_i(I)$ we will find that $\eta_i(I)$ is often larger than $\eta_c(I)$. The corresponding sensitivity $S_{\eta i,I.}$ will also be different.

In the tables below we have listed values for these various measures (for representative values of I) for three tropical sites. There is some uncertainty about the notation used in the data source: $P_r(i)$ is assumed to be given in % rather than as a fraction. Otherwise the mean annual precipitation would be about 100 times higher than is likely for the locations given and one would have to assume that the probabilities given only applied to time when rain is actually falling. In the tables, I is defined as the rainfall intensity at which gutters are just full (or for interception analysis the rainfall intensity at which gutter overshoot commences).

To size a gutter we might use such conveyance criteria as

$$S_{\eta c,I} = 0.07$$

or $\eta c(I) = 0.95$

Applying all these criteria gives the range shown at the top of each page. For the three sites these criteria indicate one should size a gutter so that its flow capacity corresponds to about 1.7 mm per minute rainfall. Note that for no site would spill fraction exceed 2% if the gutter were sized for 2 mm/minute of rainfall.

The high intensity fraction for such a (I = 2 mm/min) gutter might however reach 8%. However analysis of overshoot (where loss from any small part of the gutter equals the high intensity fraction for that part) is more difficult to handle in this 'overall performance' way. So useful conveyance *v* interception comparisons cannot be made here.

Kampala, Uganda R_a = 1402 mm; b = 0.029 min/mm; i_{mean} = 0.00267 mm/min; I_{design} = 1.4 to 1.9 mm/minr					
Gutter design capacity expressed as rain intensity	I	0.67 mm/min	1.33 mm/min	2.00 mm/min	2.50 mm/min
Normalised Capacity	I/i_{mean}	246	491	737	921
Exceedence $P_r(I)$ *i.e.*	Prob $i>I$	0.00092	0.00027	0.000084	0.00003
Spill Fraction	$sp(I)$	0.197	0.062	0.019	0.008
Conveyance effic'ncy	$\eta_c(I)$ = 1-$sp(I)$	0.803	0.943	0.981	0.992
High intensity fraction	$F_{hi}(I)$ =$e(I)/i_{mean}$	0.43	0.21	0.087	0.044
Interception effic'ncy	$\eta_i(I)$ = 1-$F_{hi}(I)$	0.57	0.79	0.91	0.96
Sensitivity: harvest to gutter capacity	$S_{\eta c,I}$	0.28	0.141	0.063	0.028

Lae, PNG R_a = 1875 mm/year; b = 0.048 ; i_{mean} = 0.00357; I_{design} = 1.6 to 1.8 mm/min					
Gutter design capacity expressed as rain intensity	I	**0.67 mm/min**	**1.33 mm/min**	**2.00 mm/min**	**2.50 mm/min**
Normalised Capacity	I/i_{mean}	187	374	561	700
Exceedence $P_r(I)$ *i.e.*	Prob $i>I$	0.0021	0.00038	0.00006	0.00001
Spill Fraction	$sp(I)$	0.146	0.022	0.003	0.001
Conveyance effic'ncy	$\eta_c(I)$ = 1-$sp(I)$	0.854	0.979	0.997	0.999
High intensity fraction	$F_{hi}(I)$ =$e(I)/i_{mean}$	0.692	0.168	0.034	0.007
Interception effic'ncy	$\eta_i(I)$ = 1-$F_{hi}(I)$	0.31	0.83	0.97	0.99
Sensitivity: harvest to gutter capacity	$S_{\eta c,I}$	0.46	0.145	0.03	0.007

Surabaya, Indonesia R_a = 1445 mm/year; b = 0.040 min/mm; i_{mean} = *0.00275 mm/min*[1] I_{design} = 1.3 to 1.8 mm/min					
Gutter design capacity expressed as rain intensity	I	**0.67 mm/min**	**1.33 mm/min**	**2.00 mm/min**	**2.50 mm/min**
Normalised Capacity	I/i_{mean}	242	484	726	908
Exceedence $P_r(I)$ *i.e.*	Prob $i>I$	0.0013	0.00029	0.00006	0.00002
Spill Fraction	$sp(I)$	0.201	0.04	0.008	0.002
Conveyance effic'ncy	$\eta_c(I)$ = 1-$sp(I)$	0.799	0.96	0.992	0.998
High intensity fraction	$F_{hi}(I)$ =$e(I)/i_{mean}$	0.52	0.17	0.047	0.017
Interception effic'ncy	$\eta_i(I)$ = 1-$F_{hi}(I)$	0.48	0.83	0.95	0.98
Sensitivity: harvest to gutter capacity	$S_{\eta c,I}$	0.39	0.15	0.043	0.018

Source: Adimula A, 1998, Rain-rate Distribution for Tropical Regions,

APPENDIX 4 CAPACITY OF 'SHORT' PIPES

The capacity of a smooth vertical downpipe running full (neglecting entry, exit and "bend" losses) woud be:

Table A4.1: Flow capacity of downpipes

Pipe OD (mm)	25	32	40	50	63	75
Capacity at 90° (l/s)	2.5	5.0	9.0	16	30	48
Capacity at 45° (l/s)	2.1	4.2	7.5	13	25	37
Capacity at 35° (l/s)	1.7	3.4	6.2	11	21	32
Capacity at 15° (l/s)	1.2	2.4	4.4	8	14	23

(The angle quoted is that between a line drawn from downpipe entry to downpipe exit and the horizontal.)

However, extra head losses at pipe entry and at bends, and unrecovered velocity head at pipe exit will reduce these capacities.

'Head-loss' (*h*) is the difference in water level between the two ends of a pipe necessary to drive the specified flow along that pipe. The four main phenomena causing head-loss in a flow are:

Pipewall friction

Causing loss $h_1 \approx k_1 Q^2$ where k_1 is proportional to pipe length L and roughly proportional to $1/D^5$. D is pipe internal diameter).

Entry headloss

$h_2 = k_2 Q^2$ where k_2 is not greater than $.025/A^2$ in suitable (MKS) units. A is the pipe's internal cross-sectional area. For a rounded rather than sharp-edged entry, h_2 can be neglected.

Exit headloss

(Kinetic energy not recovered), $h_3 = .05/A^2$ or less. This maximum value (i.e. no recovery) however commonly applies.

Bends

Cause an extra headloss which depends upon their sharpness: it is common to replace the effect of a bend by an equivalent increase in pipe length and hence pipewall friction of $6D$, so $h_4 = k_4 Q^2$ where $k_4 = k_1 \times 6D/L$

Taken together, total headloss = $h_1+h_2+h_3+h_4 = (k_1+k_2+k_3+k_4)\,Q^2$

The table 5.1 above effectively only allows for h_1 and not for the other effects, and we should like to know roughly how much to reduce the capacities in that table to account for them. Unfortunately each system is different, but we can at least calculate the necessary (downwards) correction for a couple of representative cases.

Case (a) a 40mm downpipe has length 4m and drop 2m; there is also one bend.

4m of 40mm pipe has $k_1 = 49400$

1 bend is equivalent to an increase in length of 6 x .04 = .24m, $\therefore\ k_4 = 3000$

$A = .000\ 126\ \text{m}^2$ so assume a sharp entry so that $k_2 = 15900$

and a sharp exit will give $k_3 = 31700$

Thus using $k_1+k_2+k_3+k_4$ instead of k_1 in a formula of type $h_L = k\ Q^2$ requires us to multiply Q by factor $(49400/100000)^{0.5}$ = **0.70** (i.e. a 30% reduction)

Applying this factor to the tabulated flow (S = 0.5) of 360 l/min gives a flow of 250 l/min and a velocity of 3.4 m/s (equivalent to 0.56 m).

Case (b) a 63mm downpipe has length 2m and drop 0.5m; there are 2 bends

4 m of 63 mm pipe has $k_1 = 4444$

2 bends are equivalent to an extra 0.25 m of length, giving $k_4 = 278$

$k_2 = 2572$ and $k_3 = 5146$

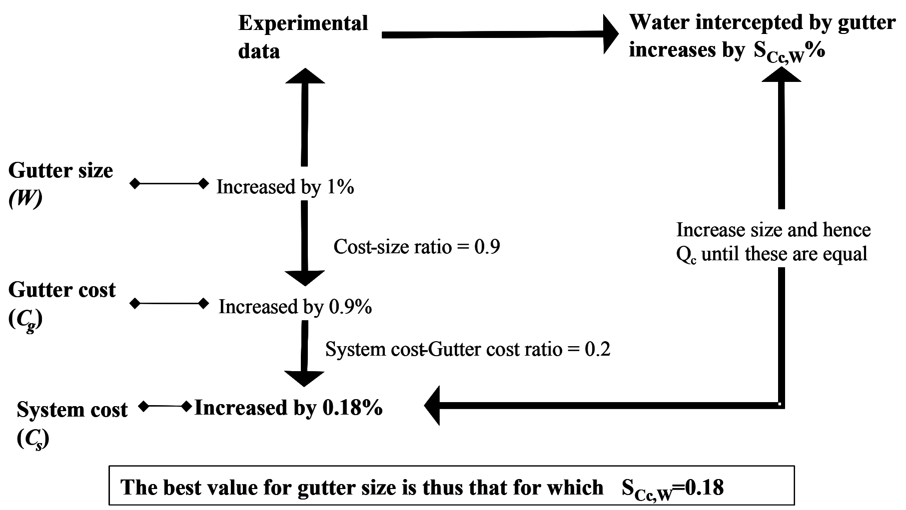
So correction factor for capacity is $(4444/12440)^{0.5}$ = **0.60** (i.e. a 40% reduction)

Applying this factor to the tabulated flow ($S = 0.25$) of 840 l/min gives a flow of 500 l/min and a velocity of 2.7 m/s (equivalent to 0.36 m)

APPENDIX 5 INTERCEPTION ANALYSIS

The basic approach used for Interception analysis is that set out in Appendix 7, namely to seek the gutter width *w* that gives the Interception-to-Width sensitivity corresponding to the maximising the system's benefit to cost ratio. This approach was applied to the interception performance of gutters with drops of respectively 10 mm and of 100 mm and a roof slope of 22^o. Unlike conveyance, interception shows a sharp rather than gentle cut-off when a critical rainfall intensity is reached. If the run-off flowrate is high enough to overshoot the gutter, then we assume that none of it is intercepted. (With conveyance the gutter capacity flow is conveyed and only any excess is over this is spilled).

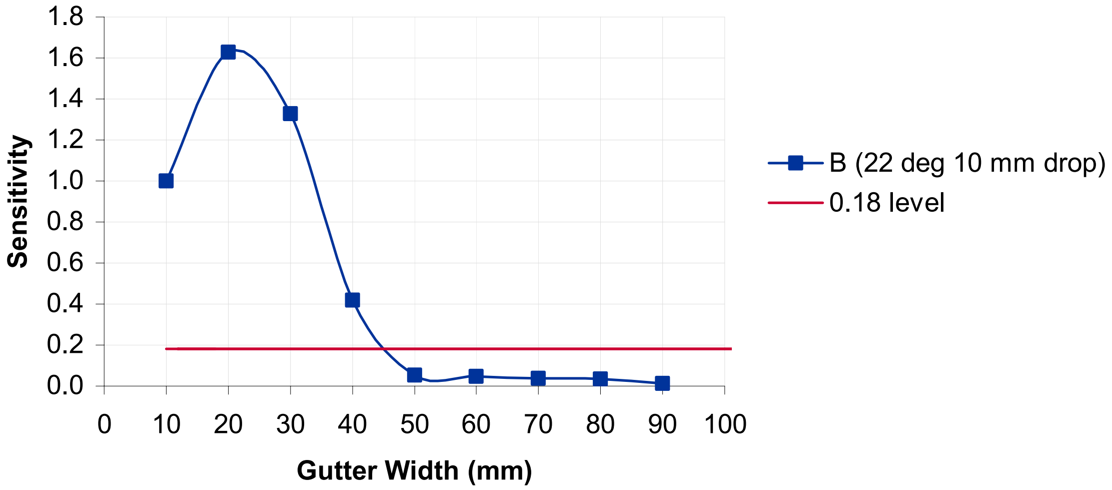
Figure A. 5.1



As can be seen, the gutter aperture is being increased until the sensitivity of water intercepted to gutter width equals that of system cost to gutter width. The diagram is slightly simpler than that used in the conveyance section, as the experimental data provides more direct information on the amount of water captured, without relying on further manipulation of rainfall data.

An example plot is given below:

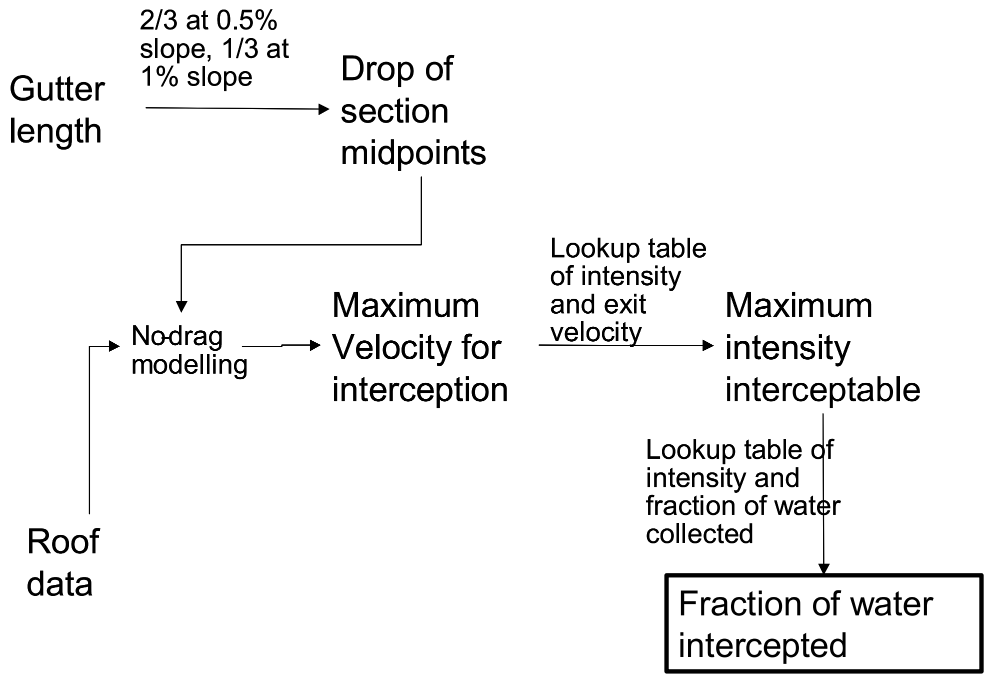
Figure A. 5.2: Sensitivity of water intercepted to gutter width (Sri Lanka data)



APPENDIX 6 COMBINED INTERCEPTION AND CONVEYANCE MODEL

A6.1 Explanation of interception data processing

As interception precedes conveyance, it was treated in isolation of the latter. Information from several sources was combined to model the interception performance of the gutter. This is summarised in the flow chart below:



The complexity in modelling interception performance arises from the slope of the gutter changing its performance along its length. To solve this problem several simplifications were required:

1. The gutter slope was specified at 0.5% for 2/3 of it's length, and 1% for the remaining 1/3.
2. The gutter is of a constant aperture along its length.
3. The jet leaving the roof was treated as following a single parabola with no spread. Although pulsation would exist, it was taken as causing benefits at some points and losses at others, and as such given no net effect.
4. The effects of wind were neglected on high rainfall intensities. To collect water from lower rainfall intensities the gutter inside lip was set 20mm inside the roof edge.

From experimentally-generated throw data, a simple calculation may be performed to predict the velocity of the water when leaving the roof edge in the absence of air resistance. By plotting this against drop, a regression may be performed to eliminate the effect of air resistance. A regression was then performed of exit velocities against rainfall intensity, and a lookup table produced of typical rainfall intensities and their respective roof exit velocities.

The gutter to be modelled was divided into a number of sections. For each section the drop at the midpoint was calculated. Given this drop and the throw (known from the gutter aperture and position relative to the roof edge), the exit velocity of water to just be caught by the gutter was calculated. The lookup table then gave the corresponding rainfall intensity at which this would occur.

The rainfall data from three locations used in the conveyance section was also processed here. For each site the exceedance probability at high rainfall intensities was plotted and a curve of the form $P(I) = Ae^{-bI}$ was fitted to the data. From this the amount of rainfall above intensity I per unit time was found, and thus the fraction of yearly rainfall falling below intensity I calculated. A second lookup table was produced giving rainfall intensity and fraction of rain falling below this intensity for each site. From this the performance of each section of the gutter was calculated, and these results aggregated to give an overall gutter performance.

A6.2 Interception and Conveyance Model

Following the production of the interception model, the discretisation approach was extended to model overall gutter performance. For a given configuration the capacity of each section of the gutter was determined. The gutter performance was modelled over a series of rainfall intensities, ranging from very high to moderate, for which latter condition the model predicted collection and conveyance of *all* water landing on the roof. For a given rainfall intensity, an algorithm determined how much (if any) run-off each section would intercept. The sum of this and water flowing from the previous section were compared to the capacity of the section. If the water entering the section were less than the capacity, all that water would be conveyed. If the water entering the section were greater than the capacity, the gutter would convey at a flow rate equal to its capacity, and the remaining water be spilled.

Using the probability data from tropical countries, it is possible to predict the quantity of rain falling in each small rainfall intensity band, and thus the overall efficiency of the gutter.

APPENDIX 7 OPTIMISATION OF SYSTEM PERFORMANCE

We wish to maximise the ratio of water captured (Q) to system cost (C) by optimising the gutter width (w): This optimum width we can denote as w_o.

$Q = f(w)$

The 'water captured' we can treat just as 'water intercepted' when we are exploring the economically optimum gutter size for run-off interception, subject to conditions such as pre-specified drop. Similarly we can treat it as 'water conveyed' (i.e. not spilled) when seeking the optimum size for conveyance. Normally however our interest is in optimising width for a system in which both interception and conveyance affect final water yield.

The system cost is the sum of tank cost (A) and gutter cost (assumed to be of form $B\,w^a$):

$C = A + Bw^a$

To maximise water captured: system cost, the following condition must be satisfied:

$$\frac{d\left(Q/C\right)}{dC} = 0 \qquad \text{giving} \qquad C\frac{dQ}{Dw} - Q\frac{dC}{dw} = 0$$

$$\therefore \frac{dQ}{dw} = \frac{Q}{C}\frac{dC}{dw} = \frac{Q}{C}aBw^{a-1} = a\left(\frac{Bw^a}{C}\right)\frac{Q}{w}$$

Rearranging this last equation yields

$$S_{Q,w} = \frac{dQ/dw}{Q/w} = a\lambda \qquad \text{Equation A. 7.1}$$

$\text{where}: \lambda = \dfrac{Bw^a}{C} = \text{fraction of total cost attributable to gutter}$ and $S_{Q,w}$ we call the 'sensitivity of water capture to gutter width'.

The economically optimum gutter width w_o is that which satisfies Equation A. 7.1.