IMPROVING THE VIABILITY OF ROOFWATER HARVESTING IN LOW-INCOME COUNTRIES

By

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CONTENTS

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DECLARATION

Several portions of this work rely on data gathered by students working under the supervision of the candidate or field-workers working with instruction from the candidate. Where this is the case, appropriate acknowledgment has been given.

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The analysis presented in this thesis is that of the candidate except where otherwise stated.

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"*Roofwater Harvesting: A Handbook for Practitioners*" with T. H. Thomas, Netherlands, IRC International Water and Sanitation Centre (2007) ISBN 9789066870574

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“Quantifying the first flush phenomenon” *12th International Rainwater Catchment Systems Conference*, New Delhi with T. H. Thomas (November 2005)

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SUMMARY

Domestic roofwater harvesting (DRWH) provides an innovative solution to meeting water needs. There is renewed interest in the technology; however problems of implementation, concerns about water quality and health, and a reputation for high cost inhibit its wider take-up. This thesis is an investigation of two of the constraints, namely cost and water quality.

The performance of systems is discussed along with the losses at various points along the system. A number of tank-building techniques are investigated and economies of scale and cost ranges are developed. Lessons learned from a range of designs are then used to develop a set of strategies whereby costs can be saved in DRWH tank construction. Two of these strategies are investigated theoretically:

- The trade-offs of investment and servicing costs inherent in reduced quality construction are analysed using net present value analysis

- Underground cylindrical tanks ure analytically modelled using a modified elastic structural theory which is verified with finite element analysis. More complex shapes are modelled using finite-element analysis alone

Several of the strategies developed are used to design tanks which are field tested. The strategies used are found to significantly reduce initial cost, however several designs require a discount rate to be introduced before they are viable long-term.

The second section deals with the quality of harvested roofwater, initially looking at the overall system, health risk pathways are traced and an overall picture of changes that take place over the course of these pathways is built-up. Treatment of incoming water from the roof is then investigated, particularly concentrating on quantifying the first-flush phenomenon which is exploited in devices that divert the first “dirty” part of a rainstorm away from a store before allowing the “clean” part into the store. The effect is found to be slower than expected and most devices are too small and reset too quickly to be very effective. A rational method for sizing first-flush device based on desired material removal is developed using a mass-balance approach.

ABBREVIATIONS AND SYMBOLS

ABBREVIATIONS

ADR	Average daily runoff
DFID	Department for International Development
DRWH	Domestic roofwater harvesting
Et	Ethiopia
EUC	Equivalent unit cost
FC	Faecal coliform, ferrocement
FE	Finite element
FF	First flush
GI	Galvanised Iron
HH	Household
HPC	Heterotrophic plate count
IRR	Internal Rate of Return
JMP	Joint monitoring programme (run by WHO and UNICEF) for monitoring progress towards water and sanitation MDGs
lcd	Litres per Capita per Day
LDPE	Low density polyethylene
MDG	Millennium Development Goal
MOSR	Modulus of subgrade reaction
NGO	Non-Governmental Organisation
NPV	Net Present Value
PSD	Particle Size Distribution
RWH	Rainwater harvesting
SL	Sri Lanka

TC	Total coliform
TNC	Too numerous to count
Ug	Uganda
UNICEF	United Nations Children’s Fund
VLC	Very-Low-Cost
WHO	World Health Organisation

SYMBOLS

A	Area; Availability factor
B	Breadth
γ	Specific weight of water
c	Correction constant
C	Cost; bacterial concentration
C_0	Initial bacterial concentration
C_1	Equivalent unit cost
E	Elastic (Young’s) modulus
E_s	Elastic modulus of soil
$f_{\sigma,c}$	Calculated stress fraction
$f_{\sigma,s}$	Simulated stress fraction
ff_d	First-flush design diversion
g	Gravitational acceleration
H	Height, depth
h	Water level
I	Rainfall intensity
k	Modulus of subgrade reaction
k_a	Accumulation constant
k_r	Reset constant, removal constant
k_w	Washoff constant

k_d	Die-off constant
$\dot{L}_d$	Load deposition rate
L	Length; Contaminant load
L_0	Initial contaminant load
L_{max}	Maximum load
M_x	bending moment
n	Manning roughness
N_θ	radial hoop force
P	Wetted perimeter
p	Pressure
p_r	Reaction pressure
p_{rs}	Reaction pressure provided by soil
p_{rt}	Reaction pressure provided by a tank wall
p_w	Water pressure
Q	Flow rate
Q_r	Runoff flow rate
Q_w	Withdrawals flow rate
Q_x	Shear force
R	Hydraulic radius; Ratio
$R_{s/c}$	Ratio of simulated to calculated
r	Radius; rainfall
S	Slope
t	Material thickness; time
t_c	Equivalent material thickness contributed by soil
t_s	Required material thickness supported by soil
t_u	Required material thickness in an unsupported tank
v	Velocity

V	Volume
$V_{in,net}$	Net volume of water entering a store
V_r	Volume of runoff over a time-step
V_S	Storage Volume
V_t	Volume at time *t*
V_{t-1}	Volume at time *t*-1
V_w	Volume of water withdrawals from a system over a time-step
δ	Deflection
μ	Poisson's ratio
σ_h	Hoop stress
σ_s	Maximum stress in an tank supported by soil
σ_u	Maximum stress in an unsupported tank

About 1.1 billion people do not have reasonable access to safe drinking water.

About 1.1 billion people have an income of less than US$1 a day.

…These are usually the same people.

1. INTRODUCTION

1.1. THE NEED

The water situation in many low-income countries is grim. More than one billion people have no access to clean drinking water and those that do, often spend considerable time walking and queuing to collect it. Many water professionals are becoming worried about the increasingly difficult problems of finding and improving water sources while some existing water sources are now becoming depleted or polluted.

1.1.1 THE SECOND WORLD WATER DECADE AND THE MILLENIUM DEVELOPMENT GOALS

We have just embarked on a new water decade dubbed the "Water for Life decade". The background for this is the a set of development goals set at the largest ever meeting of world leaders in New York in September 2000. The "Millennium Declaration" (UN Resolution 55/2), among other things commits to:

> *"Halve by 2015 . . . the proportion of people who are unable to reach or to afford safe drinking water"*
>
> [United Nations, 2000]

This formed number 10 of 18 targets identified in the "Millennium Development Goals" (MDGs) [United Nations, 2001] which inform the development policies of many governments including the UK Department for International Development [DFID, 2005] and non governmental organisations (NGOs).

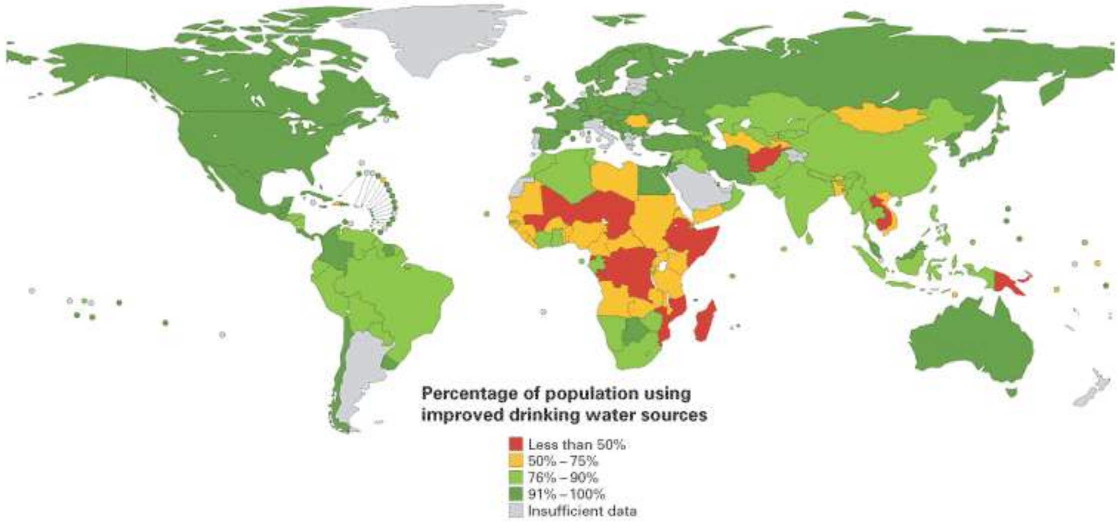
The Water for Life decade and the MDGs are of course only the most recent in a long line of international goals to target and improve water supply

Table 1.1: Water targets and resolutions

Date	Event	Goal	Outcome
1977	United Nations Water Conference, at Mar del Plata	International Decade for Clean Drinking Water (1981–1990) Water for all	30% improvement in water supply, but offset by population growth – little real reduction in the number of people with access to safe drinking water
1990	Global Consultation on Safe Water and Sanitation for the 1990's, New Delhi	Some for all rather than more for some	5% improvement, largely negated by population growth (non-coverage only decreased by 10 million despite gains of 900 million)
1990	World Summit for Children, New York	Water for all	
2000	Second World Water Forum, The Hague	By 2015 to reduce by one-half the proportion of people without sustainable access to adequate quantities of affordable and safe water By 2025 to provide water, sanitation, and hygiene for all.	Passed on to the Millennium Summit
2000	Millennium Summit	By 2015 to reduce by one-half the proportion of people without sustainable access to adequate quantities of affordable and safe water	See below

At the half-way point, according to the WHO/UNICEF joint monitoring programme on water and sanitation (JMP), water provision has shown an uneven 5% gain and is globally on-track to achieve the MDG, however there are serious regional discrepancies with East Asia, Oceania and Africa standing out as particularly vulnerable (see Figure 1.1) and rural areas continuing to be proportionally underserved. Moreover population growth over this period has meant that the number of people without access remained about the same – about 1.2 billion. As of 2006, growth also seems to be deteriorating [WHO & UNICEF, 2006] as implementation of the easier solutions are concluded leaving more challenging situations to be addressed. Due to population growth, non-sustainability and reduced growth in water provision, the projected likelihood is that Africa will actually have 47 million *more* people without adequate water supply in 2015 than in 1990 [WHO & UNICEF, 2006].

Figure 1.1: Water coverage in 2002



Source [WHO & UNICEF, 2004]

The concept of access is a contentious issue. For measuring the MDGs access has been defined as "the percentage of the population with reasonable access to an adequate supply of safe water in their dwelling or within a convenient distance of their dwelling". Again this creates questions of what "adequate supply" and "reasonable distance" are. The JMP defines the baseline used for assessing the MDGs in their *Global Water Supply and Assessment Report 2000* [WHO & UNICEF, 2000] It defines "*reasonable access* as "the availability of 20 litres per capita per day at a distance no longer than 1,000 metres".

There is some debate as to what is meant by "safe". The JMP has chosen to use "Improved water sources" as a means of measurement. "Improved" being defined in the MDGs as piped water, public tap, borehole or pump, protected well, protected spring or rainwater. It is notable that rainwater is included in this list as it is the first time this has been recognised as a high-quality water source in international guidelines.

1.1.2 THE "WATER CRISIS"

It has long been recognised that the world is entering a water crisis. While, for many this is due to underinvestment and administrational failure, the crisis also extends to deeper problems with water supply. Population pressures mount on many water sources and increased competition both inter-regionally and between sectors such as irrigation and domestic water are becoming more widespread. Groundwater has often been used as the mainstay of domestic water supply as its quality is usually reliable, however water tables are now often in overdraft and the quality of many is changing due to pollution or the emergence of minerals such a arsenic and fluorides.

The Rio Earth Summit, recognising the impending crisis acknowledged that new solutions were needed:

> *"One realistic strategy to meet present and future needs, therefore, is to develop lower-cost but adequate services that can be implemented and sustained at the community level."*
>
> Chapter 18 of Agenda 21 [United Nations, 1992]

More recently, the 2003 World Water Forum in Kyoto found that new sources are needed to meet the MDGs and other targets. Predominant among these is rainwater harvesting:

> ***"Key Issues: Safe clean water for all***
> *Increasing water use efficiency through developments in science and technology and improved demand management are essential. But these alone may not be sufficient to meet the growing demand for water in most developing regions and particularly in cities. All options to augment the available water supply* ***, including increased storage through the use of rainwater harvesting*** *, groundwater recharge and dams, need to be considered, ensuring that all those who will be affected will also benefit."*
>
> [3rd World Water Forum, 2003]

Another problem encountered, particularly with community-based water supply is reliability. In its baseline study in 2000, The JMP reported that only 70% of rural water supplies are in working order in Africa and 83% in Asia [WHO & UNICEF, 2000]. The report also notes that this is likely an overestimate due to under-reporting and places fairly undemanding criteria to define functioning[1]. Private water supplies tend to have better reliability as the maintenance necessary lies clearly within the household.

International funding for water remains very uncertain, making national and regional planning difficult. Aid for water rose through 1980s (the UN Water Decade) and early 90s but peaked in 1996, and is now back to pre-1990 levels. The trend may be reversing but current aid flows are extremely volatile, with year-on-year changes of over 50% [OECD, 2006]. Large gaps between planned investment and actual funding continue to be reported [Water Aid et al., 2003]. Indeed for many households, it is becoming clear that water will not "be provided" and self supply water provision is increasingly practiced and actively promoted by many agencies.

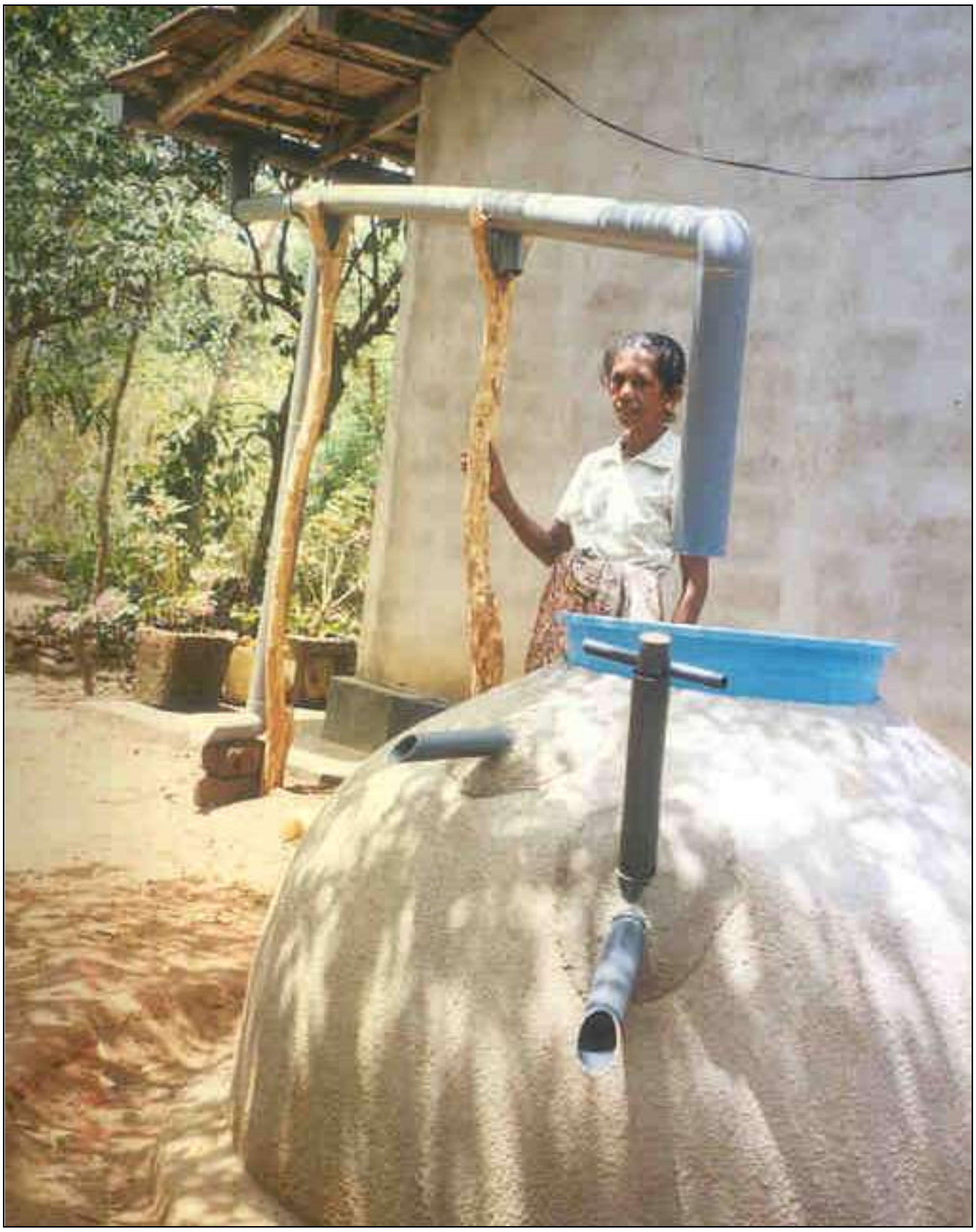
[1] The criteria for a handpump, for instance, is that it is itself functioning 70% of the time with a 2 week lead time between breakdown and repair

Of course this brings us back the main challenge – how can a household where the income is less than one dollar per day provide safe, convenient water for itself?

1.2. ROOFWATER HARVESTING

Rainwater harvesting (RWH) is an old practice in many areas of the world but has recently been rediscovered by water professionals seeking new sources to augment and in some extreme cases, replace those currently in use. The basis of the practice is that water falling on a catchment surface is sent by some conveyance to a store, from where it is used. A typical domestic roofwater harvesting (DRWH) system is pictured in Figure 1.2.

Figure 1.2: Roofwater harvesting system in a Sri Lanka



In the case of most domestic systems, the catchment will be the roof of a building – usually the dwelling itself. The roof offers a reasonably protected site that will be clear of much of the debris found at ground-level and so should provide better water quality. It should also be impervious, so almost all of the water that falls upon it can be directed into the store. The roof is, however, a limiting factor in the design of a roofwater system as the product of its area and the rainfall give the total runoff available to the user[1]. Several programmes have attempted to augment or replace existing roofs with this in mind [Gould & Nissen-Petersen, 1999].

The store, usually a tank or cistern, is a key component and acts to flatten out any fluctuation in rainfall and usage allowing water use to be, to a greater or lesser extent, independent of rainfall. Larger stores can flatten out longer-term fluctuations (such as dry seasons or even droughts) so generally, large stores are seen as desirable and DRWH systems are often described using their tank size. The cost is however also tied to the store size so, in practical terms, a balance must be struck between cost and security of supply.

Rainwater systems are decentralised and independent of topography and geology. They deliver water directly to the household, relieving the burden of water carrying, particularly from women and children. Implementation is similar to managing the installation of on-site sanitation and once systems are in-place they are owned by the householders who can then manage their own water supply.

Roofwater harvesting does have a number of limitations, however. It is not suited to being used as a stand-alone water supply solution in any but the most water-stressed situations as the increase in tank capacity necessary to bridge a long dry season can be prohibitively expensive. The storage provided by a tank does, however give households good security against short-term failure of alternative sources.

Despite its advantages, domestic roofwater harvesting remains a niche technology and, when considered at all, is usually only considered when all other options have been eliminated. The problems come under three main categories:

- high cost
- uncertain quality

[1] There are also some losses such as wetting, splashing and evaporation during and after the rainstorm that will reduce this. These losses are often simplified into a “runoff coefficient” that may also include assumed gutter losses. These are further discussed in Section 2.2

- difficulty in implementation

Niches where roofwater harvesting is particularly attractive include:

- where groundwater is either difficult to secure or has been rendered unusable by fluoride, salinity or arsenic
- where the main alternatives are surface water sources
- where management of shared point sources has proved unsuitable
- where the carriage of water is a particular burden on household members or where householders are prepared to invest in water convenience.

1.3. CONSTRAINTS TO RAINWATER HARVESTING TAKE-UP

The initial stages of this research took the form of a number of interviews with water professionals and householders. The interviews were carried out in three countries

- Uganda – a country with a fair experience of NGO-led rainwater harvesting
- Sri Lanka – a country where roofwater harvesting is used by the water authority when deemed appropriate
- Ethiopia – a country with very little experience of roofwater harvesting.

In each of these countries discussions took place with about 10 water professionals ranging from engineers from NGOs, researchers and some government officials. Parallel to this top-down view, group interviews were done with community members in these countries. Separate groups of women and men were interviewed to reduce gender bias. The discussions were necessarily wide-ranging however a number of common themes emerged.

1.3.1 COST

The cost of domestic roofwater harvesting is usually seen as high by most water supply professionals. Table 1.2a shows the costs of a number of water supply options as reported by water professionals during the interviews and Table 1.2b shows cost on 3 continents according the JMP.

Table 1.2: Per capita costs of water supply

a. based on surveys

	Uganda	Ethiopia
Town water		$70 –90
Tube well	$15–40 (~500hh) < $150 (~50hh)	> $150 (~500hh)
Spring source with gravity distribution	$15	$17-20
Rainwater Harvesting	$30	-

b. based on JMP data [WHO & UNICEF, 2000]

	Africa	Asia	Latin America
House connection	$102	$92	$144
Standpipe	$31	$64	$41
Borehole	$23	$17	$55
Dug well	$21	$22	$48
Rainwater	$49	$34	$36

Rainwater harvesting is about twice the cost of the cheapest competitor, but less expensive than deep groundwater in high-risk areas where wells could fail or sources are limited.

Part of roofwater harvesting's reputation as a high cost option is caused by the high expectations of water professionals themselves. Providers tend to think in terms of complete solutions, i.e. all water needs should be met by one source. In low-income countries this is rarely the case and householders tend to use three or four sources depending on need and availability [Ariyabandu, 2001b]. In this context large storage tanks are unnecessary and costs can fall appreciably. Roofwater harvesting suffers from strong diseconomies of scale (see Section 2.4) in terms of supplying water needs, a small (say 1,000 litre) tank may supply 70% of a households water needs over the year (mainly in the wet season) whereas a tank 5 times the size will supply 90%, only a 20% improvement. This is because water is drawn and replenished more often with a small system whereas a large one may only fill once or twice a year.

1.3.2 WATER QUALITY

Water professionals, particularly, are concerned about water quality. The overall attitude is that roofwater is an unprotected source and is liable to contamination. This is especially true of those with little experience of DRWH. Monitoring programmes where indicator bacteria are found, sometimes in large numbers, do little to dispel this unease. Little appears to be known about how, or what kinds of contaminants can find their way into a rainwater tank or what happens to them once inside. More information is needed on the effects of various interventions to protect water from being contaminated, partly to reassure professionals that roofwater can be considered a protected (or at least, risk-reduced) source and partly as several often used interventions may not be effective with results smeared by other processes in the system.

The attitude of users is much more cavalier. Experienced users appear happy to use roofwater for drinking and cooking and express few concerns about the water quality. New users are more cautious and tend to use rainwater for uses requiring a lower quality such as clothes washing but slowly move towards higher-quality uses over a couple of seasons. The use of inlet treatment is sporadic with many users downgrading their inlet treatment by, for example, using a courser mesh on filters so that it becomes less dirty. Overall users report few, if any, health problems with use of roofwater.

1.3.3 MANAGEMENT ISSUES

Roofwater harvesting is fundamentally different from most water supply options. These differences have profound effects on the management and implementation of any project involving roofwater harvesting:

- It is based on a finite volume of water that can be depleted if not well managed, making it a poor candidate for community supply unless strong measures are taken to prevent overuse

- It is strongly seasonal in nature meaning that there must also be another water source available. This source (or sources) must be able to cope with the demands of households using roofwater harvesting, especially as the largest demand will be in dry periods. It does not, however, have to be as high a quality

- *Domestic* roofwater harvesting requires a large number of small civil works rather than the large-centralised works of most water projects, requiring different approaches to management

- The cash flow of roofwater harvesting systems is that of a large initial cost with extremely small maintenance charges. This is in contrast with most water supply where maintenance is a large part of the overall costs. Most projects are costed based on donor funded initial works with users paying for upkeep – this paradigm is often unsuited to DRWH.

1.4. AIMS AND OBJECTIVES OF THIS RESEARCH

The overall aim of this research is to improve the viability of roofwater harvesting as an alternative technique for domestic water provision. More specifically, the research aims to address several of the constraints discussed in Section 1.3, particularly the broad themes of cost and water quality – the management issues cited above are beyond *technical* solutions and require changes of attitude and software solutions that are beyond the scope of a simple engineering study.

The specific objectives of the research are to:

- Reduce the cost of the storage tank and thereby reduce the overall system cost by investigating current best practice, extending lessons learned to new designs and analysing how successful the strategies used were.

- Reduce the uncertainties about water quality by researching water quality of roofwater systems and quantify the effects of interventions to improve the quality of the water being fed into the tank. An attempt is also made to produce a method whereby first-flush diversion can be designed along rational grounds rather than empirical best practice or simple guesswork.

1.5. STRUCTURE OF THE THESIS

This thesis is an investigation of two of the constraints listed in Section 1.3 above – cost and water quality.

- Chapter 2 discusses the performance and limitations of domestic roofwater harvesting systems, particularly with regards to water delivery. The chapter focuses on measures of performance, the limiting factors controlling the runoff from the roof into the tank inlet, trade-offs between tank size and performance and strategies employed to mitigate routine water shortages. The mass-balance method presented in this chapter is used throughout the thesis to make economic judgements of the value of storage types and sizes and to quantify the effects of inlet interventions on water delivery.

Part One deals with system cost, specifically tank cost as tanks are usually the bulk of the cost of a DRWH system in low-income countries ranging from 60% to 90% of the overall system cost.

- Chapter 3 deals with existing practice, particularly with regard to tank design. In this section existing designs and techniques are discussed and classified. Typical cost-volume relationships are developed that will be used in later chapters

- Chapter 4 analyses designs and formulates a number of strategies to reduce the cost of DRWH systems

- Chapter 5 is a financial analysis of one of these strategies – quality reduction. The compromises involved with this strategy on the short, medium and long-term economic performance of the system are determined using net-present-value analysis. The chapter uses experience gained from an existing reduced-quality design – the tarpaulin tank – to develop an overall picture of tank economics in terms of initial and ongoing costs

- Chapter 6 is a structural analysis of another strategy – underground construction. A theory is developed from an analytical treatment of cylindrical tanks and foundation design to calculate the contribution that can be expected from soil in cylindrical tanks. Dome-bottomed tanks are also considered through finite-element analysis

- Chapter 7 is a report of the design process leading up to the development of new designs for water tanks, a description of the designs developed and an initial analysis of their effectiveness

- Chapter 8 is an analysis of the performance of these new designs during field trials and final conclusions as to their effectiveness and the effectiveness of the strategies used

- Chapter 9 describes the “roofwater harvesting ladder” – a tool developed as part of the work that could aid in the participative selection of DRWH systems by communities

The second section deals with the quality of harvested roofwater, particularly investigating some of the processes by which the quality of the water can be protected, maintained and enhanced with a view to making the water at the outlet suitable for drinking with little or no further treatment.

- Chapter 10 takes the form of a literature review exploring the health aspects of roofwater with particular emphasis on water quality and the changes that take place over time and in different parts of the system. Consideration is also given to mosquito breeding

- Chapter 11 describes water quality interventions that can protect or improve the quality of roofwater, particularly that which is used for drinking

- Chapter 12 describes first flush systems and determines the general characteristics of the effect based on theory derived from stormwater drainage practice and a number of field measurements. The field measurements also allow the parameters that affect scale of the effect to be determined.

- Chapter 13 describes the outcome of applying a mass balance to determine how first-flush diversion will affect the overall water quality and water delivery performance of a DRWH system in use. A rational method for sizing first-flush device based on desired material removal is also developed.

- Chapter 14 summarises the main conclusions from the work and makes recommendations for further research

2. PERFORMANCE AND LIMITATIONS OF ROOFWATER HARVESTING

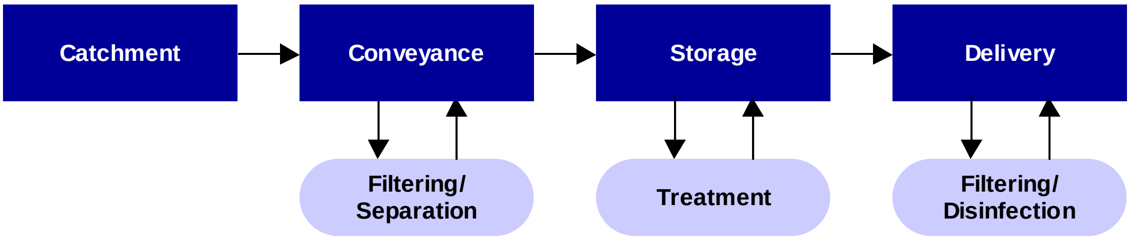
An understanding of the components of a roofwater harvesting system and their contributions to performance is crucial to the understanding of all physical and most economical phenomena related to roofwater harvesting. This chapter forms an introduction to the physical components of a roofwater harvesting system and, particularly how they form into a chain of processes each with its own losses and characteristics. Some attention is given to the design characteristics that affect these losses for gutters and roofs, while tank design forms the mainstay of Part One and Water treatment systems Part Two.

A mass-balance model incorporating the component process train is then described which will be used for a number of analyses throughout the Thesis. A number of measurement indicators are also developed. The model and indicators are then used to show the generalised performance of DRWH systems in various climates and under different usage strategies.

2.1. COMPONENT OVERVIEW

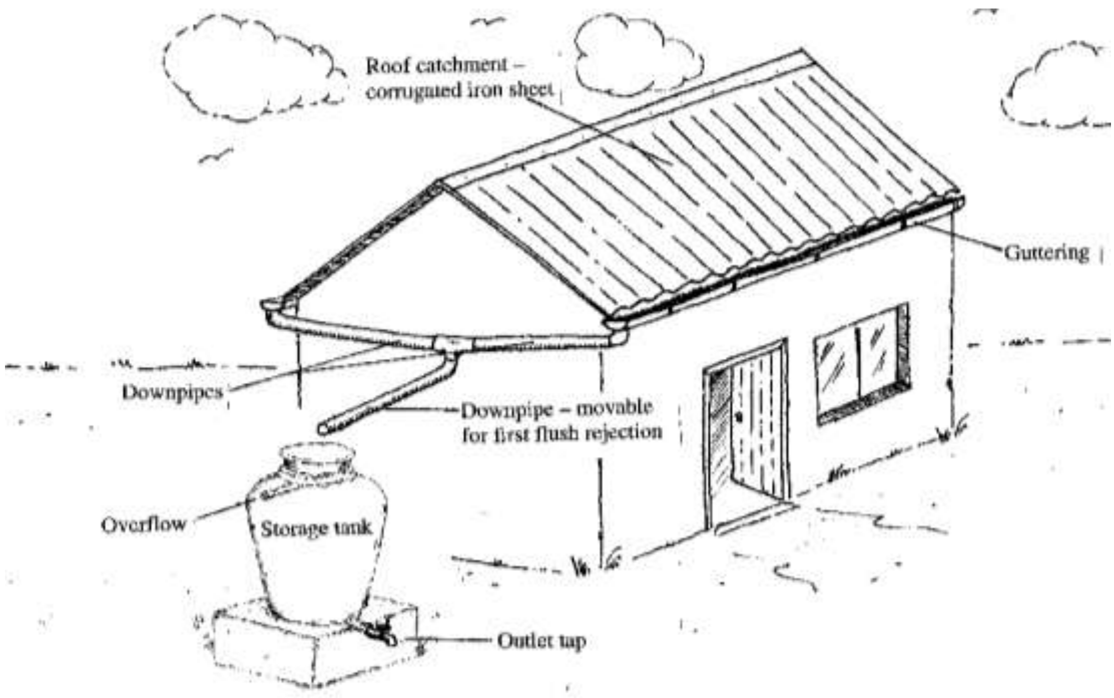
Rainwater harvesting systems can roughly be broken down into 4 primary processes and 3 treatment processes. These are outlined in Figure 2.1:

Figure 2.1: Process diagram of domestic rainwater harvesting systems



There is a considerable range in complexity of domestic rainwater harvesting systems, from simple opportunistic practice where the catchment may be a tree, the conveyance a banana leaf and the storage and delivery an earthenware pot, to highly sophisticated systems with automatic treatment at each stage of the process, electronic monitoring and dual reticulation systems.

Figure 2.2: Typical very low cost roofwater harvesting system in a low-income country



A typical "very low cost" (VLC) rainwater harvesting system in low-income countries is illustrated in Figure 2.2. The catchment is a roof, usually made of an impervious material such as corrugated galvanised iron sheet, the conveyance is by guttering and downpipe, the storage is a tank, and delivery is by a tap connected to the tank. Installed treatment includes a manual "first flush" system and a before-tank filter. There are also a number of processes that occur within the tank itself such as settlement, floatation and pathogen die-off. Finally, the household may employ some technique of post-storage disinfection such as chlorination, solar disinfection or a candle filter.

2.2. CATCHMENTS

The catchment is the initial limitation on the performance of a rainwater harvesting system. It determines the quantity and to some extent the quality of water that enters the tank.

2.2.1 RUNOFF QUANTITY

With ground-based catchments, losses can be through infiltration and evapotranspiration. These losses are usually collected together into a "runoff coefficient" which is the fraction of the water falling on a catchment that is delivered to the storage inlet. Small ground catchments have fairly constant runoff coefficients.

The loss from impermeable roof catchments is also often also expressed as a runoff coefficient; however the losses from a roof cannot be through the same mechanisms as those for ground catchments as there is (usually) no plant matter to transpire or permeable material and water table for the rain to infiltrate into. The losses from a roof fall into four categories.

- Initial wetting of the surface that must occur before runoff can take place - this water will ultimately be evaporated
- Some roof surfaces such as tile or asbestos sheet will continue to absorb some water for a time after runoff takes place until they are saturated – again, this will eventually evaporate
- Water can splash off the sides of the roof and be lost
- Some evaporation of surface water will occur during the storm – this may be significant with very low rainfall intensities but will be very small at the intensities of found with tropical rainfall

Runoff coefficients quoted for RWH systems actually expresses the *average* loss fraction for the roof surface over a year and may also include losses from the delivery system which are discussed in Section 2.3.1.

Splashing can be estimated by considering the distance that a raindrop can rebound when it impacts with a hard surface. Typical raindrop size is just over 2mm across a wide range of rainfall intensities [Hudson, 1993]. At these sizes terminal velocity is 6-7 m/s. Droplets of this size, travelling in this range of velocity rarely splash more than 200mm on impact on a smooth, hard surface such as a roof and 70% fall before reaching 100mm from the impact site [Allen, 1988; Yang et al., 1991; Pietravalle et al., 2001]. On larger roofs therefore, splash effects are negligible, however on smaller (>20m^2) roofs, runoff can be reduced by 10% or more by splash.

Evaporation, primarily of water absorbed into the fabric of the roof and accumulated debris is the presumed mechanism for losses in a study by Ragab et. al. [2003] which found runoff coefficients ranging from 0.38 to 1.2 on 6 tiled roofs in the UK. Other studies that have measured runoff coefficients for roofs in the UK and the USA have also resulted in a wide range of coefficients between 0.5-1.5. Coefficients of greater than unity have been attributed to prevailing winds, local rain-shadowing and equipment difficulties, however their existence highlights the uncertainty of predicting runoff from such a small, usually inclined catchment.

The losses also change over the course of the rainstorm. All studies show an initial loss of between 0.5 and 1mm before any runoff is observed and some studies show changes in runoff over the course of the storm, e.g. Hollis and Ovenden [1988] who noted storms of less that 5mm show an average runoff coefficient of 0.57 but larger storms exhibit an increased coefficient of 0.90. Pitt [1987] suggests a sliding scale of runoff coefficient based upon storm size with coefficients for pitched roofs ranging from 0.25 for the first millimetre rising to 0.75 after 3mm and 0.99 after 50mm.

As there is considerable uncertainty and the overall result of single (i.e. constant) coefficients is simply to change the apparent roof size, for the models in this thesis the usual runoff coefficient is taken as unity with a 0.5mm initial loss assumed to occur at each rainfall event.

2.2.2 RUNOFF QUALITY

Runoff quality also varies by catchment type. Ground catchments are prone to contamination from many sources including human and animal faecal matter, rotting vegetation and the soil itself. Higher quality water for drinking should be caught from a surface that is less easily contaminated. This usually comes in the form of the roof of the building but can be a separate structure. GI sheet roofs fare best due to their relative smoothness [Fujioka, 1993] and the sterilising effect of the metal roof heating under the sun [Vasudevan et al., 2001a]. This is further discussed in Chapter 10

2.2.3 TYPES OF CATCHMENT

ROOFS

Roofs are the most popular catchment for water for domestic purposes. An impermeable roof will yield a high runoff of good quality water that can be used for all household purposes. Roof types are detailed in Table 2.2.1.

Table 2.2.1: Characteristics of roof types

Type	Typical runoff coefficient	Notes
GI Sheets	>0.9	• Excellent quality water. Surface is smooth and high temperatures help to sterilise bacteria
Tile (glazed)	0.6 – 0.9	• Good quality water from glazed tiles. • Unglazed can harbour vegetation • Contamination can exist in tile joins
Asbestos Sheets	0.8 – 0.9	• New sheets give good quality water • No evidence of carcinogenic effects by ingestion [Campbell, 1993] • Slightly porous so reduced runoff coefficient and older roofs harbour moulds and even moss
Organic (Thatch, Cadjan)	0.2	• Poor quality water (>200 FC/100ml) • High colouration due to dissolved organic material which does not settle

The poor performance of organic roofs would seem to preclude them from use for rainwater harvesting systems, however organic roofs have been employed with varying levels of success. The water is usually used for secondary purposes but can is sometimes used as drinking water in particularly desperate circumstances [Pacey & Cullis, 1986]. Various treatments for thatch roofs have been tried such as covering with polythene sheeting, however experiments as part of the DFID project showed that the sheeting tends to degrade in the sunlight quickly and can only be used for a single season. It can also block smoke leaving the dwelling and retain moisture in the roof resulting in the organic material itself degrading [DTU, 2002]. A novel solution has been tried in Ethiopia where a foldaway roof was constructed [Nega & Kimeu, 2002] Problems remain, however as it relies on user intervention to fold and unfold the roof at height and when it is raining. A more practical solution may be the use of auxiliary catchments such as those used in the ALDP project in Botswana [Gould & Nissen-Petersen, 1999]. These can also be simple polythene structures [Ariyabandu, 2001c]. The problem of UV degradation vs. user intervention remains but the latter is now at a more sensible height.

PAVED GROUND CATCHMENTS

A bare-ground catchment has a much lower runoff coefficient than a hard roof (in the region of 0.1 – 0.3), however they are usually much larger and so can yield a high overall runoff. Artificial ground level catchment surfaces may yield a higher runoff with paved surfaces having a coefficient on the area of 0.6 – 0.7 and so courtyard runoff is often collected [Li & Liang, 1995] and stabilised threshing floors are also employed [Pacey & Cullis, 1986]. The water from these catchments is not usually of high quality, however and so can only be used for secondary purposes such as watering livestock or gardening.

2.3. GUTTERS

The water from the roof must be conveyed to the store in some way, this is usually by way of a system of guttering. Other systems such as roof slides [Waller, 1982] or ground level drains [Zhang et al., 1995] can be used but are less popular for rainwater harvesting purposes as they either spill water or allow ground-borne contamination into the conveyance system.

The gutters define the upper limit of rainfall *intensity* that can be conveyed to the storage as they are flow limited by friction and the water level will rise with flow-rate until the gutter overtops. This overtopping height is critical. If it is made too low by employing a too small gutter, significant losses may occur, yet if the gutter is made too large in an attempt to catch and convey every last drop of water, costs may become untenable.

Gutters in low-income countries can often be the weak link in the rainwater harvesting system [Mwami, 1995] and installations can be seen with gutters coming away from their mountings, leaking at joints or even sloping the wrong way. Beyond the mere functional failure, poor guttering can also be a health hazard if it allows water to remain in the gutter and become a breeding ground for mosquitoes [Montgomery & Ritchie, 2002]. This situation is exacerbated by the common NGO practice of supplying a free water store but insisting that beneficiaries install the guttering. As a general rule, the cost of the gutters is not considered important as the storage tends to dominate the system cost, however, as smaller and cheaper stores are used, the storage cost becomes less dominant and gutters can even demand half of the total investment.

2.3.1 PERFORMANCE OF GUTTERS

The two main criteria for guttering are that it *catch* the water from the roof and then *convey* it to the tank. On the surface this seems simple enough, however the relative complexity of achieving this simple aim often confounds, resulting either in poor designs that fail to deliver water to the tank or overly conservative designs with a high cost.

WATER CONVEYANCE

The flow performance of a gutter varies along its length resulting in a spatially varying flow [Still & Thomas, 2002], however for a long gutter it can be approximated by the Manning formula:

$$Q = Av = A\frac{1}{n}R^{\frac{2}{3}}S^{\frac{1}{2}} \qquad \text{Equation 2.1}$$

Where: Q is the flow in channel (m^3/s); A is the cross-sectional area (m^2); v is the velocity of flow in channel (m/s); n is the Manning roughness coefficient (usually between 0.01 and 0.15 for gutters); P is the Wetted perimeter (m); R is the Hydraulic radius (m) ($R = A/P$); P is wetted perimeter (m); S is the Slope

Using this formula an idea of the actual size of gutter needed can be developed for any gutter profile. The critical point is at the end of the gutter just before the downpipe where the entire volume of water from the roof will be flowing in the gutter. The flow rate at this point is the product of the rainfall intensity and the effective roof area. Rainfall intensity can be an important limit in tropical areas as rain tends to fall in a relatively few, heavy storms. In work relating to attenuation of microwave transmission, Adimula et. al. [1998] found that in three tropical locations, rainfall with an intensity of more than 2 mm/min accounted for about 10% of the rainfall. Still and Thomas [2002], presented analysis based on this data that considers the overspill resulting from flow that is above the maximum gutter flow and concluded that gutters should be sized for a rainfall intensity of 2mm/min. The analysis considers that gutter losses will only ever be a fraction of the water falling on the roof (assuming competent design and installation) as the gutter will always convey water up to its design flow, so despite the fraction of rainfall that occurs above 2 mm/min, actual annual water spillage will be in less than 2%. The gutter overflow limit is also significant to the design of the rest of the system as it forms the maximum flow *any* part of the system after the roof will be required to accept.

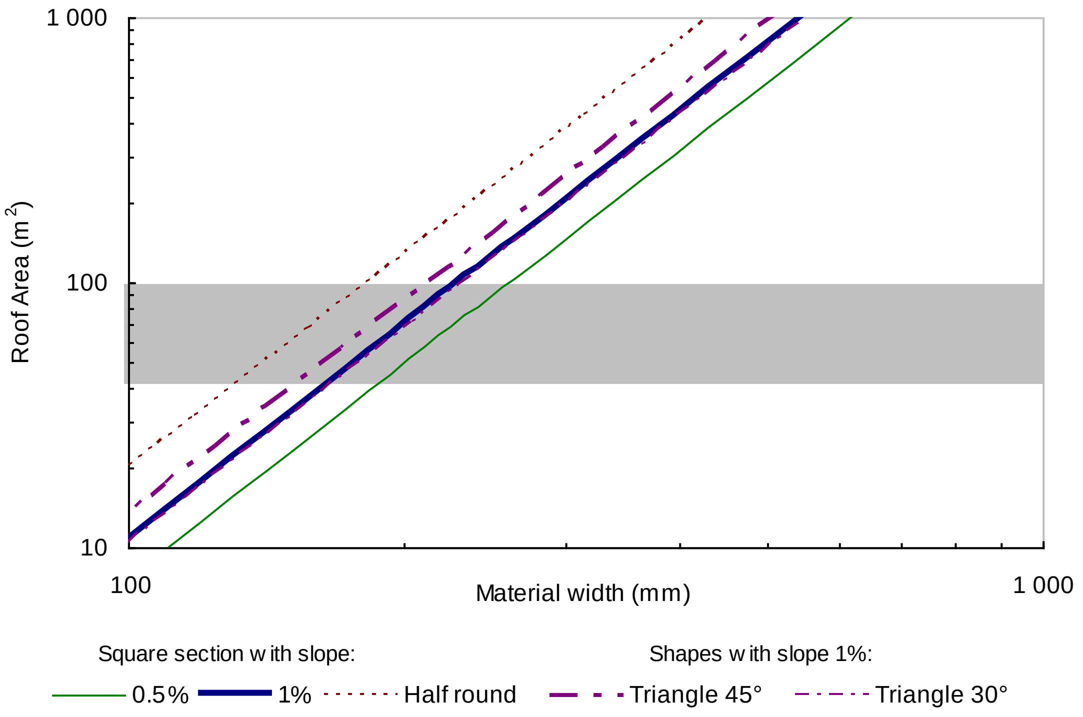
The Manning formula indicates the key variables which are:

- Material friction (usually from plastic or sheet steel)

- Profile shape
- Slope

Of these factors, material is usually set by local availability but slope and profile shape are in the hands of the designer. Slope is a trade-off between flow and interception as a high slope will increase the maximum flow but will result in the gutter falling far below the roof edge at its discharge end causing problems with interception as discussed in the next section. The profile (or shape) determines the hydraulic radius and thus the relative level of friction in the gutter – the smaller the perimeter, the higher flow the gutter will be capable of for a given cross sectional area. Coincidentally, the perimeter also determines the material use and consequently the bulk of the cost of the gutter so efficient shapes pay in two ways.

Figure 2.3: Width of sheet material required to make gutters to service various roof areas



Note: the lower material requirements for half-round gutters, which also are superior to square-section gutters in being less liable to hold stagnant pools if poorly aligned.

Figure 2.3 shows the sizing in terms of material needs for the most popular gutter profiles. The shaded area represents the range of typical domestic roof sizes. Typical roof areas in Developing areas are around 50m^2 but roof sizes for the poor can be much smaller. For a roof of 50m2 the guttering requirement is shown in Table 2.2. Typical gutter widths for such a roof quoted in the literature are shown in Table 2.3 and generally quote larger gutters (sometime much larger) than are necessary for water conveyance.

Table 2.2: Guttering for a $50m^2$ roof

	Square 0.5% slope	Square 1.0% slope	Half round 1.0% slope	45° Triangle 1.0% slope
Material use	214mm	189mm	150mm	175mm
Gutter width (at top)	71mm	63mm	96mm	124mm
Cross sectional area	$47cm^2$	$39cm^2$	$36cm^2$	$38cm^2$

Table 2.3: Gutter sizes quoted in literature

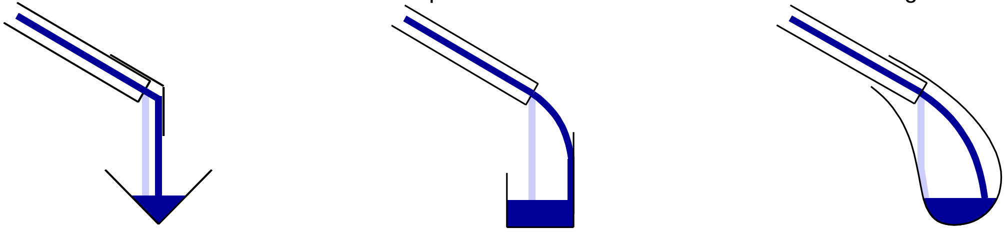
Source	Section	Roof size	Slope	Cross sectional area
[Herrmann & Hasse, 1996]	Square	$40 – 100m^2$	0.3 – 0.5%	$70cm^2$
	Half Round	$40 – 60\ m^2$	0.3 – 0.5%	$63cm^2$
[Nissen-Petersen & Lee, 1990]	45° Triangle	Not specified	1.0%	$113cm^2$
[Edwards et al., 1984]	Not specified	Not specified	0.8 – 1.0%	$70\text{-}80cm^2$

INTERCEPTION

When water falls from the roof, it can curl, around the roof edge under surface tension, it can drop vertically down or it can follow a roughly parabolic path off the edge depending on the rainfall intensity and the roofing material. Wind also complicates this as storms are often accompanied by high winds that can blow the stream of water from the roof from its natural path. Work in Sri Lanka by G.T. Still as part of the DFID project, indicates that to intercept the water for rainfall intensities from 0 to 2mm/min requires the mouth of the gutter to be 60 mm wide for a 10cm drop from the roof edge. This distance can be reduced by the use of “splash guards”, “upstands” and by enclosing the gutter-roof junction.

Figure 2.4: Methods of augmenting interception

a. splashguard c. upstand a. enclosed gutter



The splashguard (Figure 2.4a) developed in Kenya and described by Nissen-Petersen and Lee [1990] consists of a piece of downward pointing sheet metal at the lip of the roof. The off-shooting water hits the splashguard and is diverted vertically downwards into the gutter. The upstand (Figure 2.4b) effectively raises the interception height of the gutter allowing it to be mounted lower while still effectively intercepting the runoff. Enclosing the gutter-roof junction [Tapio, 1995] effectively makes interception loss impossible but uses considerably more material and makes the gutter almost impossible to clean as well as making evaporation of any retained water less efficient [Wade, 1999].

The cumulative losses from gutter interception and conveyance over a period of time are usually lumped in with the roof runoff coefficient, however no direct information is usually given about the gutter system used adding to the uncertainty in the use of a one-value runoff coefficient in the modelling of a RWH system. For models used in this thesis the gutters are assumed to be competently designed and installed, so perfect capture and transportation are assumed up to 2 mm/min rainfall intensity after which the gutters are assumed to convey only the flow derived from 2 mm/min

2.3.2 GUTTER TYPES

Rectangular sheet metal gutters with small upstand in Uganda

Extruded plastic gutters in Sri Lanka Note: Chains used to direct downflow

Experimental shade cloth gutters in South Africa [Picture Morgan [1998]]

Vee shaped gutters with splash guard in Ethiopia

There is a staggering variety of guttering available throughout the world. From prefabricated plastics to simple gutters made on-site from sheet metal and even bamboo.

PLASTIC

Gutters made from extruded plastic are popular in high-income countries; they are durable and relatively inexpensive. Mounting is usually by way of purpose built brackets and there is an array of hardware for joining, downpipe connection and finishing ends. They are less available in low-income countries and tend to be expensive there, however in countries with a good industrial base, such as Mexico, India and Sri Lanka, plastic gutters are readily available for reasonable prices.

ALUMINIUM

Aluminium guttering is extremely popular in countries such as Australia and the USA where it dominates the market. It is rolled on-site from coils of sheet metal in lengths to suit the house, eliminating in-line joints. Aluminium is naturally resistant to corrosion and so the gutters should last indefinitely. In low-income countries where it is available, the cost of the sheet is over 1.5 times the cost of steel of the same gauge and the material is less stiff so for a similar strength of gutter a larger gauge of material is required, resulting in gutters up to three times the price. This makes aluminium gutters prohibitively expensive, however aluminium sheet is a growing market in low-income countries so the price will almost certainly come down over time.

STEEL

In Africa galvanised sheet steel gutters dominate. They are either made in small workshops in lengths and joined together or can even be made on-site by builders. Workshop-made gutters are usually square section and can employ an upstand to aid interception. Market surveys indicate the cost of these gutters tends to be in the order of 2-3 times the cost of similar gutters made on-site but they are readily available in a number of configurations (open lengths, lengths with closed ends and with attached downpipe connectors), standard mounting hardware is available and their quality is usually slightly better than those fashioned on-site.

On-site gutters are usually of a vee shape as described by Nissen-Peterson [1990] and adopted by several agencies such as CARE Zimbabwe [CARE Zimbabwe, 2000]. The shape is quite efficient but reportedly has a tendency to block with debris. Mounting the vee shape is also more difficult and they are usually tied directly under the roof or onto a splashguard.

WOOD AND BAMBOO

Wooden Planks and bamboo gutters are widely described in the literature [Agarwall & Narain, 1997] [Pacey & Cullis, 1986] [Institute for Rural Water, 1982], They are usually cheap (or even free) and all money tends to stay in the community. They do, however, suffer from problems of longevity as the organic material will eventually rot away and leak. The porous surface also forms an ideal environment for accumulation of bacteria that may be subsequently washed into the storage tank.

HALF PIPE

Half pipes have been proposed as an inexpensive form of guttering [Hapugoda, 1999] and are used in many areas. The manufacture is relatively simple, and the semi-circular shape is extremely efficient. The cost of these gutters depends on the local cost of PVC pipe, which may be more expensive than an equivalent sheet metal gutter and the opening size at the top is fixed to the standard sizes of pipe available which may not be appropriate. A variant on the half pipe is a full pipe with a slit or groove cut into it and mounted over the edge of the roof enclosing the edge. The design is adept at catching the water, however less so at transportation as the gutter can have no slope.

FLEXIBLE GUTTERING

The challenge of unusual shaped houses has confounded many gutter designers. The best solution so far appears to be in the area of flexible sheet material. Polythene has been tried, but UV radiation eventually degrades it and it becomes brittle and fails. Morgan [1998] has experimented with shade cloth (a tarpaulin-like material) in Zimbabwe and developed a bag-like flexible gutter with a nominal slope. The material is connected directly to the roof by wires on top and bottom as with an enclosed gutter.

2.3.3 MOUNTING

Ad-hoc mounting of vee shaped gutter using wires and roundwood in Ethiopia

Bracket for mounting half round gutter onto eaves in Sri Lanka

Plastic brackets mounting gutter to facia board in Sri Lanka

Mounting gutters to roof in low-income countries presents particular problems. The roof edge is very often not straight, facia boards are frequently missing and eaves end at a random distance from the edge of the roof. Any mounting system must account for these deviations and also must allow the gutter slope to be controlled within fairly fine limits.

The usual method of gutter mounting is to use the fascia board usually used to finish the edge of the rafters. Brackets can be mounted to this or nails can simply be put through the top of the gutter with a short length of small pipe as a stand-off. The mounts can be at different heights along the line of the roof to give some adjustment in height, however no adjustment is possible for distance from the roof edge unless packing material is used. The result can be a little hit-and-miss and often requires wider gutter to intercept the water falling from a crooked roof.

The fascia board is often missing so brackets have to be mounted on the top or side of the rafters themselves. The bracket can sometimes be rotated to give height control but the rotation is strongly limited by the rafter width. More often the bracket is mounted parallel to the rafter and projected by varying amounts to adjust both planes simultaneously; so that to achieve a drop the gutter will increasingly project from the building. To combat this the brackets can be bent or individual brackets can be made for each support but this is a time consuming process and can result in the gutters having a varying slope and even points of negative slope. It is also worth bearing in mind that the adjustment will take place on an empty gutter whereas a full gutter will flex the brackets somewhat altering their position.

The roof edge itself is an attractive place to mount the gutter. The gutter will automatically follow any lateral movement of the roof and the length of the mountings can be adjusted to give fine control of the drop. The mounting is also very cheap as only wire is required. The difficulty is in mounting the gutter firmly. Most under-roof systems presently employed use a triangle of wires to tie the gutter under the edge or to a splashguard. This results in a gutter that can be blown from side-to-side which interferes with good water interception (the gutters do however move out of the way of ladders automatically). The gutter is also naturally suspended with its centre below the roof so only half the gutter width is available to intercept falling water. The wires themselves are an obstruction when cleaning the gutter, as a brush cannot simply be swept along the length of the gutter. Finally, care must be taken to ensure there are enough tie wires so that the full weight of the water does not damage the roof edge.

2.4. STORAGE

2.4.1 MODELLING STORAGE PERFORMANCE

The performance of a store can be determined by the use of a mass balance model. Assuming that rainwater that is used by the household is removed from the store and considering the balance of water moving in and out of the store over a period of time, the water stored can be calculated:

$$V_t = \begin{cases} V_S & V_S \leq V_{t-1} + V_{in} \\ V_{t-1} + V_{in} & 0 < V_{t-1} + V_{in} < V_S \\ 0 & 0 \geq V_{t-1} + V_{in} \end{cases} \qquad \text{Equation 2.2}$$

Where: V_t is the volume in the store at time t (the end of the time-step); $V_{t\text{-}1}$ is the volume in the store at time t-1 (the beginning of the time-step); V_S is the total storage volume; V_{in} is the net incoming water volume over the time-step

As rainfall is most commonly measured and reported as a depth of water collected over a period of time (e.g. 1 day), the time-step can simply be set to that period and V_{in} is then simply the difference between the runoff delivered to the tank and the total withdrawals over that time:

$$V_{in} = V_r - V_w$$

Where: V_r is the total volume of runoff delivered from the gutter/downpipe system over the time-step; V_w is the total volume of withdrawals over the time-step

In cases where changes take place over the course of the rainfall event such as flow-dependent changes in volumetric efficiency of filters and gutters or dynamic system behaviour such as the activating and resetting of a first-flush device, V_{in} can be calculated as the integral of the inward water flow rate delivered from the gutter/downpipe system (Q_r) minus the total outward water flow from withdrawals (Q_w):

$$V_{in} = \int_{t-1}^{t} Q_r dt - \int_{t-1}^{t} Q_w dt$$

The value of V_r is developed by the incoming rainfall and the various inefficiencies in the delivery system discussed in Sections 2.2.1and 2.3.1.

V_w is the result of user behaviour. For a daily or monthly time-step model the JMP recommended 20 lcd may be sufficient to determine if the system delivers “adequate water” as defined by the MDGs, however real user behaviour is more complex as actual withdrawals from the system depend on the water available. These water use strategies are further discussed in Section 2.4.3

Significant differences have been found with the use of monthly and daily data when using mass-balance models, particularly when small storage volumes (<2m^3) are used [Heggen, 1993; Thomas, 2002]. Dynamic system behaviour such as the activating and resetting of first-flush devices (Discussed in Chapters 12 & 13) require even shorter time-steps.

2.4.2 MEASURES OF SYSTEM PERFORMANCE

Comparing the different systems requires some measure of performance. The actual measure used will depend on the type of analysis being considered.

DEMAND SATISFACTION

Demand satisfaction – or sometimes simply "satisfaction" is one of the simplest measures of RWH system performance. The concept is easy for most householders to grasp and is useful in economic analysis of user benefit. Satisfaction is simply the fraction (or percentage) of the water demand that is delivered by the RWH system.

TIME SAVED

The time saved is simply an extension of demand satisfaction which is easily converted to time by simply multiplying by the total time needed for the householder to fetch water from an alternative source over the analysis period. As over 80% of the economic benefit of all water provision is in the value of time saved, [Hutton & Haller, 2004] this is an important measure for economic analysis though not especially useful to a householder as it is unlikely they will have summed all the time spent fetching water – though the process of doing so may itself be enlightening.

LONGEST EMPTY PERIOD

The Longest empty period is simply the longest period that the system does not deliver water. It is found by counting the number of consecutive days the system delivers no water over a period and taking the largest number. A modification that makes the period slightly more realistic is to use 5 consecutive non-dry days as the criterion to stop counting, as this will avoid a single, short rainfall event resetting the counter during a long dry spell. The measure is useful to householders as it provides information relevant to a psychological barrier. In group and household interviews carried out at the beginning of this research, the most frequently asked question was "how long will the system fail to provide water?"

RELIABILITY

The reliability of the system is the fraction of days the system meets the total demand. As an economic measure it is somewhat flawed as it fails to take into account days when a fraction of the demand is met (and water fetching is reduced rather than eliminated), however it is often more comprehensible to householders than demand satisfaction. In practice reliability and satisfaction are very close to each other and can be used interchangeably. Reliability tends to be a more useful measure when water conservation strategies are considered as these can result in inflated figures for satisfaction.

EFFICIENCY

System efficiency is an interesting measure though of little practical value. There are, in fact two kinds of efficiency that can be considered:

- Capture efficiency is the fraction of water falling on the roof that is used by the system – it gives an idea of how large the system can grow and still capture water. While some abhorrence to “waste” in the system is natural, it should be recognised that a system that catches all the available water is likely to be very expensive and underused

- Storage factor is the ratio of the water delivered in a year to the storage volume – it can give a rough idea of the return on the tank cost. This should, ideally be more than one.

2.4.3 WATER USE STRATEGIES

RWH systems do not exist in a vacuum but instead form a part of an set of options for water supply available to a household [Ariyabandu, 2001b; Howard, 2002]. This has two main ramifications:

- Households have other options that can be exploited when the system does not produce water so the system needn’t be called upon to produce the total water demand

- Householders will exploit the RWH system based upon a conscious decision about the relative worth of rainwater and that of other sources based on their ease and quality.

Rainwater provides first-choice water for most households owning a system as it provides maximum quality water with great convenience. It is, however, limited in volume and has significant day-to-day uncertainties in its replenishment and, in the tropics, strong seasonal variations that may leave the system with little or no replenishment for several months. Experience gained from interviewing practitioners and observing the behaviour of users suggests that for the first year or so, householders use the tank water lavishly resulting in a quick emptying of the system; in subsequent years conservation strategies are developed that ensure the water store is useful long into the dry season. These strategies reduce the water use either through an overall reduction or through a reassignment of lower-quality water from other sources to lower-quality uses (such as bathing and cleaning). The result is a reduction in rainwater use when the tank water level is low or when there is uncertainty in its replenishment. Conversely – water use can rise when there is more rainwater available. The strategies followed can be categorised as:

TOTAL DEMAND STRATEGY

Where the total household water demand is met by the system. This can be useful in situations where there is little seasonality in rainfall or where all other alternatives are impractical or unusually costly but usually results in either very large and expensive systems or poor penetration into the dry season. In this case *system demand* (D_s) will is equal to the *total household water demand* (D).

$$D_s = D \quad \text{Equation 2.3}$$

The "total demand" demand strategy is often used by an inexperienced household for the first year or so and continues to be the main paradigm for water provision agencies [Hartung & Patschull, 2001].

FRACTIONAL DEMAND STRATEGY

The systems is used to provide part of the household water demand (say just drinking water, or just water for drinking, cooking and washing dishes) year-round with the balance comes from alternate sources. The reduced system demand may be expressed in terms of a *demand fraction* (f_D)

$$D_s = f_D D \quad \text{Equation 2.4}$$

or simply as a reduced volume. Fractional demand is often employed by inexperienced but frugal householders.

VARIABLE DEMAND STRATEGIES

As households become used to the performance of the system, demand becomes more flexible and the demand fraction fluctuates based on tank level or season.

Variable demand based on water level in tank

Here the demand fraction fluctuates depending on the *volume water remaining in the tank* (V_p). This is usually expressed as a fraction of the *tank volume* (V_S). When the tank level falls below a certain fraction of the volume, the *low-level fraction* (f_l), withdrawals are reduced to a *reduced system demand* (D_{sl}). When the tank volume rises above a certain fraction of the volume, the *high-level fraction* (f_h), withdrawals can be raised to a *heightened system demand* (D_{sh}). At other times the nominal system demand (D_s) is withdrawn:

$$V_w = \begin{cases} D_{sh} & V_p > f_h V_S \\ D_{sl} & V_p < f_l V_S \\ D_s & f_l V_S \le V_p \le f_h V_S \end{cases} \qquad \text{Equation 2.5}$$

Variable demand based on season

The demand fraction fluctuates depending on previous experience or anticipated rainfall patterns. If a household gets this right, they can have a large store of water to ration over the dry season, however there is a strong risk of misjudging the date to start rationing and thus waste tank water. High and reduced demands (D_{sl} & D_{sh}) are used as before. Modelling this behaviour can be problematic, as the *average dry season* must be estimated from the rainfall data. An actual user should have more success as they can also rely on local knowledge and weather predictions from the meteorological office.

If it is envisaged that the households will use a variable strategy (which is likely), a further measure can be introduced: *Maximum low-use period*; the longest period that water will need to be reduced. It is found in the same way as the maximum dry period but using the *reduced system demand (D_{sl})* as a criterion rather than no water delivery.

2.4.4 DIMENSIONS AND SCALABILITY OF SYSTEM PERFORMANCE

The performance of the tank is dependant on a number of variables most of which can be described in common dimensions by using an appropriate ratio. The process also results in output that can be scaled to any system size. As the main inputs and outputs of the system are runoff and demand, it is sensible to express the other variables in terms of these flows.

AVERAGE DAILY RUNOFF (ADR)

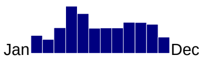
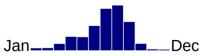
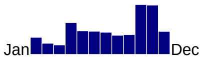
Early analyses divided runoff and tank size by demand in an effort to orient the outputs towards the user. This was later abandoned as demand is flexible while runoff is a climate-oriented variable. Through scaling, the average daily runoff (ADR) can be applied to all roof sizes, demand profiles and storage volumes. It also has the advantage of providing an early reality check on the viability of RWH in a particular situation as it is impossible to draw more water than is available to the system. As a result of this process dimensions in this thesis are stated as follows:

- Storage size is expressed in "Days ADR" which is the storage volume (length^3) divided by the ADR (length^3/time)
- Demand is expressed as a multiple of ADR
- Roof size is eliminated as ADR is a function of runoff

2.4.5 CLIMATE TYPES USED MODELLED

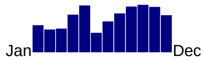
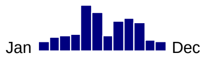
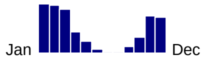
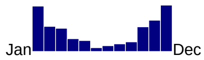
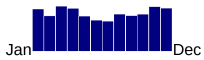
The rainfall data used for mass-balance analysis is based on ten years of daily data obtained for the three locations used for the DFID project and is detailed below in Table 2.4. They provide a range of locations, climate types and rainfall patterns.

Table 2.4: Data sources for mass-balance models with daily data

Country	Town	Köttek climate type	Mean annual rainfall	Rainfall Pattern
Uganda	Mbarara	Aw	1 700	
Ethiopia	Addis Ababa	BSk	1 200	
Sri Lanka	Kandy	Af	1 800	

For modelling in Part Two where dynamic phenomena are considered, fifteen minutely data was obtained. Unfortunately this is all US data and so has less geographic scope than the daily data, It has been possible, however to represent a number of tropical climate types and rainfall patterns as shown below in Table 2.5.

Table 2.5: Data sources for mass-balance models with 15-minutely data

State	Town	Köttek climate type	Mean annual rainfall	Rainfall Pattern
Puerto Rico	Corozal	Am	1 900	
Texas	Big Lake	BSh	480	
California	Blue Canyon	Dsb	1 700	
Hawaii	Kekaha	As	550	
Rhode Island	Newport	Cfa	1 200	

2.4.6 TANK SIZE AND SERVICE DELIVERY

The performance of Rainwater harvesting systems in Mbarara using four different performance measures is shown in Figure 2.3. Similar graphs for Kandy and Addis Ababa are in Appendix A. Two versions of each measure are shown; the measure itself which displays the overall system behaviour as tank volume is varied and a unit quantity where the measure is divided by tank volume, presenting a picture of *relative* changes with changing tank volume. On the unit quantity graph, a negative slope indicates that the relative quantity is falling, therefore, when the quantity is a benefit, displays a *dis*economy of scale.

Figure 2.3: Demand satisfaction from different volumes of water tank (based on Mbarara data)

a. Quantities
b. Unit quantities

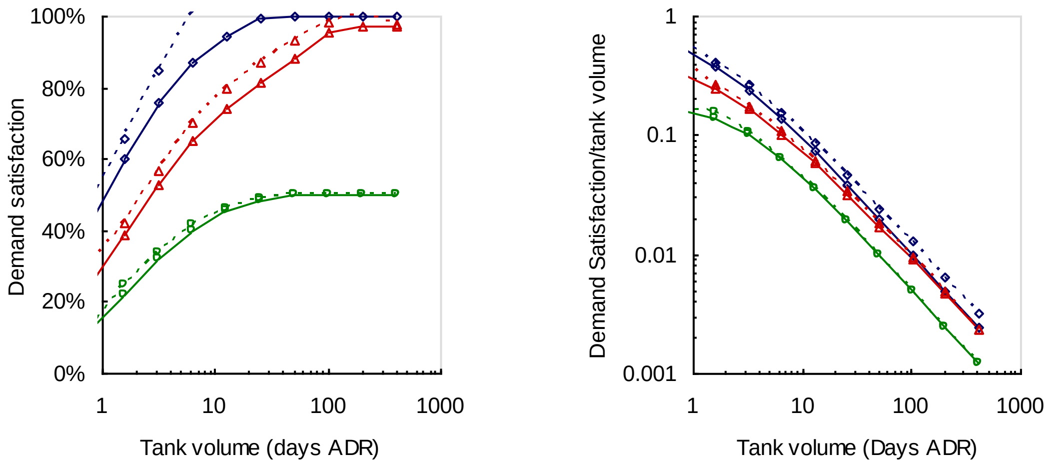
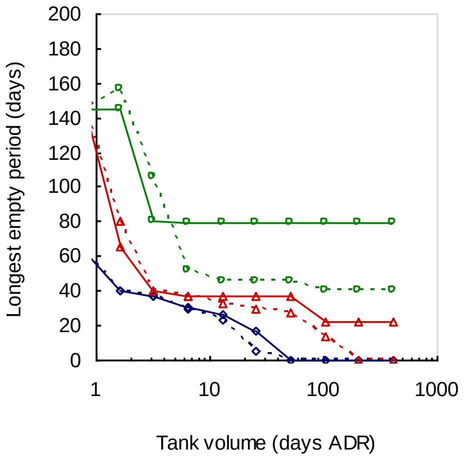


Figure 2.4: Longest empty period from different volumes of water tank (based on Mbarara data)

a. Quantities



b. Unit quantities

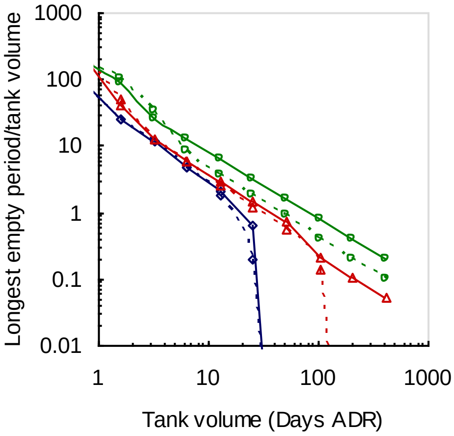
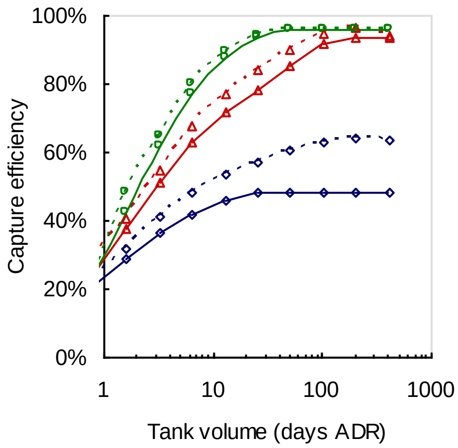


Figure 2.5: Capture efficiency from different volumes of water tank (based on Mbarara data)

a. Quantities



b. Unit quantities

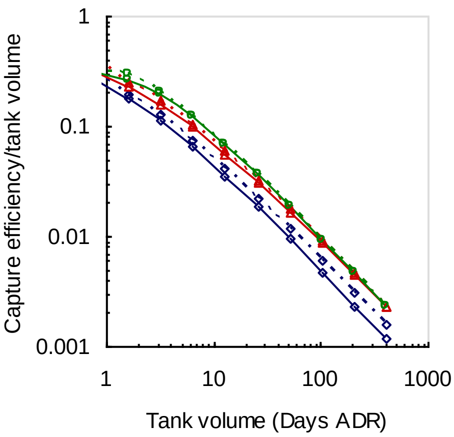
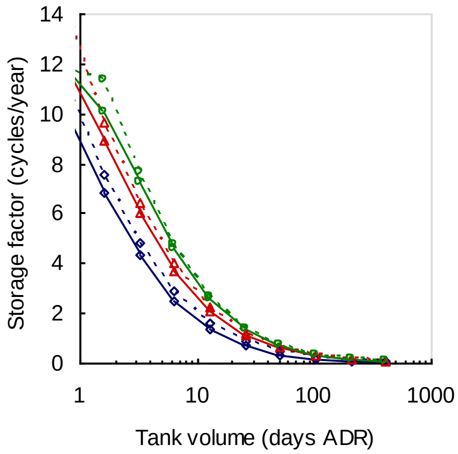
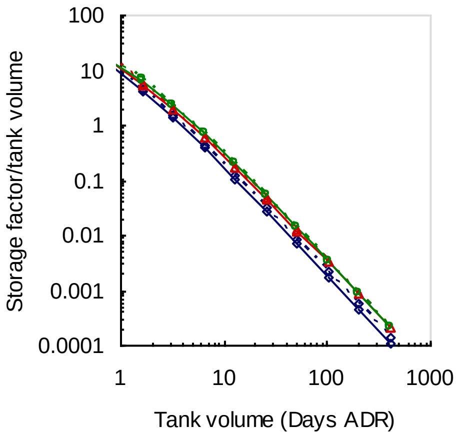


Figure 2.6: Storage factor from different volumes of water tank (based on Mbarara data)

a. Quantities



b. Unit quantities



Fixed demand = 0.5 ADR
Variable demand = 0.5 ADR Nominal
Fixed demand = 1.0 ADR
Variable demand = 1.0 ADR Nominal
Fixed demand = 2.0 ADR
Variable demand = 2.0 ADR Nominal

As can be seen from the graphs of demand satisfaction (Figure 2.3), the performance of RWH systems show strong diseconomies of scale with the charts showing a diminishing return on increased size which reduces to no increased return at all when the of roof area limits the performance rather than the tank size. The systems "*roof limit"* when the demand satisfaction reaches the reciprocal of the demand (as expressed in terms of a multiple of ADR). When demsnd is low, however the systems can provide 100% of the demand and the systems therefore "*demand limits"*. This usually means that the store once filled never empties and has a region of dead storage which has been paid for but not used.

Capture efficiency shows a similar diseconomy of scale, however the maxima is simply the demand unless the demand is equal or higher than the ADR, when the efficiency reaches its maximum slightly before 100% due to initial losses from the roof. Some insight into why this is so can be gained by examining the graphs of storage factor (Figure 2.6) where a reduction with tank size is shown that mirrors the demand satisfaction curves. As tanks get smaller, they are filled and emptied more often by users so with smaller tanks, the volume is used several times in a season. Conversely, for larger tanks the volume may only be used once or twice. In extreme cases the tank volume may never be completely used either due to a cautionary attitude to drought or poor design. The measures are also much more sensitive to demand and slightly more sensitive to management strategy than to location, this is especially true of the locations with bimodal rainfall while monomodal/monsoonal rainfall presents slightly different plots with a reduction in the performance diseconomy as the tank size becomes sufficient to bridge the dry season.

These diseconomies mean that *economic* return is best with small tanks. Thomas and Rees [1999] have shown that to optimise economic return from a DRWH investment requires use tanks that meet only around half a household's water demands. Conversely, the pursuit of the Total Demand strategy requires significantly larger sizes of tanks, and comensuratly high costs, and its promotion by water supply agencies has, in large part, produced the perception of high cost of DRWH as a water supply option.

Economic return is not the only criteria for selection for many householders. As mentioned in Section 2.4.2, the longest dry period is a significant worry for householders. Measuring this shows less of a diseconomy than the other measures. The plots of bimodal rain areas also show a plateau where the store bridges the shorter dry season and finally reaches zero when the store bridges the longer dry season. The longest dry period is much more climatically sensitive than the other measures as the fall is related directly to the length of the dry season.

Household water-use strategy has a significant effect particularly at higher nominal demands. Variable Demand can significantly reduce the longest empty period, to the point where a medium sized tank can, bridge dry seasons only possible with much larger tanks using a Total Demand strategy. This is especially found in locations with bimodal rainfall. It is also possible that the Variable Demand strategy can bridge a dry season not possible for *any* tank size using Total Demand.

2.5. CONCLUSIONS

The performance of rainwater harvesting systems depends upon the losses incurred in the different parts of the chain. The roof causes the earliest potential loss, however this is fairly uncertain and small after an initial loss. Calculations in this thesis assume an initial loss of 0.5 mm and then no significant loss. Gutters can be a significant problem in RWH systems, however if correctly designed and installed they should convey to the store the vast majority of the water running off the roof. They also form a flow limit of the system as a whole so after guttering all other components can be sized to this flow. Calculations in this thesis will assume a perfect conveyance up to a rainfall intensity of 2mm/min after which the flow resulting from this intensity only will be conveyed.

Predictions of storage performance rely on the use of a mass-balance model where withdrawals are balanced against runoff and the resultant added to or taken way from the store. There are a number of measures of performance but all show there to be significant diseconomies of scale as tank size is increased. Varying demand is shown to be as effective as increased tank size at reducing the longest empty period where nominal demand is high, so it is possible for even medium-sized tanks to bridge long dry seasons if water is used knowledgably.

PART ONE:
STORAGE COST REDUCTION

3. CURRENT PRACTICE IN ROOFWATER HARVESTING STORAGE

During initial interviews and focus group sessions with householders in Sri Lanka, Uganda and Ethiopia, the largest problem highlighted by respondents was cost – current RWH systems are too expensive for poor people to afford. In the areas where interviews took place, a typical household is only able to spend between £10 and £25 on a rainwater harvesting system. Interviews with water professionals also revealed that DRWH was considered an expensive option. Section 2.4.6 discussed the strong *dis*economies of scale inherent in rainwater harvesting systems and simple size reduction offers a very effective method of cost reduction. The second biggest problem cited, however, was inadequate water quantity – The sub-1m^3 that systems people are being offered (or are using) are too small to meet householder's requirements, particularly with regards to bridging dry seasons.

The water storage tank forms the largest single cost component of a RWH system and so is the most likely candidate for cost reduction. 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 in and with some designs the amortisation of special equipment and moulds. In this and the following chapter a number of strategies for reducing tank cost are developed.

Chapter 4 will discuss several means of cost reduction such as material reduction, material substitution and changing labour content. Paths toward these goals include improved formwork, shape optimisation, functional separation, mass production and using existing containers.

This chapter provides the groundwork for the analysis of cost reduction, discussing the needs and techniques of water storage structures and providing a baseline of current practice for cost improvements It focuses on four main areas:

- The first section discusses the basic requirements of a water storage tank and the analysis of the stresses in water tanks.
- The next section is a literature-based discussion and critique of the current state-of-the-art in water tank materials and designs.
- Section 3.3 discusses economies of scale, unit costs and the effect of these upon a tank’s contribution to household water supply.
- Section 3.4 ends the chapter with a discussion on non-monetary considerations.

3.1. Requirements of DRWH Tanks

When considering the function of a water tank, it must be of an appropriate volume for its intended performance. In a domestic situation, it should also fulfil a number of other requirements:

- The tank should not have excessive loss through seepage or evaporation (certainly less than 5% of daily demand)
- It should have sufficient structural strength to withstand water pressure and normal wear & tear
- 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 appropriate for its intended use. Drinking water in particular requires that:
  - the tank be covered to prevent entry of light, and sealed against intrusion by vermin including mosquitoes
  - the tank be ventilated to prevent anaerobic decomposition of any washed-in matter
  - the tank not give the water an unacceptable taste

In addition to these requirements, a number of desirable criteria should also be considered. It should ideally also:

- be affordable
- be durable (or easy and cheap to maintain in good condition),
- have a means by which water can easily enter and easily be withdrawn (into the normal household receptacle used in the area)
- have the ability to deal with excess input by overflowing in a manner which doesn't damage the tank or its foundations
- be easy to clean *or* be 'self-cleaning'
- have sufficient structural strength to withstand occasional large natural forces
- Look attractive

3.1.1 STRESSES IN TANKS

In order for a tank to have sufficient structural strength to withstand pressure resulting from the water it contains, it must be designed with knowledge of the stresses that this pressure causes in the tank material.

PRESSURE FORCES IN TANKS

Water exerts a pressure proportional to its depth equivalent to 10 kPa per metre of depth. 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 3.1b)

Figure 3.1: Action of pressure in a water tank

a. Straight sided   b. Doubly curved

Generally this pressure puts the tank walls into tension. This is unfortunate because many materials traditionally used for building and transferred to tank construction such as cement mortar 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 as hoop stress. This stress can be found using the equation:

$$\sigma_h = \frac{p\,r}{t} \qquad \text{Equation 3.1}$$

Where: σ_h is the hoop stress (Pa); p is the water pressure (Pa); r is the tank radius (m); t is the wall thickness (m)

The simple hoop-stress formula is, however only applicable when the walls of the tank are free to move as shown in Figure 3.2a. The movement is only very small and can be achieved by using a flexible material in the joint between floor and wall such as bitumen or by allowing the wall to slide outwards along the floor. In the more usual case where the walls are fixed to the base of a tank, they will tend to bow out as shown in Figure 3.2b

Figure 3.2: Movement of tank walls due to pressure

a. Unconstrained a. a. Constrained

This will change the hoop stress and also cause shear and bending stresses as shown in Figure 3.3. The wall will be stressed in shear in a horizontal plane at its bottom edge where the water pressure forces it outwards but the base opposes this (Figure 3.3a). Another stress is due to bending of the tank wall as it bows 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 3.3b) both acting vertically up the wall which, unless dealt with appropriately, can cause cracking of the inside face leading to failure.

Figure 3.3: Stresses caused by constrained walls

a. Shear stress b. Bending stress

Quantifying this situation is rather more complex and uses the technique of shell theory where the tank walls are idealised as being very thin. The tank is also considered to be made of a homogeneous material which obeys Hooke's law. The relevant equations [Flugge, 1967] are:

$$N_\theta = \gamma r\left(h - x - he^{\frac{-\lambda x}{r}} \cos\frac{\lambda x}{r} + \left(\frac{r}{\lambda} - h\right) e^{\frac{-\lambda x}{r}} \sin\frac{\lambda x}{r}\right) \qquad \text{Equation 3.2}$$

$$M_x = -\frac{\gamma r t}{\sqrt{12\ 1-\mu^2}}\left(\left(\frac{r}{\lambda} - h\right) e^{\frac{-\lambda x}{r}} \cos\frac{\lambda x}{r} + he^{\frac{-\lambda x}{r}} \sin\frac{\lambda x}{r}\right) \qquad \text{Equation 3.3}$$

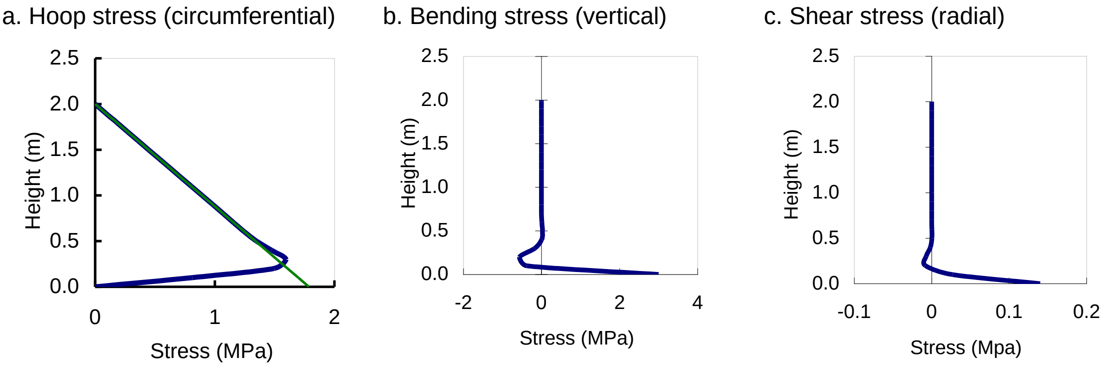
$$Q_x = \frac{\gamma t \lambda}{\sqrt{12\ 1-\mu^2}}\left(\left(\frac{r}{\lambda} - 2h\right) e^{\frac{-\lambda x}{r}} \cos\frac{\lambda x}{r} + \frac{r}{\lambda} e^{\frac{-\lambda x}{r}} \sin\frac{\lambda x}{r}\right) \qquad \text{Equation 3.4}$$

where: N_θ is the radial hoop force (N/m); M_x is the bending moment (Nm); Q_x is the shear force (N/m); γ is the specific weight of water (N/m^3); r is the tank radius (m); h is the water level (m); x is the height of the stress to be calculated (m); t is the wall thickness (m); μ is Poisson's ratio for the material; λ is defined by the expression:

$$\sqrt[4]{3\ 1-\mu^2\ \left(\frac{r}{t}\right)^2}$$

As the section is considered "thin" they give results in terms of "force" (N/m) rather than stress (N/m^2). Stress is obtained by dividing the result by the wall thickness. The equations may be coded into a spreadsheet and used to provide curves for designing tanks. Typical output is shown in Figure 3.4

Figure 3.4: Stress curves for cylindrical tank with fixed base



The minimum necessary wall thickness is that for which the calculated stress is below the maximum material strength – or, more sensibly the material strength divided by a safety factor. Typical thicknesses necessary for cylindrical tanks of a range of sizes are detailed in Table 3.1. The calculated thicknesses are far below that necessary for breakage with safety factors varying from 4× to 10×.

Table 3.1: Calculated and actual wall thicknesses for cement mortar above-ground tanks

Capacity (m^3)	Dimensions – height:diameter (m)	Calculated thickness (mm)*	Typical material thickness (mm)	Dimensions taken from
1	1.2:1.1	3.5	50	[Luong, 2002]
2	1.4:1.5	5.5	50	[Luong, 2002]
5	1.8:1.8	9.5	40	[LRWHF, 2000]
10	2:2.5	13.8	60	[Gould & Nissen-Petersen, 1999]

* for 1:3 mortar (max tensile strength 3MPa) and with no safety factors included

STRESSES IN FLAT-WALLED TANKS

In addition to these stresses, rectangular and other flat walled tanks have additional stresses localised at the corners because the pressure will bow out the flat walls in much the same way as at the base in a cylindrical tank. The corners restrain this movement in a similar way to a constrained base, resulting in bending and shear stresses. Plate theory can be used to analyse these structures, however commercial finite element programmes are considerably simpler – though care needs to be taken in their use.

In practice, flat walled tanks have not met with any great success despite seeming advantages in ease of manufacture – particularly with their potential for mass manufacture and transport of flat panels for assembly on-site. Few such tanks have been made and flat-walled tanks are manly found as small brick-built structures.

More material efficient tanks can be designed that are not cylindrical. Examples include the "Thai" jar and the Sri Lankan pumpkin tank. Savings are made through several routes:

- Lower surface-area to volume ratio (see Section 4.2.1)
- Reduced diameter at high-stress points such as the bottom of the tank
- Reduction in bending moment through shapes that control the stress to predominantly occur tangential to the surface of the tank

Such shapes do not lend themselves to straightforward analytical solutions, however some solutions can be found using numerical methods and shell theory [Flugge, 1967; Kelkar & Sewell, 1987; Zingoni, 1997]. Such analysis is beyond the scope of this thesis but has been used in parallel research by Still [Still, 2006]. Ghali [Ghali, 2000] presents an analysis based on an elastic foundation analogy that can be used to derive a matrix for any radially symmetrical structure and produce a finite-element solution along the height of the structure. A more convenient method is, however to use one of the commercially available finite element analysis programmes for the analysis.

3.2. MATERIALS AND TECHNIQUES

The following section is a review of different techniques currently being employed to build water tanks. The techniques are analysed for their advantages and disadvantages and some conclusions drawn about how they are best used. A number of case-studies of designs of particular note are also included in Appendix C. Chapter 4 draws from this material to generate a number of generic strategies to reduce tank cost.

3.2.1 PRECAST CONCRETE


A pair of precast tanks in rural Australia
[picture: Everlast tanks Pty. Ltd]


A precast concrete tank in urban Germany
[picture: Mall GmbH]


Precast roof plates being placed on a tank in Brazil
[picture: [Gnadlinger, 1999]

In some developed countries such as Australia and Germany precast concrete tanks form a large part of the market. The tanks are cast in sizes up to 35m^3 under controlled conditions, delivered to the site by truck and installed with a crane. The economies inherent in this strategy revolve around the ability for the factory to specialise in this type of construction, the use of appropriate jigs and the ease of installation reducing on-site labour costs. In Germany most tanks are sited underground to reduce space requirements.

There have been several attempts to build such tanks in low-income countries such as Brazil [Szilassy, 1999] and Kenya [Lee & Visscher, 1990] using shuttering with corrugated iron, however the technology has generally proven too expensive to be widely replicated. Precast rings have been used successfully in Bangladesh [Ferdausi & Bolkland, 2000] already being produced in quantity for well lining. This ability to mass produce items gives the technique some promise in the field of tank components such as segmented covers and filter boxes and concrete is often used for ancillary work around tanks such as foundations, drainage and soakaways.

3.2.2 STEEL

A large steel tank in rural Australia
[picture: Pioneer Tanks Pty. Ltd]

A corrugated steel tank in rural Uganda - note the concrete ring around the bottom repairing leaks
[picture D. Rees]

An “Oil” drum in rural Uganda
[picture D. Rees]

Steel tanks of various sizes have been used throughout the world for many years and are still popular today. They range from the steel drums found outside many houses in East Africa to gigantic 1.5 Ml structures used to supply remote communities in Australia. The tanks can be delivered to a site and installed in a short time by a skilled person. An extremely firm foundation is often not required, as the steel structure will “give” a little to accommodate any settling.

In low-income countries problems with corrosion on the bottom of the tank have been observed after about two years service. Building a concrete ring around the base of the tank can repair this, however such failure in the field has limited the steel tank's acceptance and wider application. The problem does not generally appear in tanks built in developed countries as steel tanks are generally either coated with plastic on the inside of the tank [BHP Pty. Ltd., 2000] or lined with a plastic composite bag [Pioneer Tanks Pty. Ltd, 2000].

Oil drums (usually ~ 220 l) are one of the most widely dispersed water containment stores in the world. However, they have unique problems due to their previous use.

- Most drums now used for containing water have previously contained chemicals, many of which are toxic.
- They have usually been opened in such a way that they are uncovered and thus present an ideal environment for mosquito breeding and yield low quality water.
- Water extraction can be a problem.

These problems can be solved by careful selection and cleaning, the use of a cover and installation of a tap. When these steps are taken, drums form a readily available supply of inexpensive (if small) storage units.

3.2.3 PLASTIC

Plastic tank intended for underground installation in Germany
[Picture: Roth GmbH]

Plastic tanks in Uganda
[Photo D. Ddamulira]

Tarpaulin lined underground tank in rural Uganda
[Photo D. Rees]

Plastic tanks, usually made from HDPE or GRP form the fastest growing segment of storage provision. They are already popular in developed countries where they compete directly with older technologies such as steel or concrete on a direct price basis. In low-income countries, these tanks are generally more expensive by a factor of 3-5 which has slowed their adoption, however this is changing, In Sri Lanka the price penalty of a plastic tank is down to about 1.5-2 and in South Africa they are generally considered cheaper [Houston, 2001].

Even in countries where there is a price premium for plastic tanks, they are often employed by water supply organisations, as they are quick to install and are known to work reliably (usually backed by a manufacturers guarantee). Householders also like the tanks and see them as the most up-to-date method of storing water, however problems of cleaning, of the water heating up in the black tanks, and proneness to accidental damage when softened by the sun have been identified.

An application of plastics that is highly cost effective is the use of plastic lining materials with otherwise local techniques. An example of this is the tarpaulin tank originally used by Rwandan refugees in southern Uganda and subsequently improved by ACORD and widely replicated [Rees, 2000]. The cost of the tank is roughly ½ of an equivalent ferrocement tank. The frames of the tanks are, however liable to termite attack and the tarpaulins themselves have failed in service in some areas reportedly also due to termites although contrary stories exist. These problems and their ramifications are further discussed in Chapter 5.

3.2.4 FERROCEMENT

Household ferrocement tank in Ethiopia [Picture: S. Akhter]

Mass produced cement jars for sale by the side of the road in Thailand [Picture: R. Ariyabandu]

Ferrocement "pumpkin" tank in Sri Lanka [Picture: T. Ariyananda]

Ferrocement (FC) is the technology of choice for many rainwater harvesting programmes, the tanks are relatively inexpensive and with a little maintenance will last indefinitely. The material lends itself to being formed into almost any shape and apart from tank construction, is used for boat building and even sculpture. It has several advantages over conventionally reinforced concrete principally because the reinforcement is well distributed throughout the material and has a high surface area to volume ratio.

- Cracks are arrested quickly and are usually very thin resulting in a reliably watertight structure
- Ferrocement has a high tensile strength (up to 35 MPa [Naaman, 2000])

- Within reasonable limits, the material behaves like a homogeneous, elastic material

The technique was developed in France in mid 19th century and was initially used for pots and tubs and even boats, [Morgan, 1994] but was supplanted by less labour intensive reinforcement methods. Water tank construction with ferrocement has been ongoing since the early 1970s [Watt, 1978] was popularised by its use in Thailand [IDRC, 1986] and has spread to Africa [Nissen-Petersen & Lee, 1990], South America [Gnadlinger, 1999] and Sri Lanka [Hapugoda, 1995] among others.

The method of construction involves the plastering of a thin layer of cement mortar (typically 1 part cement to 3 parts sand mixed with about 0.4 parts water) onto a single or double layer of steel mesh. Traditional ferrocement has up to 6 layers of mesh/cm of thickness with a volume fraction of up to 8% steel [Naaman, 2000] while "ferrocement" water tanks have a much lower volume fraction due to their use of only 1 or 2 layers for up to 50mm thickness. It has been suggested that they are not ferrocement at all but a form of "mesh reinforced mortar" tank [Skinner, 1995]. The low volume fraction of mesh in an FC tank makes its function uncertain. At the typical 0.1–0.5% volume fractions found in FC tanks, the contribution to strength is only 1–4%, so the mesh doesn't contribute greatly to the structure. The mesh is also placed in the centre of the matrix and so will play no part in bending resistance. It does, however form an armature upon which to plaster mortar and remove any need for shuttering. In the case of above-ground tanks, the mesh can limit dangerous projectiles in the event of a bursting failure.

Despite being described as a "low skill" technique, workmanship is a strong issue with all ferrocement constructions. The thickness of mortar covering the mesh can be as little as 5mm giving little room for error. Solid backing formwork such as steel sheet or concrete blocks can reduce thickness variation and is often the reason behind successful designs. Increasingly though, these formers are being abandoned due to their cost and to gain flexibility in size. The mesh armature itself can be used as a base to cement on to, but such tanks usually have an increase in wall thickness and thus cost compared with those that use solid formwork (see Section 4.2.1).

The most popular design of ferrocement tank continues to be the straight sided cylinder. Removable formers are easy to construct using sheet metal, or BRC mesh can be used to stiffen the armature, and there are usually few foundation rotation problems as the base is wide enough. There can be some problems of cracking at the base if stress concentrations are not accounted for in the design and there have been some reports of cracking at the lid interface. To combat this, several designs such as the Sri Lanka “pumpkin” tank [Hapugoda, 1995] have been produced with a rounded shape to break up the sudden junctions.

Even more popular than cylinders in terms of number built, but not technically “ferrocement” is so call “Thai jar”, which has been mass-produced in Thailand for about 20 years. There are millions of these jars throughout Thailand with capacities ranging from $0.5m^3$ to $3m^3$ [Bradford & Gould, 1992] The jars are manufactured by plastering mortar with no reinforcing onto a mould in the shape of the jar. The moulds are centrally produced to a low cost and the jars are made in reasonable numbers in small workshops [Ariyabandu, 2001a].

Another method of employing mass production techniques is to make the tank in sections. This has been applied in India at the Structural Engineering Research Centre [Sharma & Surya Kumar, 1981; Sharma, 2005] where the tank is made in either full height or half height segments either on-site or in a central location. The segments are then shipped out by truck and joined together on-site in a single day. The segments themselves have tightly controlled and, in consequence, much-reduced thickness as they can be made horizontally at a comfortable height on well-designed jigs. Segmented techniques have also been tried in Brazil [Gnadlinger, 1999] with the segments made on-site. The material cost is similar to same-sized ferrocement tanks made on a former but the tanks are quicker to build.

The realm of ferrocement has also seen several attempts to reduce cost by replacing metal reinforcing with other materials such as bamboo and hessian. Although there have been some success stories [Sharma et al., 2001] there are a number of notable and large-scale failures. In Thailand, over 50,000 bamboo-cement tanks had already been built before a study by Vadhanavikkit and Pannachet. [1987] revealed that fungi and bacteria were decomposing the bamboo and within a year the strength of the reinforcing had reduced to less than 10% and some had rotted away altogether. The study concluded that the majority of bamboo cement tanks would fail, some suddenly and dangerously. Another programme in East Africa by UNICEF and Action Aid in the 1970’s developed the “ghala basket” an adaptation of a traditional grain basket made waterproof by the addition of mortar. By the mid 1980s, it was becoming clear that these baskets were susceptible to rotting and termite attack and the design was abandoned [Gould, 1993].

3.2.5 BRICKS

Burned brick tank in rural Uganda
[Picture: V. Whitehead]

Tank made from stabilised soil blocks in urban Uganda
[Picture: T. Thomas]

A communal masonry tank in Rural Ethiopia
[Picture Water Action]

A plastered rectangular brick tank in rural Sri Lanka
[Picture: D. Rees]

Bricks and blocks of various types are widely used for building in many low-income countries. The materials are found locally and local people prepare the bricks themselves, thus the cost of the bricks is usually low and all monies remain in the local economy. They can be made from a number of different materials such as burned clay, cut stone, soil stabilised with a small amount of cement or even concrete. Unfortunately, while bricks are useful for building work they are less well suited to tank construction. Due to the lack of continuity between bricks, the tensile forces have to be carried by the mortar and by adhesion between the mortar and bricks, which is usually fairly low. Brick tanks can also suffer a cost disadvantage as the thickness of the tank is set by the width of the bricks and if the bricks are poorly fitting, such as when making a cylinder of small diameter from rectangular bricks, they can actually demand more cement than an equivalent ferrocement tank.

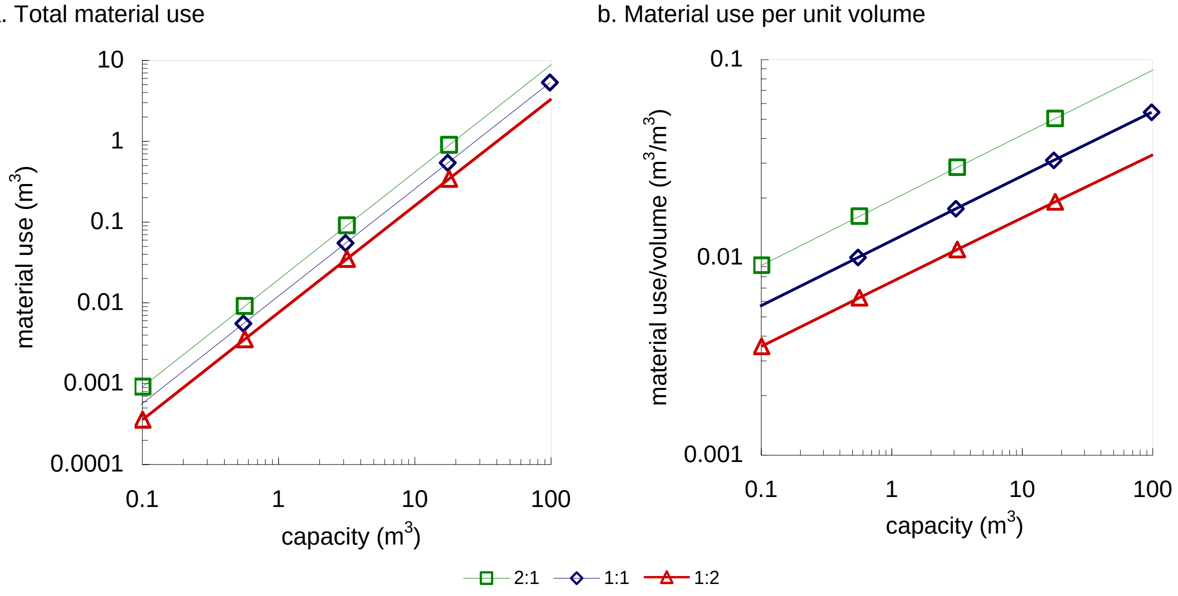
Interlocking bricks, usually using stabilised soil, have been tried in several places including, Thailand [IDRC, 1986] and Uganda [Rees & Thomas, 2000; Kalebbo, 2001]. A machine for making interlocking mortar blocks designed for tank building is also commercially available [Parry, 2001]. Most of these designs, however only interlock by a vertical tongue and groove so must rely on shear between the mortar and block to take the tensile hoop stress. A more satisfactory solution used in Thailand is to interlock the block horizontally on top and bottom. The Thai design also avoided the problems of wall thickness and fit by using thin blocks with angled edges.

3.3. ECONOMIES OF SCALE AND UNIT COSTS

3.3.1 DEVELOPMENT OF ECONOMIES OF SCALE

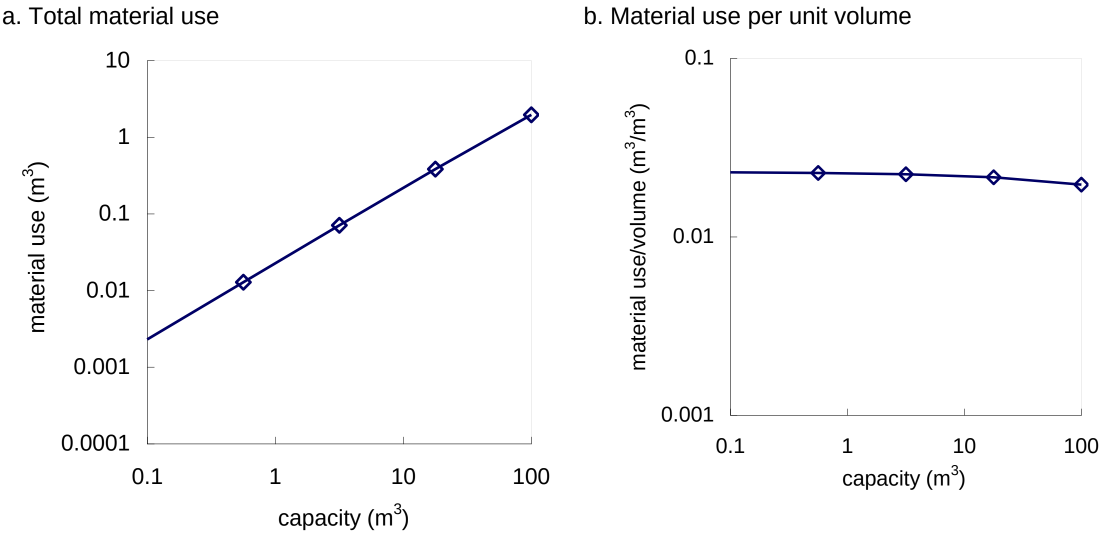
When the Equation 3.2 – Equation 3.4 are used to calculate the maximum stress in a series of tanks with constant aspect ratio a simple power law relationship of both cost and unit-cost to volume is established. A surprising result here is that unit cost *rises* as volume goes up, suggesting that many small tanks are a better investment than one large one on a per volume basis.

Figure 3.5: Theoretical material requirement for tanks of constant aspect ratio



Of course, real-world tanks are restricted by real-world constraints and are rarely scaled with a constant aspect ratio as they inevitably have to fit under roof eaves and therefore rarely exceed 2m in height. Adding this constraint, the diseconomies of scale disappear as can be seen in Figure 3.6.

Figure 3.6: Theoretical material requirement for tanks of constant height



These charts also only reflect the amount of material used in the walls of the tank. Tanks usually have a cover and floor which further complicates the matter as, as particularly at very wide aspect ratios the cover and floor can be considerable parts of the overall cost. Moreover, the analysis of these components is much more complex than the analysis of the walls of a cylindrical tank.

The tank cover can take several forms:

- *A flat slab* will tend to sag in the middle giving rise to bending stresses that are greatest at the rim. With a uniform, distributed, vertical loading such as self-weight or water pressure, these stresses vary with the square of the radius and inversely with the square of the thickness [Young & Budynas, 2001]. A tank cover will mostly only have to withstand self load so the load will be proportional to its thickness, so the stress will actually vary with the square of radius and the inverse of thickness.

- *A dome with a circular cross section*: Bending moments will be developed with an inflection at some point along the length of the dome. With a uniform distributed vertical loading, the absolute value of the maximum stress varies with the radius and inversely with thickness, with self loading the pressure load comes from the weight so the maximum stress is only proportional to the radius.

- *A dome with a catenary cross section:* If only self load is considered, all forces will be compression within the material of the dome, so the thickness can take any value as all as the self-load force is proportional to the thickness. In practice the minimum thickness would be that needed to support a person climbing on the cover

These are further complicated by the possibilities of ribbing and internal support which will be very design specific. The domed roofs will also put additional stress on the top of the walls, increasing hoop stress at the top and adding some bending at the bottom.

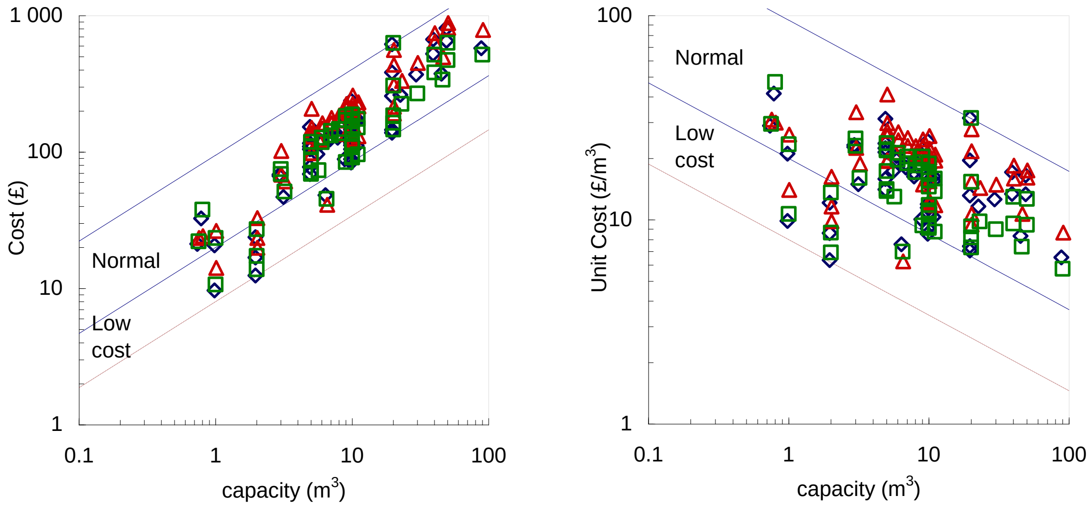
The floor is dependent on local soil conditions and can modelled as a slab of fixed thickness which is subject to a pressure load and so the maximum stress will vary with the square of the radius and inversely with the square of the thickness.

It is clear that the economies of scale for water tanks is more complex than can be adequately described by using a simple set of assumptions and calculations. An analysis of a number of existing designs is more revealing. Figure 3.7 shows the costs of 60 tanks calculated from their bills of materials and material and labour costs in three countries.

Figure 3.7: Costs of 60 tank designs derived from bills of materials and material and labour costs in three countries, economies of scale and cost ranges

a. Overall cost

b. unit cost



The cost-capacity relationship follows a definite pattern, particularly when a family of designs is considered. Regression lines through the data show the increase in cost with tank size is roughly equivalent to:

$$\frac{C_a}{V_a^{0.65}} = \frac{C_b}{V_b^{0.65}} \qquad \text{Equation 3.5}$$

Where: C_a is the cost of tank "*a*"; C_b is the cost of tank "*b*"; V_a is the volume of tank "*a*"; V_b is the volume of tank "*b*"[1]

Or for the fall in unit cost:

$$\frac{C_{1,a}}{V_a^{-0.35}} = \frac{C_{1,b}}{V_b^{-0.35}} \qquad \text{Equation 3.6}$$

Where: $C_{1,a}$ and $C_{1,b}$ are the unit costs of tank "*a*" and "*b*" respectively

Using these formulae it is possible to scale any design to give an estimate of its cost or unit-cost at another capacity.

The lines on Figure 3.7a are drawn according to Equation 3.5 and those in Figure 3.7b are drawn according to Equation 3.6. They demarcate two cost regions. Most costs lie in a region labelled the "normal" region however the costs of three tanks fall below this into a region of "low cost". The tanks are two sizes of Thai jar and the Ugandan tarpaulin tank. The reasons for this will be discussed further in Chapter 4. The bottom boundary of the low cost region is fairly arbtrary as future designs could conceivably be below the line; however it demarcates the current limits of low cost designs.

An underlying goal of the design component of this research project is to "fill out" this region. Although the Thai jar is an excellent design, it does not scale well as it relies on workshop practice for its material and labour economies – so the finished tanks must therefore be small enough to transport. The only scaling option is then to add more jars. This means that the unit cost will remain the same so by the time several have been acquired the EUC for the total storage is unexceptional. The tarpaulin tank is a large design but has unique scaling problems and questions of longevity. These are explored in Chapter 5.

It is traditional to discuss tanks in terms of a unit cost – usually cost per litre or cost per cubic metre [Gould & Nissen-Petersen, 1999] however this simple description does not take economies of scale into account and so will tend to favour any technology that is built to a large scale; e.g. a unit cost of 1.5 pence per litre is exceptional for a $1m^3$ tank, but fairly ordinary for a $10m^3$ tank. A more useful concept is the "equivalent unit cost" (EUC), which is a more accurate

[1] The exponent actually varies between 0.60 and 0.66 depending on location.

way of describing the unit cost of a tank. The EUC is what a tank using a particular technology would cost if it were scaled to 1m^3 (1000 litres) capacity and is given by:

$$C_1 = \frac{C}{V^{0.65}} \qquad \text{Equation 3.7}$$

Where: C_1 is the equivalent unit cost; C is the cost of a tank of volume V; V is the tank volume (in m^3)

Some typical equivalent unit costs are given in Table 3.2.

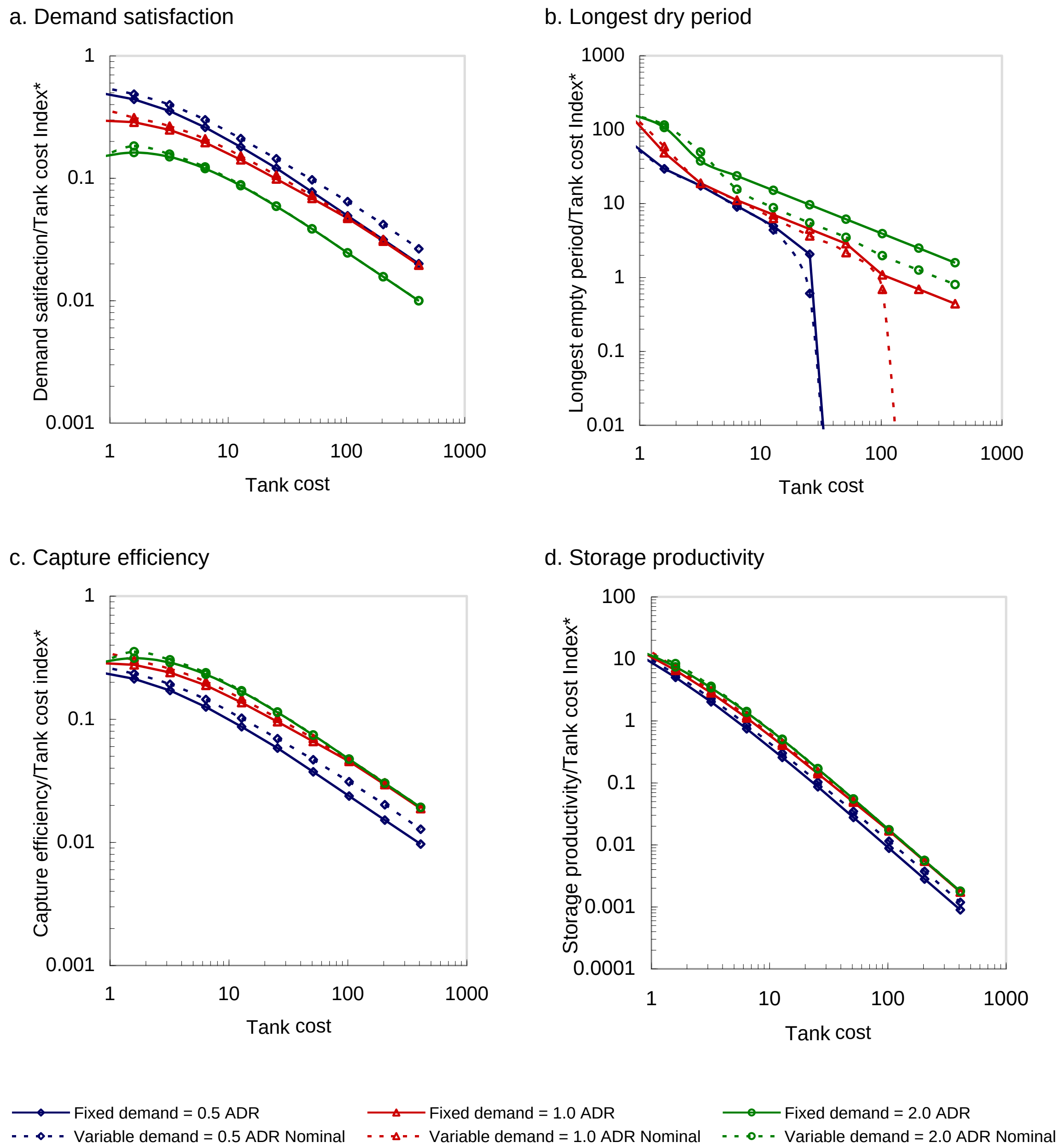
Table 3.2: Equivalent unit costs of rain-tanks (based on Ugandan material and labour costs)

Tank type	Tank cost	Tank capacity (m^3)	Simple Cost per m^3	Equivalent unit cost
Moulded plastic	$750	25	£18	£93
Open frame ferrocement	$350	10	£21	£78
Pumpkin tank	$160	5	£19	£56
Plate tank	$150	10	£9	£34
Thai jar	$44	2	£13	£28
Tarpaulin tank	$62	5	£7	£22

3.3.2 EFFECTS OF TANK ECONOMIES OF SCALE ON WATER SERVICE DELIVERY

As discussed in Section 2.4.6, rainwater storage suffers from significant *dis*economies of scale in terms of water delivery to the household with large changes in volume resulting in only small changes in the actual amount of water delivered to the household. The economies of scale discussed above beg the question of how changes in *investment* affect water delivery. If Figure 2.3 is redrawn with unit quantities derived from cost index rather than volume, the result for Mbarara, Uganda is shown in Figure 2.3 with results for three locations shown in Appendix B.

Figure 3.8: Measures of tank performance based on Mbarara data



* Tank cost index is based on normalising to the cost of a 1 day ADR tank

As can be seen, applying economies of scale in tank construction to the mass balance model Reduces the diseconomies of scale discussed in the last chapter. Tank construction economies of scale are however not large enough to eliminate or reverse the diseconomies scale in of rainwater storage.

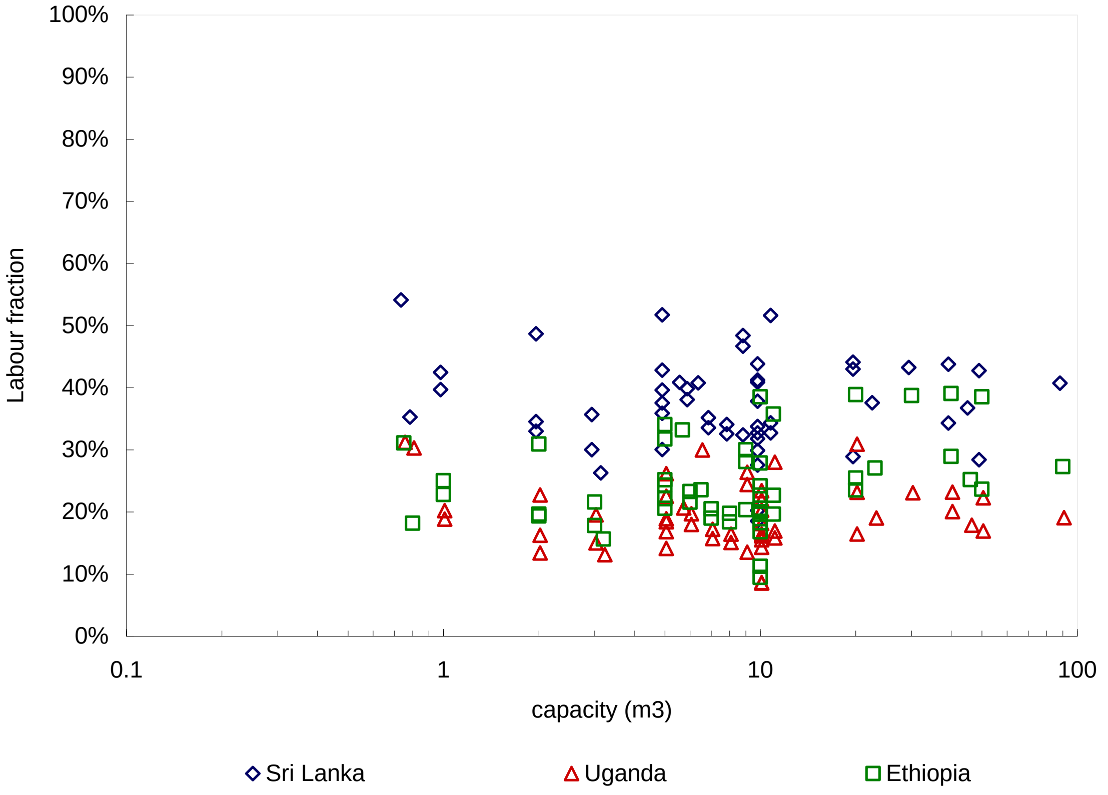
3.4. NON-MONETARY CONSIDERATIONS

As well as monetary accounting, a number of other measures of tank designs can be used that can reflect particular concerns of water providers. Such measures go beyond simple “cost” analysis and consider such issues as use of beneficiary time and local balance of payments.

3.4.1 LABOUR CONTENT

Generally, the labour used in building tanks will be local, so tank-making will generate local employment and money will go into the local economy. So using designs whose construction has a high labour content will usually be better for the community as a whole than using designs with a high component of “bought in” materials. Labour-time varies with tank size in much the same way as overall cost does, so the easiest way of expressing labour cost is as a *fraction of total cost*. This fraction stays fairly constant throughout changes in tank size as shown in Figure 3.9.

Figure 3.9: Labour cost as a fraction of total construction cost based on bills of materials for 60 tank designs derived from bills of materials and material and labour costs in three countries



The fraction does show strong variation from design-to-design with the difference between highest and lowest labour fractions of 23% for Uganda, 30% for Ethiopia and 36% for Sri Lanka. Variations of almost 100% are found from country-to-country as the relative costs of labour and materials change. The average labour fraction for Sri Lanka is 37%, Ethiopia 25% and Uganda 19%. Had high-income countries been included in the analysis the range would doubtless have been higher still. This variation becomes particularly important as materials are reduced at the expense of labour, a trade-off that will become untenable in Sri Lanka while still viable in Uganda. It also effects the relative merits of low-capital cost – high maintenance designs as seen in Chapter 6. Some labour-cost fractions for production of tanks are given in Table 3.3.

Table 3.3 Labour cost as a fraction of the total cost, for selected rainwater tanks

Tank type*	Ethiopia (variable material cost & low labour cost)	Uganda (high material cost & low labour cost)	Sri Lanka (low material cost & medium labour cost)
Pumpkin tank	35%	25%	50%
Open frame ferrocement	25%	25%	35%
Tarpaulin tank	20%	20%	33%
Plate tank	15%	15%	30%
Thai jar	10%	20%	30%
Moulded plastic	<5%	<5%	10%

3.4.2 POTENTIAL FOR HOUSEHOLDER CONTRIBUTION "IN KIND"

HOUSEHOLD LABOUR CONTENT

Often, in projects whose beneficiaries are very poor, household contribution to costs is made in the form of "unskilled" labour[1]. If householders are willing and able to provide this, then choosing a design with a high unskilled labour content will allow a larger store to be provided than would otherwise be possible. Similarly, if the tank is being purchased outright, a high household labour content will allow the householder to afford a larger system than they would otherwise. This, of course will only continue up to the stage where the need for labour and the organisation thereof is considered to be a burden. In the case of water agency provision, this is an issue that should be discussed with the community at the technology-selection stage.

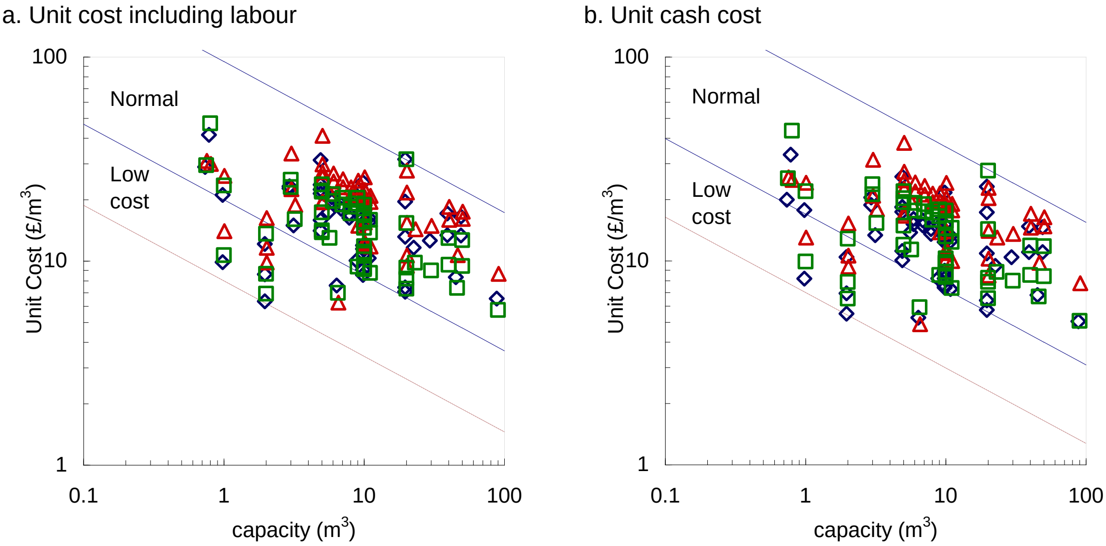
In scaling exercises, the household labour cost is most easily considered as a constant fraction of the total cost. The fraction can be removed from the scaled cost when budgeting a

[1] Though widely used, the term "unskilled" is something of a misnomer when certain techniques are being used. In Sri Lanka when one design required a "wattle and daub" construction the masons involved in tank construction were actually corrected by one householder in the proper use of this technique. In the end she built her own tank structure and "skilled" input was only needed in the final stages or waterproofing and plumbing the tank.

programme and labour times can simply be scaled at the same rate as the tank cost and presented as days labour when designs are presented to householders.

While household labour is often ignored in costing programmes aimed at the poor, another approach is to allocate an opportunity cost to such a contribution to reflect the loss of time to the household (HH). This opportunity cost can be expressed as a fraction of the unskilled labour rate. In much of the analysis carried out in this thesis, the fraction is taken as 50%. The real value of householders' time is more uncertain, though it will usually lie somewhere between zero and the full unskilled labour rate. Due to this uncertainty, costs in this thesis will usually be expressed with HH labour content valued both at the full cost of unskilled labour and also with HH labour content removed from the costing (referred to as "Cash cost") to allow the reader to assign the opportunity cost whatever they deem appropriate. Figure 3.10 shows the difference made by removing household labour. All costs reduce slightly (and the "normal" and "low cost" ranges are also lowered), however some costs reduce by more than others. The relative contribution of household labour is shown in Table 3.4.

Figure 3.10: Tank costs with and without household labour included in costing



Note: Figure 3.10b redrawn from Figure 3.7b for comparison

Table 3.4: Household labour fractions

Tank type	Unskilled household labour (as a fraction of total cost – Uganda)	Unskilled household labour (as a fraction of total labour)
Open frame ferrocement	14%	60%
Pumpkin tank	6%	65%
Thai jar	4%	50%
Tarpaulin tank	4%	70%
Plate tank	2%	50%
Moulded plastic	1%	50%

Labour cost of "local" materials

Another contribution of time rather than cash is in the production or gathering of local materials as will be discussed in Section 4.2.2. For some designs this will be building materials such as sand and gravel (which make up 3-6% of the cost of most cement-based designs), for others it could be thatch and poles. While this does reduce cash cost, it increases the householder's time contribution and so should made explicit in any technology-choice analysis.

3.4.3 EASE OF IMPLEMENTATION

Mobilisation and organisation costs can be significant, particularly in sparsely populated areas or in projects where only a low number of systems are to be built. In these cases technology choice may be dominated, not by tank cost but by ease of implementation. Interviews with water professional revealed one of the main reason agencies use expensive plastic tanks is their ease of implementation – just 'deliver and connect'. Other designs, particularly those with a high householder labour content, will require close supervision throughout the building process.

3.5. Conclusions

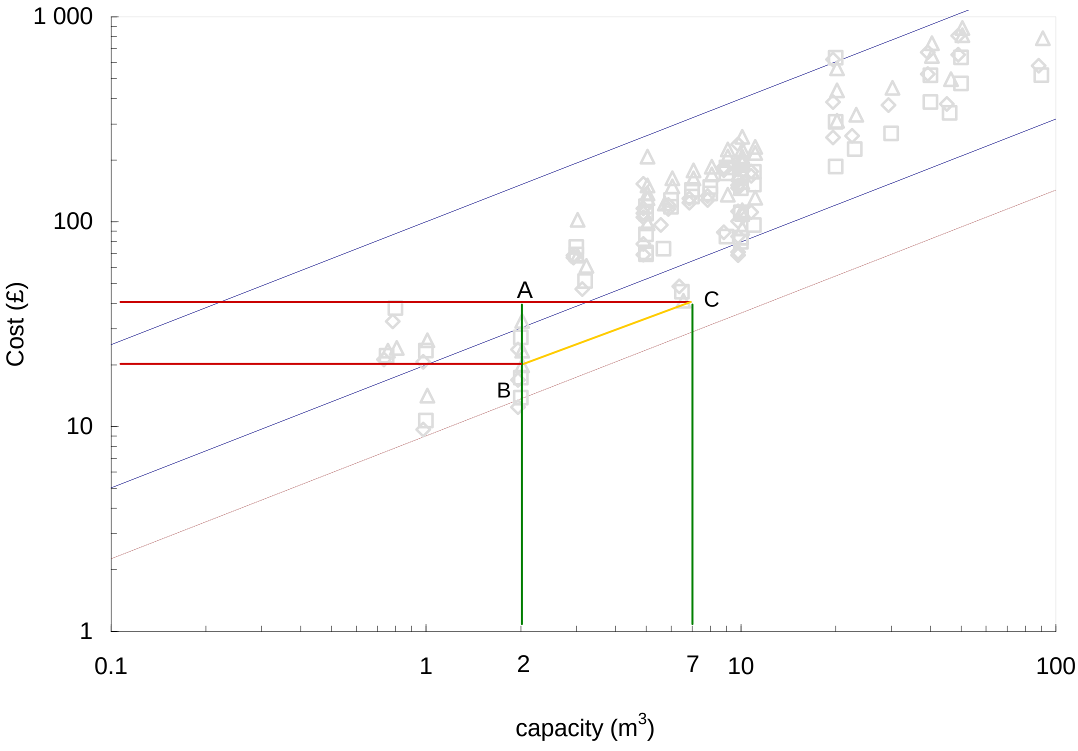
A number of current methods of forming tanks have been investigated and lessons drawn from this experience. The economies of scale involved in tank construction have been considered from theory and derived from an investigation of bills of materials of 60 tanks. The sensitivity of cost to tank volume has been found to be approximately 0.65. Baseline costs have been found from this data and "normal" and "low cost" ranges defined. The economies of scale in tank building offset the diseconomies of scale in water delivery but are not large enough to eliminate or reverse them. Relative labour content has been found not to be sensitive to tank volume but to specific design and to location.

4. REDUCING TANK COST

Having discussed existing rainwater harvesting tanks in Chapter 3, and established a means of cost comparison, this chapter takes the lessons learned from this material and discusses the means by which these costs can be reduced in preparation for the design work presented in Chapters 7 and 8.

Figure 4.1 shows the normal economies of scale of roofwater harvesting storage as described in Section 3.3.1.

Figure 4.1: Effects of cost reduction of rainwater harvesting storage



Considering a $2m^3$ store costing £40 (point A); the capacity and cost of this tank puts it toward the bottom of the "normal" range. If the cost of the tank is halved by a scalable technology, the cost of the $2m^3$ tank will fall into the "low cost" range (point B). The cost reduction can be seen in two ways:

- The tank is cheaper and therefore more affordable for a householder or the cheaper tank allows a water provider to install twice as many systems of the same size
- If the same amount of money were available per household, the normal economies of scale show that householder or provider could afford to buy a $7m^3$ tank with the cheaper technique (point C).

If the larger tank is used, the diseconomies of scale discussed in Section 2.4.6 should be taken into account. A $2m^3$ tank, for example, can provide a Southern Ugandan household[1] with 87% of its water needs, increasing the tank size to $7m^3$ raises this to 98% providing only a 11% increase in return. However with the $7m^3$ tank the longest period when the tank is empty is only 18 days as opposed to 27 days with the $2m^3$ tank. Varying demand reduces this to zero without difficulty whereas extreme measures such as reducing consumption to 20% when the tank is half full must be imposed for the $2m^3$ tank to bridge the dry season.

There are a number of strategies that can be pursued to reduce size-for-size tank cost:

- Streamlining the production process through workshop or mass production
- Using existing containers
- Reducing system quality to a functional minimum

Superfluous tank quality can be reduced by four basic methods:

- Material reduction (using thinner sections, changes in concentration)
- Material substitution (using cheaper materials or "free" materials)
- Changing labour content (moving from bought-in labour to household labour)

[1] Household size assumed to be 5 persons, roof area $30m^2$, Nominal demand 20lcd.

4.1. THE QUALITY DIMENSION

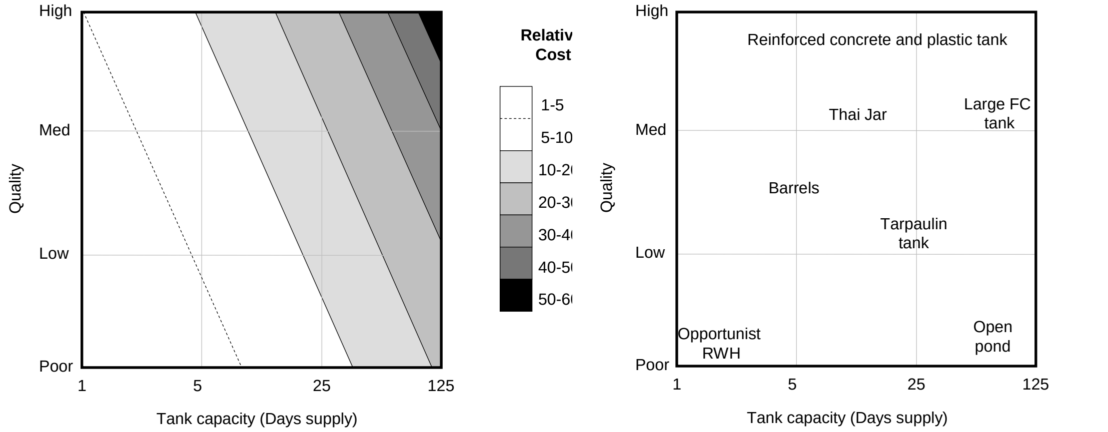
Just as meeting the demand for cheap transport has revealed a multiplicity of solutions from bicycles to three wheeled taxis to buses, a product-oriented approach to rainwater harvesting provision can have multiple answers. The "bus equivalent" is now quite commonplace in East Africa and parts of South Asia in the form of a public tank on a community centre or school. The "bicycle equivalent" is less common and less simple to define.

For some years there has been in existence a "sanitation ladder" [WHO/AFRO, 2001], a catalogue of designs from which a project manager, a community or individual can select an appropriate well-designed sanitation system to suit local conditions and the available funds. A benefit of the ladder approach is a community can adopt technology in stages and "climb the ladder". Such "ranges" are the norm in consumer products and usually form the basis for consumer choice. Rainwater harvesting systems are very amenable to this product range approach, as uncertainties about the location of the water resource don't exist; it simply falls from the sky. They are however, slightly more complex than sanitation systems as their service provision is in two main areas; the *quantity* of water that can be obtained from the system and the *quality* of the system (which includes such things as longevity, ease of extraction, pride of ownership and the engineers need to do a "proper job" – It does *not* necessarily equate to water quality).

It is in fact the system quality aspect that is predominant in the sanitation ladder whereas choices of rainwater harvesting systems are dominated by the question of quantity, usually simply defined by tank capacity, with a certain system quality taken as read. This is especially true for designs created or promoted by water agencies and NGOs who form the mainstay of RWH promotion. Figure 4.2 shows how rainwater-harvesting systems can be mapped onto system quality and tank capacity axes, each with its own demand on resources available to build the system. Generally, rainwater harvesting projects in developing countries are at the medium quality level using materials and techniques taken from the housing sector. Developed countries usually operate with a higher level of sophistication typified by high-tech industrial inputs such as injection moulded parts and electronic monitoring.

Figure 4.2: Service framework for rainwater harvesting systems

a. Framework b. How current systems map onto the framework



It is also interesting to note that the quality of system usually found in poor households (oil drums and traditional practices) is generally lower than current offerings, implying a mismatch between the quality that can be afforded by poor households and the solutions currently being promoted. Similar differences can also be found between housing quality and current systems as can clearly be seen in Figure 4.3. This points the way to achieving the goal of reducing the cost of systems and thereby increasing the quantity of water available from a similarly priced system.

Figure 4.3: Wattle and daub dwelling in Ethiopia with large and expensive ferrocement tank

Lowering construction quality from a high standard initially mainly affects appearance. The next parameter to suffer is durability – cheap materials like wattle and daub walls do not have the durability of portland-cement mortar and will need more frequent renewal. The effect of this on the overall economics of RWH systems is explored in Chapter 5. Finally the point is reached where water quality itself is degraded, for example by omitting the cover. This point a *domestic*

RWH system should not reach unless perhaps it is cascaded to give two outputs, of successively non-potable and potable quality, matching different applications.

In reducing the quality, however, there are a number of critical functional constraints discussed in Section 3.1 that should be regarded as a minimum specification:

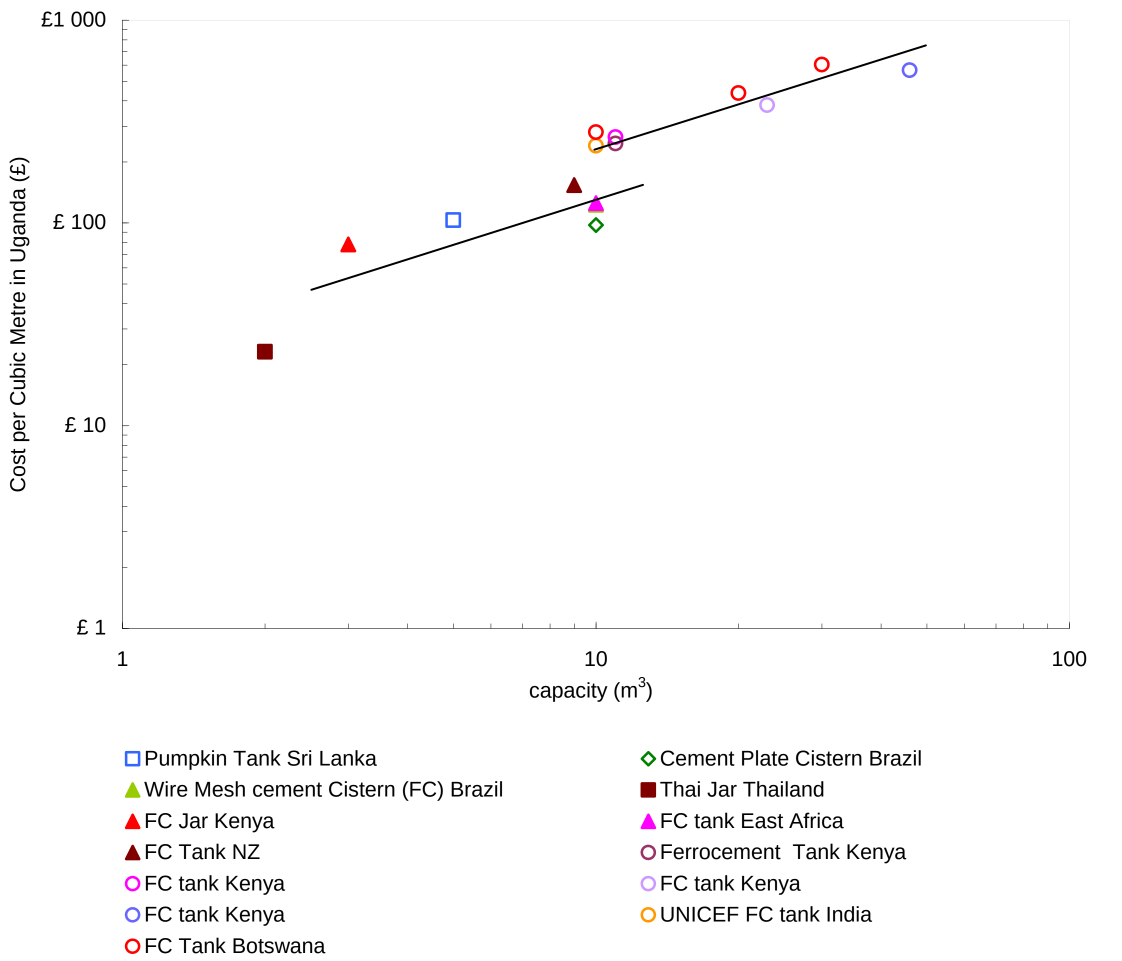
- The tank should not have excessive loss through seepage or evaporation – as compared to the water demand
- The tank should not present an excessive danger to its users, either by their falling in or by the tank failing explosively
- The water must be of a quality appropriate for its intended use – water that is used for drinking requires a certain care in storage:
  - The tank should be covered to prevent entry of light, and sealed against intrusion by mosquitoes and small creatures
  - The tank should be ventilated to prevent anaerobic decomposition of any washed in matter

4.2. STRATEGIES FOR REDUCING COST

4.2.1 MATERIAL REDUCTION

Further analysis of the tanks detailed in Figure 3.7 reveals a number of strategies to reduce material use in tanks. Figure 4.4 shows a comparison of the unit costs of 15 ferrocement tanks based on their bills of materials. From investigating the manufacturing methods of these tanks several strategies emerge.

Figure 4.4: Comparisons of costs of ferrocement tanks



Note: data taken from [LRWHF, 2000], [Gould & Nissen-Petersen, 1999], [Nissen-Petersen & Lee, 1990], [Watt, 1993], [Gnadlinger, 1999], [Wilkinson, 1984] & [Gera, 1999]

IMPROVED FORMWORK

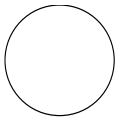
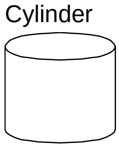
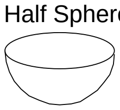
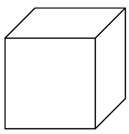
A comparison of tanks with solid formwork (filled triangles) with those made on open frames (open circles) reveals the ability of good formwork to reduce the cost of a tank by about 50%. The lowest material use by far is the Thai jar (filled square) which is built on a cement-block formwork which itself is formed on a factory made template. The formwork provides an excellent working surface and allows tight quality control of wall thickness. Formwork does suffer from a lack of flexibility as each size of tank must have its own form making it difficult to justify the investment unless a large number of similar size of tanks is being contemplated. The ability of solid formwork to help in applying a thin section of mortar is also instrumental in cost reduction through moving the tank underground where the soil can act as a slid formwork.

SHAPE OPTIMISATION

Material economies can be made on water tanks by considering the geometry of surface area to volume and the effects of reducing bending moments in the structure. The Sri Lankan pumpkin tank (whose cost is shown in Figure 4.4 as an open square) and the Thai jar are good examples.

Considering the geometry of surface area to volume, Table 4.1 shows the relationship between various shapes and their respective material economies.

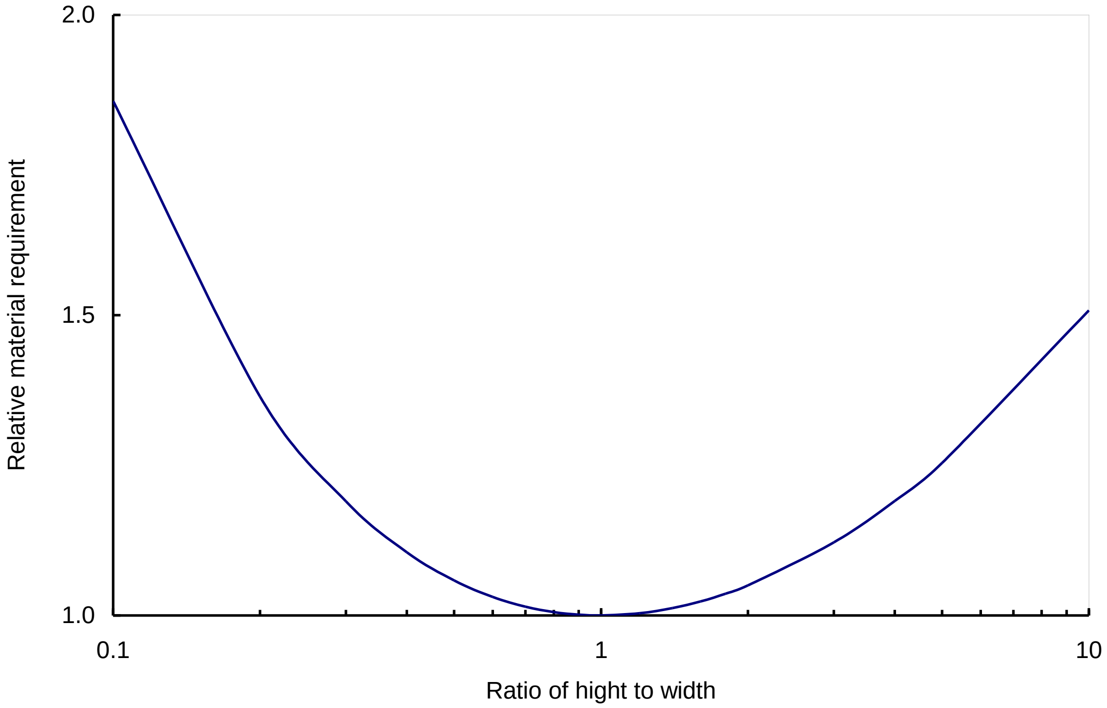
Table 4.1: Idealised tank shapes assuming constant thickness

Shape	Material Penalty	Notes
Sphere 	1.0	• Perfect spheres are only possible underground or partly underground however the shape can be approached using doubly curved structures • Good stress characteristics with little bending stress • All doubly curved structures need great skill or excellent tooling (or both) to manufacture reliably • Only suitable for mouldable materials such as cement and clay or flexible materials such as some textiles and plastic sheeting
Cylinder 	1.2	• The most popular shape for water tanks • Hoop Stresses are efficiently accepted, however a fixed joint between the tank wall and base will cause bending and shear stresses near the joint • Suitable for use with ether mouldable materials or materials which can be bent only in one direction (such as metal sheet)
Half Sphere 	1.3	• A popular shape for underground tanks as the pit is easy to excavate and it is believed to have good material economies • But: Requires a large, free standing cover • Underground tanks are simple to make with this shape using mouldable lining materials
Cube 	1.4	• Bending stresses are high toward the corners • Very simple to construct using familiar house building techniques • Suitable for all materials including bricks and blocks

Highly optimised shapes should, however be balanced against the additional skill required to form them. If skilled labour is inexpensive, they can save money, however when labour is expensive, it may be better to use a simpler shape that is quicker to manufacture. Doubly curved sections also tend to need specialised moulds that should be factored into any cost calculation. The moulds will, however, also yield the benefits from better formwork.

The aspect ratio of height:width can also have an effect on the cost of the tank. As the tank differs from the optimum proportion, material economy suffers. Figure 4.5 shows this relationship for tanks of constant section such as cylinders and cuboids. As can be seen the material diseconomies are stronger in short, wide tanks than in long thin tanks. This is a distinct advantage, as many users prefer to have tanks with a smaller footprint, as land is often at a premium.

Figure 4.5: Effect of Aspect Ratio on the material economies of tanks of constant thickness



Shape optimisation has been further studied in the context of doubly curved tanks and tanks of non-constant thickness in two research projects by Still [2006] and Rennie [2006] which have run parallel to this study. Shape optimisation also forms a part of the strategy used in the design of underground tanks described in Chapters 7 and 8.

4.2.2 FUNCTIONAL SEPARATION AND MATERIAL SUBSTITUTION

Waterproof materials are generally more expensive than non-waterproof materials so cost can be significantly reduced if the quantities of such expensive waterproof materials are reduced. Underground tanks, where the ground itself provides some of the structural strength of the tank are a good example of this. The technique can also be used in conjunction with above-ground structures using earth material such as stabilised soil blocks, rammed earth and even wattle and daub to provide strength, while waterproofing is achieved using a lining of plastic sheet, cement/water slurry or a painted dope.

GREATER USE OF "FREE" MATERIALS

The costs constraints identified by users are predominantly in the realm of cash. Other resources such as time and effort are much more available. An emphasis on using "free" gatherable local materials reduces the need for cash inputs and maximises the available cash resources. This is a standard "appropriate technology" argument but little applied to rainwater harvesting outside of the occasional use of bamboo guttering. Gatherable materials are almost never used in water storage structures as they tend to be based on earth technologies, which are not watertight and often have a lower longevity than modern engineering materials. Yet the housing of the poor almost always make use of these "traditional" materials. Functional separation provides a method whereby this problem can be removed, with the traditional materials doing what they have done for centuries and a small input of a specific engineering material performing the waterproofing function. An excellent example of this is the Tarpaulin tank developed in Southern Uganda which uses an imported tarpaulin to hold the water, while the structure itself is partly underground and partly wattle and daub. More than any other quality reduction strategy, the use of traditional materials will inevitably lead to en elevated need for maintenance, so the overall cost over a lifetime needs to be considered. The economics of this is further explored in Chapter 5 and the strategy is used in a number of designs described in Chapters 7 and 8.

MOVING THE TANK UNDERGROUND

The simple gambit of building the tank under the ground is a popular method of cost reduction in tank building. Foundations problems are avoided completely as the tank is immersed in the supporting soil and so very large tanks can be constructed with relative ease. Nissen-Petersen [Nissen-Petersen & Lee, 1990] has developed a 90m^3 tank in Kenya which has proved popular for schools and public buildings.

Of more interest to the field of Very low cost, tank building is the potential of stable soils to reliably take the force of the water meaning that any cement or render may be needed only as a sealant. Thomas and McGeever [1997] have made several tanks in West Uganda using a 25mm layer of mortar applied directly to the soil with few reported problems after 5 years service. In Ethiopia, a number of tanks have been made using a similar technique with a soil-cement [Nega & Kimeu, 2002], further reducing the requirement for imported material.

In the northern China, the soil is so stable that people simply dig their houses out of the earth. Here, a bottle shaped store known as a "*shuijao*" has been used for centuries. The ground was simply dug out and mud compacted onto the walls. Recently, the government has been improving these tanks using cement lining for improved water retention [Zhu & Wu, 1995].

Failure of underground tanks can be a problem, leaks are difficult to locate and equally difficult to repair. In a study by Ranasinghe [2001], below-ground brick tanks were found to have been holed by tree roots resulting in losses of up to 2.5 litres per day. The other major failure of underground tanks is by the water table raising and empty tanks "floating" out of the ground [Joy, 2001] or simply collapsing under the strain of the outside water [De silva et al., 2001].

Tanks lining the ground with plastics have been tried since the 1970s [Maddocks, 1975], often with little success, however designs such as the Ugandan Tarpaulin tank [Rees, 2000] and the common use of polythene lining for reservoirs and ponds, [Santvoort, 1994] show that the method can be used with careful design. The usual failure modes are tree roots as with other underground tanks, ultraviolet degradation and vermin intrusion. The tanks are, however immune to floatation as they simply flex out of the way. Reports of termites attacking underground plastic sheet tanks are common, however there are equally reports of termites living under the plastic and not damaging it. The matter of UV degradation should be neatly avoided in an underground tank, as given that an appropriate lightproof lid is a requirement for health reasons, the plastic itself should not be exposed to sunlight.

A very inexpensive method of storage is to simply use the soil. In cities such as Delhi, citizens are being encouraged to divert the water from their roofs into the ground to "recharge" the groundwater [Ranade, 2001]. As well as generally improving the groundwater level, there is a localised effect whereby the water forms a "recharge mound" under the recharge point creating a nominal private store (although the water will eventually travel outward). This method is, however often a poor choice for household water supply as it is very dependent upon a slow-flowing, pollution free, relatively shallow water table.

A summary of the advantage and disadvantages of underground and above-ground storage are shown in Table 4.2.

Table 4.2: Pros and Cons of above ground and underground storage

	Pros	Cons
Above-ground	• Allows for easy inspection for cracks or leakage • Water extraction can be by tap driven by gravity • Can be raised above ground level to increase water pressure	• Requires space • Generally more expensive • More easily damaged by accidents • Prone to attack from weather • Failure can be dangerous • Can be completely emptied by carelessly leaving the tap on
Underground	• Surrounding ground gives some support allowing lower wall thickness and thus lower costs (see Chapter 6) • Allows simple former-free shape optimisation • Safety factors can be reduced as failure presents no danger of injury • Some materials (such as mesh in FC tanks) can be omitted (see section 7.3.2) • More difficult to accidentally empty by leaving tap on • Require little or no space above ground • Unobtrusive • Water is cooler • Some users prefer it because "it's like a well"	• Water extraction is more problematic – often requiring a pump, a long pipe to a downhill location or steps to a low cellar • Leaks or failures are difficult to find • Possible contamination of the tank from groundwater or floodwaters • The structure can be damaged by tree roots or rising groundwater • If tank is left uncovered children (and careless adults) can fall in – possibly drowning • If tank is left uncovered animals can fall in contaminating the water • Heavy vehicles driving over a cistern can also cause damage • Cannot be easily drained for cleaning

A number of new tanks described in Chapters 7 and 8 use soil support in their design. The ability of soil to provide structural strength is further investigated in Chapter 6.

4.2.3 MASS PRODUCTION

Significant material and labour savings can be made if products are manufactured in quantity. Buying power of the manufacturer increases and proper workshop practices such as batching and subassemblies can be incorporated reducing labour cost. Mass production can be used for sections of the system such as filters or tank covers as well as complete tanks. The Thai jar (shown in figure Figure 4.4 as a closed square) is the best reported mass-produced tank with 1 and 2m^3 jars produced in a workshop and delivered to households, however tanks can also be made from factory-produced sections and assembled on-site allowing simple and rapid implementation. The cement plate cistern from Brazil (shown in Figure 4.4; as an open diamond) is an example of this method. The sections or components should be of a manageable size and can benefit from high-performance manufacturing practices such as vibrating tables and underwater curing. This form of cost reduction is investigated the parallel project by Still [2006]

4.2.4 USE OF EXISTING CONTAINERS

Many households already hold significant storage in the form of jerrycans, water jars and "oil" drums. These containers can be used in an organised manner to form a small but often significant storage volume. 5 jerrycans contain about 100 litres which itself should provide about 40% of total water needs (assuming $50m^2$ roof, 5 persons consuming 20lcd) or 75% of primary water (drinking and cooking – assumed to total 5lcd). Drums, found in many homes provide a higher level of service and can even be built into a reasonable VLC storage (3 drums can provide 70% of water needs). The use of drums, however also has health concerns, as the drums may have contained toxic chemicals and may not have been cleaned properly. They also need suitable covers and decent water extraction.

4.3. CONCLUSIONS

A number of means to reduce cost are discussed in this chapter along with two strategies of "spending" the cost reduction. Cost reduction can be used for two purposes:

- Reducing storage cost on a per-volume basis
- Increasing storage volume on a per-cost basis

If the increase in volume option is taken, the diseconomies of scale in water supply should be taken into account.

Cost reduction can often be achieved by reduction in system quality which will impact initially only upon appearance, then on longevity and finally on water quality. Domestic systems for which water quality is paramount should stop short of this point and satisfy a number of minimum criteria for tank operation described in Section 3.1.

Potential strategies for cost reduction are:

- Material reduction through improved formwork and shape optimisation
- Greater use of materials which can be gathered rather than bought through functional separation
- Mass production of tanks and components
- Use of existing containers

In combination, these strategies offer scope for the substantial reduction in water storage cost needed to make roofwater harvesting competitive with other modes of water supply.

Several of these techniques will be used in designing a number of low-cost tanks described in the rest of Part 1 of this thesis.

5. THE TRADE-OFFS OF QUALITY REDUCTION:

A NET PRESENT VALUE ANALYSIS OF THE TARPAULIN TANK

5.1. INTRODUCTION

One of the strategies to reduce cost discussed in Chapter 4 was “quality reduction” – the reduction of material specification to those more typically used in housing and other products used by the poor in low-income countries. A danger with this method is that, while capital costs may be reduced, servicing costs could become so high the design becomes unsustainable over time.

An example of this strategy is the Ugandan tarpaulin tank. The design has been in service for a number of years and a fair amount of experience has been gathered in its operation and maintenance. This experience can be used to analyse the cost of the tank over time to see whether the trade-offs inherent in the quality reduction strategy have paid off.

The Net present value (NPV) method where future benefits and costs are discounted to reflect future uncertainty and the preference for the “now” has been used as the magnitude of this discounting on the results can be examined.

5.2. THE TARPAULIN TANK

5.2.1 A BRIEF HISTORY OF THE TARPAULIN TANK

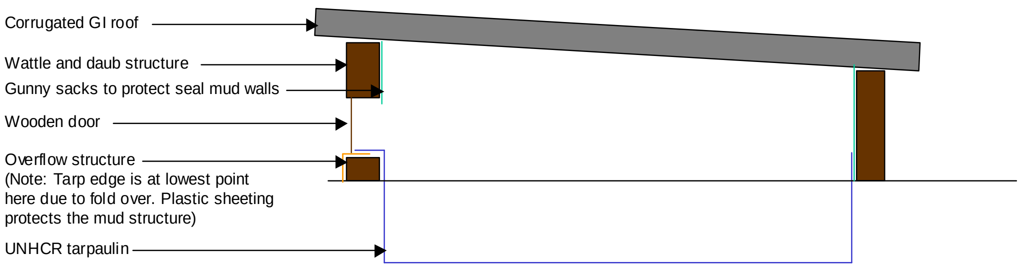
The civil war in Rwanda brought large numbers of refugees into southern Uganda. Many of these refugees settled in the mountains near the town of Mbarara in places such as the Orikinga valley where the water table is far below the surface and can be contaminated with unacceptable levels of iron and manganese [Nakanjakko & Kamugasha Karungi, 2005]. Rainfall in the area is approximately 800 – 1,200 mm/year and follows a fairly even and bimodal distribution (Jan  Dec). This makes the area an ideal candidate for rainwater harvesting.

The Refugees had little capital to buy equipment but the UNHCR had supplied several tarpaulins to be used as shelter. On finding these tarpaulins waterproof, a number of families lined pits with them and used them to collect rainwater. This was successful, however the lined pits were liable to ingress of foreign matter and were open to the sky, allowing algae to develop resulting in a reduction in water quality over time.

A Local NGO (ACORD Uganda) worked with the households to develop an improved design that would allow for increased water quality but retain the low-cost nature of the tank. The improved design featured an enclosure made from wattle and daub with a galvanised steel roof. The enclosure meant that:

- Light and foreign matter were kept out of the tank improving water quality
- The top edge of the tarpaulin could be raised about 10cm to keep ground runoff out of the tank
- An overflow arrangement could be introduced
- Access to the tank was by dipping a half-jerrycan through a wooden door

Figure 5.1: ACORD tarpaulin tank modifications



The volume of the tank was also optimised at this time to take full advantage of the size of the tarpaulin. It was found that with the standard 5 m $\times$ 4 m UNHCR tarpaulin, a structure 3.5 m $\times$ 2.5 m with a depth of 0.75 m resulted in the maximum volume of 6.6 m^3. The shallow depth was also convenient for dipping the half-jerrycan to access water at the bottom of the tank.

In 2000, the Development Technology Unit made some small modifications to the design:

- The access door was replaced with a low-cost pump
- The overflow was diverted to a pipe rather than being sent out of the access door

These modifications were a result of observed contamination of the water from the half-jerrycan which was often placed on the ground after use, carrying soil back into the tank with the next dip. The modified tank was written up as in a DTU technical release [Rees, 2000].

5.2.2 PROBLEMS IN THE USE OF THE TARPAULIN TANK

While a good cheap method of rainwater storage, the tarpaulin tank is not a durable solution in all cases. Numerous tarpaulin tanks have now been in use for several years and while some have been successful, others have seen problems in service. The problems are primarily location related – if the design works in one place in an area, it should work everywhere in that area, if it does not, it will not be suitable for the area.

Problems observed are:

- Termites can damage the wattle and daub frame
- The tarpaulin can rot. It is unclear whether this is due to insects or fungal activity; it does seem to be correlated to soil type however
- The roofing sheets can rust

These problems can be dealt with by various methods:

- The termites can be dealt with using insecticides and barrier methods such as used engine oil. The locations that are suitable for tarpaulin tanks tend to have a high proportion of wattle and daub housing and so preservation methods are well-understood, as are required maintenance routines.

- In locations where the tarpaulin becomes damaged it will require periodic replacement. Anecdotal evidence from users indicates that this should be done about every two years.

- The roofing sheets will require periodic replacement, about every ten years.

These service issues demonstrate a major trade-off inherent in the quality reduction strategy of cost reduction, which is that while the strategy can reduce the up-front cost of a tank, it may result in higher service costs and therefore higher costs overall.

5.3. NET PRESENT VALUE ANALYSIS OF TARPAULIN AND FERROCEMENT TANKS

The long-term performance of the tarpaulin tank allows the economic performance of a quality-reduced tank to be formally analysed. A net present value (NPV) approach has been taken where the return on an initial investment is discounted over time. A minimum requirement for economic viability is that in the absence of any discounting, the NPV should be positive. In a situation where all costs are incurred at start-up, "discounting" the future only reduces the perceived value benefits, so there will normally be a "break even" discount rate above which the NPV goes negative. This is called the internal rate of return (IRR). In situations where some costs are incurred over the life of a system, these too will be reduced by discounting. If two technologies giving equal benefit and having similar total costs are compared, the one with the lower fraction of its costs falling at start-up should generate the higher IRR. The tarpaulin tank is such a technology. The NPV approach allows a comparison between a low-cost, high-maintenance option such as a tarpaulin tank and a higher-cost, low maintenance tank such as a ferrocement design. Comparison between the use of different discount rates also provides some insight into why NGOs often make different technology decisions to the people they serve.

5.3.1 METHODOLOGY

A 20 year NPV analysis was performed on both the tarpaulin tank and an example of a well designed and evolved ferrocement (FC) tank design (the Sri Lankan pumpkin tank – described in Appendix C), based on water collection and household data from Southern Uganda. The results for these two designs were then compared.

To avoid cost bias, the analysis used costs derived from bills of materials and local material costs:

- The 5m^3 pumpkin tank has been made in Sri Lanka for over 10 years and has a well established bill of materials [LRWHF, 2000].

- The materials list for the tarpaulin tank was based on work carried out in Ethiopia when the technology was transferred there as a part of the DFID project [DTU, 2003]. For ease of comparison the 6.6m^3 Ethiopian tank was scaled down to 5m^3 according to a method described in Section 5.3.4

Based on these bills of materials the initial costs of the tanks with unskilled labour being provided by the householder at an opportunity cost of half the unskilled labour rate are detailed in Table 5.1.

Table 5.1: Tank costs

Tank Type	Initial Cost in Sri Lanka	Initial Cost in Uganda
Tarpaulin	£40.50	£35.03
Pumpkin	£67.91	£104.81

To create the revenue stream, the assumptions detailed in Table 5.2 were made about servicing of the tarpaulin tank. Again, householder labour was valued at half the unskilled labour rate.

Table 5.2: Servicing costs for the tarpaulin tank

Servicing task	Frequency (years)	HH Time (days)	Cost in Sri Lanka	Cost in Uganda
Inspect, repair and patch wattle and daub structure	1	2	£1.64	£1.00
Replace tarpaulin	2	1	£9.40	£12.63
Replace GI roof	10	1	£14.35	£10.23
Undiscounted servicing cost (over 20 years)			£155.50	£166.76

The ferrocement tank was assumed to be free of any servicing.

In both countries we can observe:

- The initial cost of the tarpaulin tank is lower than for the pumpkin tank

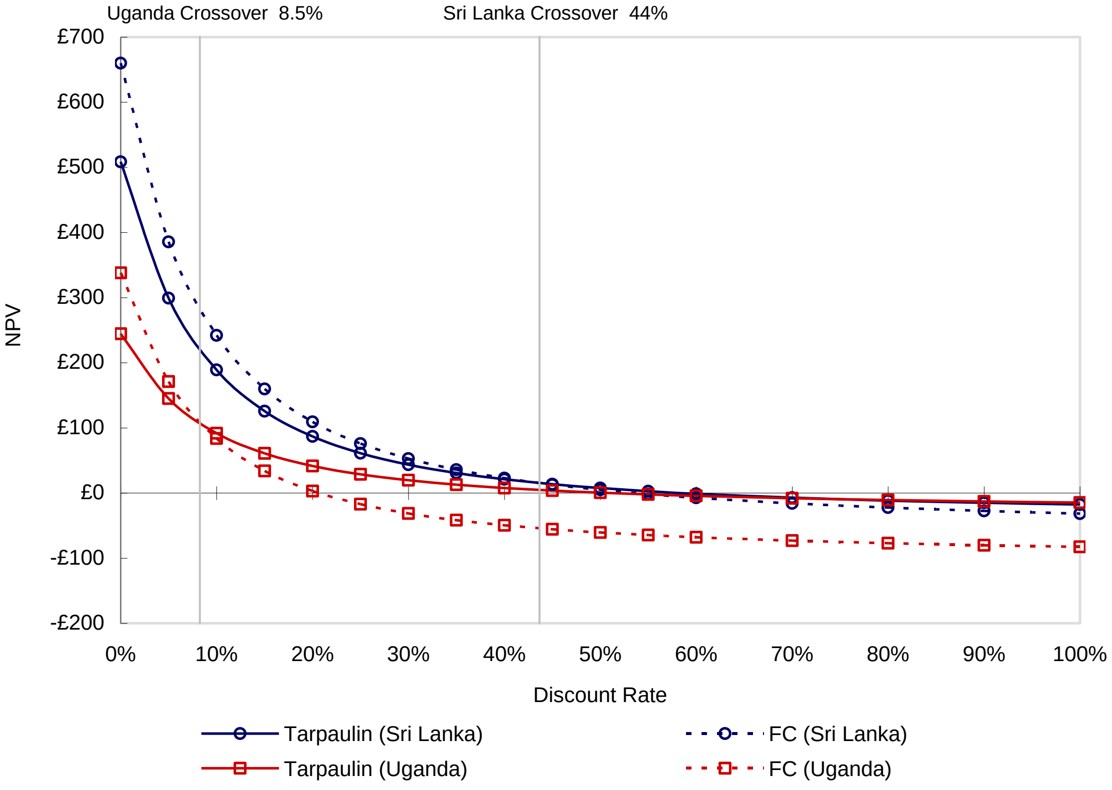
- Total cost in the absence of discounting of the tarpaulin tank is higher than for the pumpkin tank

Increasing the discount rate should reduce the NPV for both designs, but less strongly for the tarpaulin tank than for the pumpkin tank. A *crossover rate* can therefore be defined where both designs give the same NPV. Above this crossover rate, the tarpaulin tank will be more economically attractive; below it the pumpkin tank will be more attractive.

As discussed in Section 2.4.2, the vast majority of the benefit from all rural water systems is in time saving and this is especially true of household water supply like DRWH, so the “income” of the revenue stream is based on time saved fetching water from other sources. Using a mass-balance model and daily rainfall data from Mbarara, a 5m^3 tank provides a mean Ugandan household of 5 [UBOS, 2005] with about 92% the 20 lcd recommended by the Joint Monitoring Programme [WHO & UNICEF, 2006]. Return-journey times were given in the Uganda study in Drawers of Water II [Tumwine, 2002] where the median value was 19 minutes per trip, usually to fetch a 20 litre jerrycan full of water.

5.3.2 RESULTS OF INITIAL ANALYSIS

Figure 5.2: The effect of discount rate used on 20-year net present value of tarpaulin and pumpkin FC tanks built in Uganda and Sri Lanka



The results of the simulations are shown in Figure 5.2. The most economically “appropriate” solution depends on location and also on the discount rate. In the case of Sri Lanka, material costs (particularly of cement) are lower and labour rates are higher than in Uganda, resulting in the pumpkin tank giving higher NPV returns up-to a discount rate of 44% above which the tarpaulin tank becomes more economically viable. The crossover rate does, however take place when the NPV is very low. The Internal Rate of Return of the both tanks is very similar, with the pumpkin tank at 54% and the tarpaulin 58%. Uganda, where material costs are high and labour rates are low, shows a very different situation with the crossover rate much lower at 8.5% discount rate.

NGOs and other agencies use rational economic analysis with discount rates in the order of 3-10% [Hutton & Haller, 2004; Redhouse et al., 2004]. Some even argue that discounting of future benefit in water projects is inappropriate. For these agencies, the FC tank is a better fit to their economic models.

Householders are subject to both real and subjective discounting. Several studies have been done to measure the subjective discount rate used by the rural poor in low-income countries and the result suggest much higher rates underlie household investment decisions One study in Southern India [Pender, 1996] found implicit discount rates of between 26% and 69%, with a median of >50%. Another study in highland Ethiopia [Yesuf, 2004] had a median rate of 43% and a study in rural Thailand found an initial discount rate of between 18% and 67% [Leigh Anderson et al., 2004].

There is substantial variation within studies with the rate varying with principle (higher loans elicited lower discount rates) and over time [Pender, 1996; Frederick et al., 2002; Leigh Anderson et al., 2004], however what is clear is that the future is a very uncertain place to poor householders in low-income countries and they feel a strong need for a quick return on any investment.

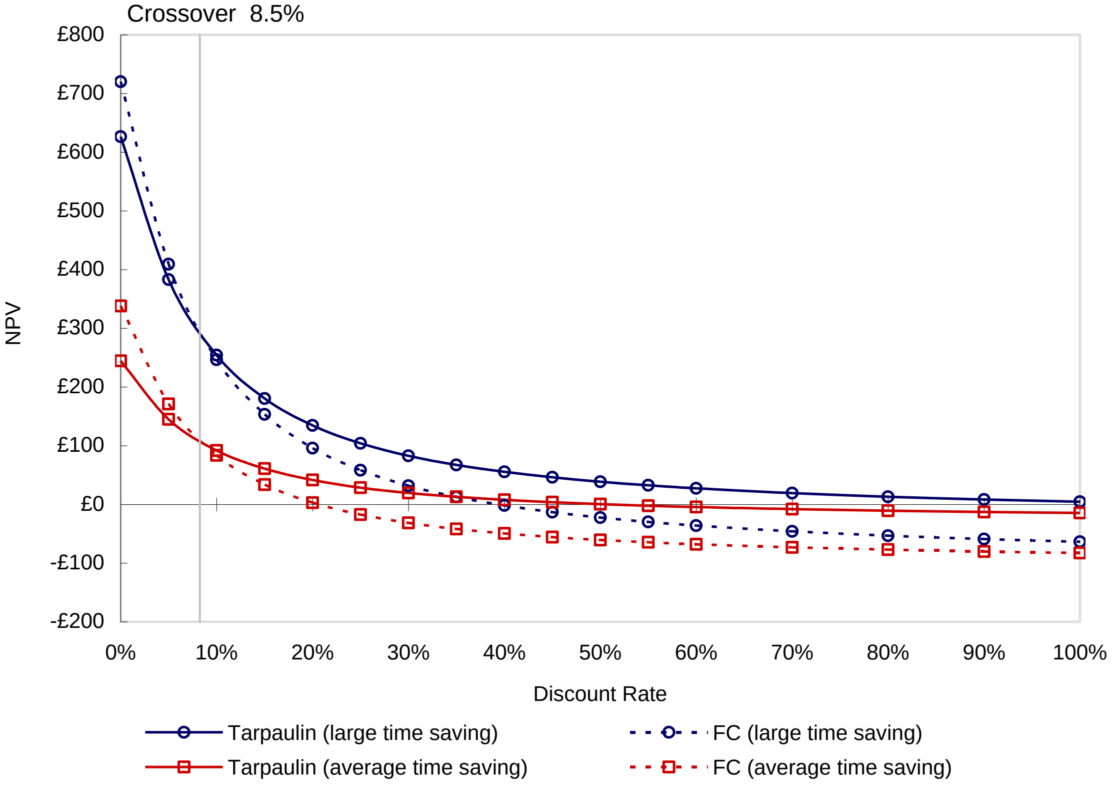
The uncertainty of future returns on investment is also reflected in the somewhat selachian interest rates that are levied on microcredit such as that needed to buy a DRWH system. Ugandan microfinance rates range around 36% p.a. [Oketch, 2006] and more globally can range up to 70% p.a. with commissions, fees and deposits raising the effective rate even higher [Fernando, 2006]. There is some attempt to cap these rates with national policy but this is not supported by the international finance institutions as high rates are deemed necessary by many economists due to the high costs of running a microfinance institution with many small transactions and a high default rate [Fernando, 2006].

In these circumstances, the low-cost, high-maintenance option looks more attractive, indeed the IRR of the pumpkin tank in Uganda is only 21%, above which it gives a negative return whereas the tarpaulin tank provides a positive NPV up to a discount rate of 51%.

5.3.3 SENSITIVITY TO TIME SAVING

In fact, with average Ugandan time saving benefits, the IRRs of both tanks are so low as to make the investment unattractive to many householders unless there is some other reason for the investment such as special-needs like old-age or illness, a high personal value attributed to time, or a desire to increase household's status with a visible asset. DRWH is, however, mostly used where water is particularly scarce. In Kibengo, a water-stressed area in south western Uganda where the DFID study was carried out, the mean household size was 7.8, mean water fetching time is 3.5 hours per day and mean roof size was $34m^2$. Under these circumstances, a $5m^3$ tank can provide 78% of the water demand. When these figures are used in the analysis using the Ugandan cost data, the results are as shown in Figure 5.3.

Figure 5.3: The effect of time saving on NPV-Discount rate curve for Uganda



5.3.4 SENSITIVITY TO TANK SIZE

Scaling the size of a conventional tank design should change capital cost and servicing cost in-line with the economies of scale described in Section 3.3.1, however the benefit side will suffer from the diseconomies described in Section 2.4.6 so the resulting NPV should decrease with increased tank size. The tarpaulin tank can be scaled; however some parts remain fixed both at the build and servicing stages which will result in reduced economies of scale when compared to other designs and change the NPV.

SCALING THE TARPAULIN TANK

The ACORD design was aimed at maximising the water stored per tarpaulin but when the design was also transferred to Ethiopia as part of the DFID research project, one of the main comments was the amount of land area the tank occupied. Keeping the UNHCR tarpaulin whole and increasing the depth makes a smaller-capacity tank than the ACORD design but with a reduced plan area.

The overall cost of the tank is a combination of two sets:

- The fixed cost of the tarpaulin, pump and ancillaries
- The variable cost of walls and roofing which will change with tank size.

The variable costs can be estimated for a tank of any capacity up to the maximum of 6.5 m^3 using some assumptions:

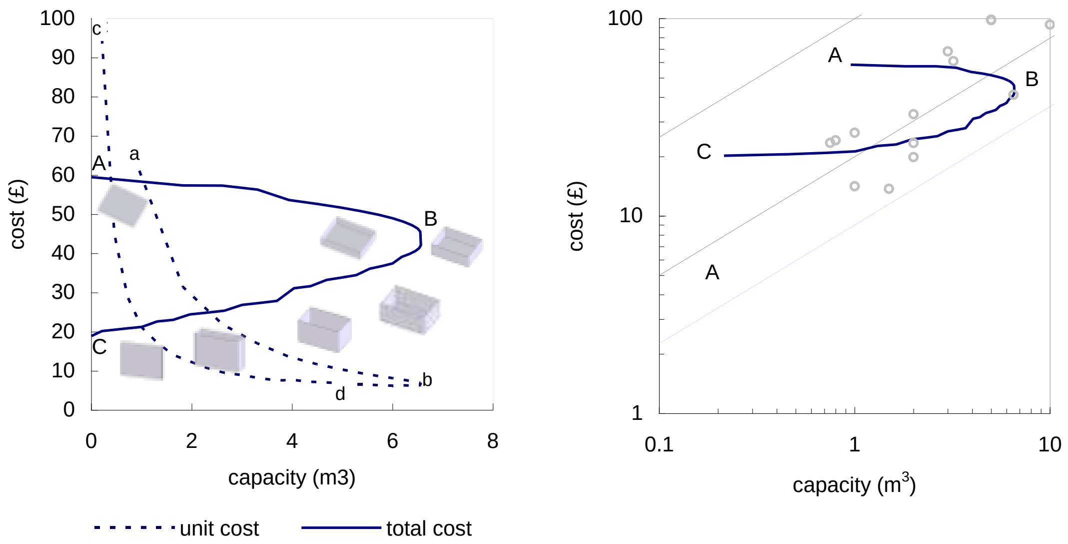
- Hole cost is directly proportional to hole volume i.e. no penalty for digging particularly deep or awkward holes. This assumption was taken in consultation with local labourers who considered most of the proportions resulting from the analysis "reasonable" and the dimensions of the UNHCR tarpaulin, which do not allow a depth of more than 2m.
- Roof cost is proportional to roof area. The roof area was calculated as the plan area of the dug hole plus 25cm overhang all round. Partial roofing sheets were allowed as local traders appeared to sell cut sheets routinely.
- Wall cost is proportional to wall area and thus perimeter length.
- Labour was rounded up to the nearest full day.
- The tarpaulin is only available in one size (5m × 4 m). It is heat sealed along the edge with nylon rope and eyelets in the hem and so should not be cut. Therefore as the long length (L) is varied, the depth (H) will follow $H = (5 - L)/2$ and the breadth (B) will follow $B = L - 1$.

When these assumptions are used to change the volume and cost of the tarpaulin tank using the experience of the Ethiopian build and ensuing bills of materials as a starting point, the costs are calculated as shown in Figure 5.4.

Figure 5.4: Loci of calculated Ugandan tarpaulin tank’s costs and volume with changes in aspect ratio

a. With illustrative aspect ratios

b. With benchmark cost ranges



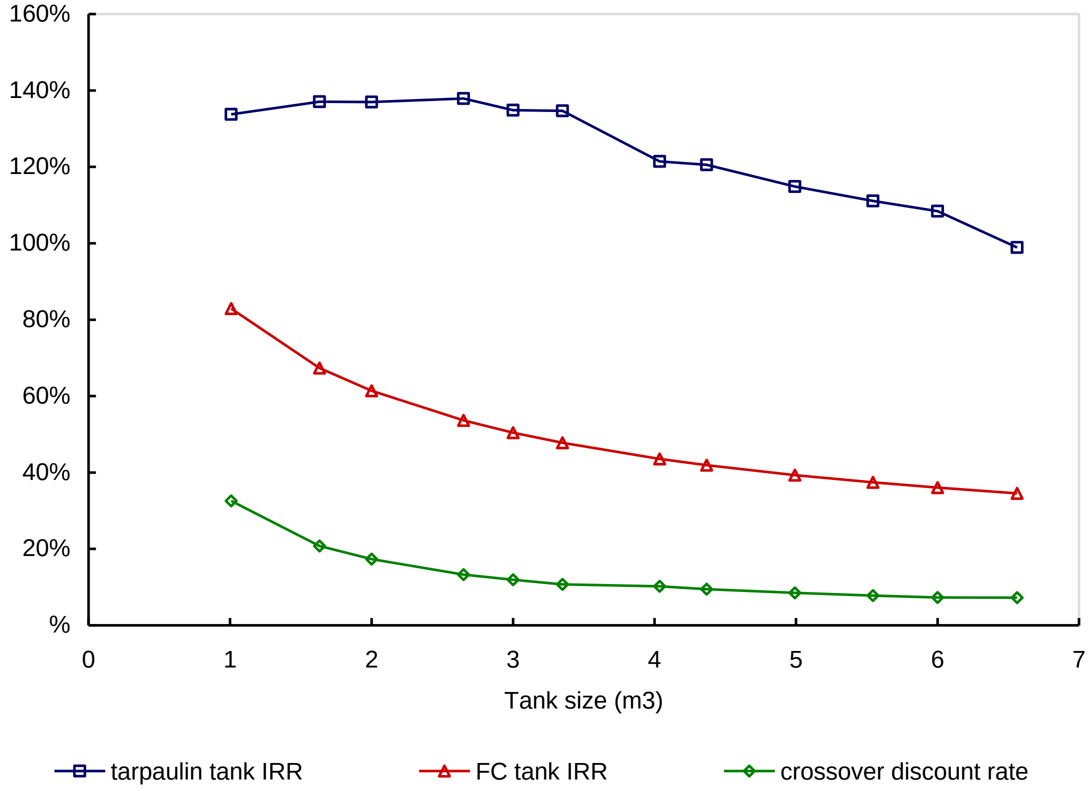
The series starts at A with the tarpaulin flat on the ground providing zero storage and the maximum roof and wall area (and therefore cost). As the aspect ratio changes with deeper, narrower tanks, the volume increases and the cost decreases with lower wall and roof areas. This continues until at B the volume reaches a maximum of 6.5m^3, however while the volume now decreases with hole depth, so too does the cost. The progression ends with another zero volume tank with a depth of 2m and no area (the costs arise from the roof overhang and the remaining perimeter). Turning to the unit cost, the cost per m^3 falls quickly until the maximum volume at b and then stays quite flat between 6.5 and 4m^3 with a minima reached at d with a volume of just under 5m^3. Figure 5.4b shows the costs superimposed on the chart of benchmarks and ranges from Figure 3.7s. The range of costs are in the low-cost range for tanks between 1m^3 and 6.5m^3 showing that based on up-front cost alone, tarpaulin tanks are cost competitive over a wide range of sizes.

Servicing costs will show a similar break-down to capital costs with the tarpaulin replacement cost remaining fixed, while the roof and wall repairs will change with tank size.

NPV ANALYSIS

Figure 5.5 shows the IRR of tarpaulin and ferrocement tanks in Kibengo as size is changed along with the discount rate at which the tarpaulin tank becomes the more attractive. The tarpaulin tank cost scaling uses the assumptions described above while the ferrocement tank, cost is scaled using in proportion to $V^{0.65}$ as described in Section 3.3.1.

Figure 5.5: The effect of changing tank size on internal rate of return and crossover discount rate of NPV curves for FC and tarpaulin tanks in Southern Uganda



The ferrocement design shows a reduction in IRR as tank size increases as would be expected considering the tank performance diseconomies discussed in Section 2.4, but the tarpaulin tank shows a different behaviour with the IRR increasing slightly until 2.5m^3 and then falling away but overall remaining flatter than the FC tank. The reason for this is that in contrast to a conventional design, where the increase in benefit as tank size increases is offset by a greater increase in cost, the tarpaulin itself remains a fixed cost regardless of change of size resulting in a reduced net benefit (= benefit – cost) for the years when this component is replaced.

The crossover rate shows a fairly consistent fall as tank size increases despite the changeable nature of the tarpaulin tank's IRR, displaying the increased effect of the high relative maintenance cost and reduced economies of scale of the tarpaulin tank. It does, nevertheless remain below the range of discount rates used by low-income households over all tank capacities.

5.4. CONCLUSIONS

The NPV method provides a useful tool to compare tanks with different capital and servicing costs over a period of time. When NPV curves are plotted over a series of discount rates a *crossover rate* is revealed above which the reduced-quality design becomes more competitive than the high-quality design. The NPV method can also be used to discover when a tank has too high a servicing cost to justify its capital cost saving, though this level will vary by type of customer; NGOs and other water agencies using low discount rates will tend to favour high capital – low maintenance designs while poor householders may opt for lower capital – higher maintenance designs that better suit their money-flow and future outlook. Overall, the lower the crossover rate, the wider the market for a new design should be, though, for poor householders a better return at, say 50% discount is more attractive than a low crossover rate. In evaluating new designs both these measures will be considered.

In the geographical areas considered, Sri Lanka tended to favour ferrocement designs as labour is expensive and materials inexpensive. Uganda, on the other hand has relatively expensive materials and cheap labour and so favoured the tarpaulin design when a discount rate of more than 10% is applied. This situation was true over a wide range of capacities and the crossover rate is lower than 20% from $1m^3$ to $6.5m^3$.

The case of the tarpaulin tank is an extreme one, as it requires the regular replacement of an item that represents a high proportion of its total cost, yet it remains competitive over a wide range of capacities when the future is discounted fairly highly – so reduced-quality designs can be competitive over time. It is, however instructive from a design point of view as it shows the worth of reduced fixed costs in scalable designs and the value of considering any frequently-replaced part to ensure that it does not form a large fraction of the total cost.

6. ANALYSIS OF SOIL SUPPORT FOR UNDERGROUND TANKS

One strategy of cost reduction described in Section 4.2.2 of this thesis is Functional Separation by moving the tank underground.

Usual civil engineering best-practice design of underground tanks, particularly in high-income countries, is based around a “worst case” scenario where the soil is considered fluid, saturated and inward soil forces dominate. This design approach produces underground tanks that are actually stronger than their above-ground counterparts. The assumption of fluid and saturated soil is, however, often untrue where rainwater harvesting is used in low-income countries. Rainwater harvesting is usually used when other sources are uneconomic to develop which usually means that the water table is far below the ground – otherwise very cheap hand-dug wells would likely be used.

When this is the case, building storage tanks underground can been used as a method to reduce tank cost. The soil can provide some of the support needed to contain the water reducing the material needed to contain the pressure or, in extreme cases, allow the tank material to merely provide a waterproof seal. While this system works well when a flexible material such as a tarpaulin is used, when typical masonry building materials such as pointed bricks or cement mortar are used, the tank can crack due to excessive soil movement before the total load is transferred to the soil. It is therefore necessary to determine how much support for the tank wall can be expected from the soil. The problem is not (or is rarely) that the soil cannot withstand the pressure of the water – it is that as the soil accepts the pressure, it will move according to its elastic modulus. Meanwhile the tank wall will move according to its elastic modulus and the two must act together. The modulus of soil is several orders of magnitude lower than that of masonry building materials and so only a portion of the force will be transferred to the soil.

This chapter concerns itself with analytical and finite-element solutions to the problem of just how great a material reduction can be expected by using soil to support the tank walls.

6.1. ANALYTICAL SOLUTIONS

An initial estimate of available support for an underground tank can be gained by considering a simple loop of material supported by an elastic soil foundation.

Ghali [2000] presents an analysis of circular tanks that simplifies the problem to one of a rectangular beam on an elastic foundation. Consider an unsupported cylindrical shell subject to an axially symmetric load (such as a water tank). Hookes law shows that the strain (δ/r) will be related to the hoop stress[1] by Equation 6.1:

$$\sigma_h = E\frac{\delta}{r} \qquad \text{Equation 6.1}$$

Where: σ_h is the hoop stress (Pa); E is the young's modulus of the tank material (Pa); r is the radius of the tank (m); δ is the deflection (m)

Combining this with the equation for hoop stress, the reaction pressure caused by the deflection can be obtained:

$$p_r = -\frac{\sigma_h t}{r} = -\frac{Et}{r^2}\delta \qquad \text{Equation 6.2}$$

Where: p_r is the reaction pressure (Pa); t is the thickness of the wall (m)

Under conditions of static equilibrium, the reaction pressure will match the water pressure. This is analogous to the reaction of an elastic foundation where $\Delta\delta \propto \Delta p_r$, so:

$$k = \frac{p_r}{\delta} = \frac{Et}{r^2} \qquad \text{Equation 6.3}$$

[1] Note: Ghali used hoop *force* (N/m) in his analysis rather than hoop *stress*, however the outcome is the same

Where: k is the modulus of the foundation (N/m^3).

The analogous soil property is described in Bowles [1996] as the "modulus of subgrade reaction" (MOSR) and is used in footing design. It is a function of the elastic modulus and Poisson's ratio of the soil.

$$k = \frac{E_s}{B\ 1-\mu^2} \qquad \text{Equation 6.4}$$

Where: B is the width of the foundation (m); E_s is the elastic modulus of the soil (Pa); μ is Poisson's ratio of the soil

Bowles also describes changes which make the soil more rigid with depth. These are measured using depth factors that are important to foundation engineering which involves *vertical* loading to the ground. Depth factors complicate the analysis, particularly as they are non-linear and their use involves lookup tables, however underground tank loadings are primarily, *horizontal*, so they may be discounted. Moreover, as the load on an underground tank is highest toward the bottom, any analysis based on a modulus measured at the soil surface can be considered conservative.

Given the similarity between the MOSR and Ghali's foundation modulus, Ghali's analysis can be carried further to include the soil support as a secondary elastic foundation. As both wall and soil must move together, the deflections will be identical so:

$$\delta = \frac{p_{rt} r^2}{E_t t} = \frac{p_{rs}}{k}$$

Hence:

$$\frac{p_{rt}}{p_{rs}} = \frac{E_t t}{k r^2} \qquad \text{Equation 6.5}$$

Where: p_{rt} and p_{rs} are the tank wall reaction pressure and soil reaction pressure respectively.

The total reaction will partly be provided by the tank wall and partly by the soil. As all forces must balance for equilibrium, the combined reactions must be equal to the water pressure:

$$P_w = p_{rt} + p_{rs}$$ Equation 6.6

Where p_w is the water pressure.

Combining Equation 6.5 and Equation 6.6 gives the fraction of the water pressure taken by the tank wall:

$$\frac{p_{rt}}{p_w} = \left(\frac{kr^2}{E_t t} + 1 \right)^{-1}$$ Equation 6.7

As the wall of the tank will only have to support this fraction of the pressure load, it follows that the equation will be the same for the fraction of stress in an unsupported wall (σ_u) to the stress on a wall supported by soil (σ_s) and the ratio of material thickness needed for an unsupported wall (t_u) to that the thickness required for a supported wall (t_s).

$$f_\sigma = f_t = \frac{p_{rt}}{p_w} = \frac{\sigma_s}{\sigma_u} = \frac{t_s}{t_u} = \left(\frac{kr^2}{E_t t} + 1 \right)^{-1}$$ Equation 6.8

Where: f_σ and f_t are the stress and thickness fractions respectively.

Solving for the supported tank thickness gives;

$$t_s = t_u - \frac{kr^2}{E_t}$$ Equation 6.9

Revealing a simple subtractive relationship where the soil support contributes the equivalent of a fixed thickness of material regardless of material strength or applied load:

$$t_c = \frac{kr^2}{E_t}$$ Equation 6.10

Where (t_c) is the soil contribution.

If this thickness is less than the required thickness for an above-ground tank, it is subtracted from the calculated material thickness as shown in Equation 6.10, if it is greater than the required thickness the ground can provide *all* of the necessary support. In this case, the tank material has no structural purpose and is purely for waterproofing; the thinnest layer that can reliably be applied can therefore be used.

6.2. FINITE ELEMENT ANALYSIS OF CYLINDERS

To test the solutions derived in Section 6.1, results for a number of tank configurations were compared with those found from finite element (FE) analysis using COSMOSWorks software. COSMOSWorks has an "elastic support" connector feature which mimics an elastic foundation removing the need to model the soil explicitly. To conserve computer time, only a 20° segment was analysed with "symmetry" constraints on its section faces to simulate a complete tank.

To obtain the stress fraction, two sets of simulations were performed, the first with no support and the second with an elastic support connector with the same value as the MOSR. The maximum first principal stress results from the simulations *with* "soil" support were divided by the maximum stress results of the simulations *without* support to give the simulated stress fraction ($f_{\sigma,s}$). This fraction was compared to the calculated stress fraction ($f_{\sigma,c}$) resulting from the stress version of Equation 6.8. An error ratio between FE simulated and calculated stress fractions was expressed as:

$$R_{s/c} = \frac{f_{\sigma,s}}{f_{\sigma,c}} \qquad \text{Equation 6.11}$$

6.2.1 RING

Initial analysis was carried out using the simplified case of ring of material with a constant pressure load applied to the inside surface as a simplified model of the water pressure. The FE model was tested with the range of dimensions and material properties detailed in Table 6.1. Five of the six values were held constant at their default values while one was changed to each of its alternative values in turn. Simulations were repeated until all combinations of default and variable values were tested. Later simulations introduced a load that varied linearly with depth to simulate the change in pressure with depth that occurs in a normal water tank. The results of these simulations are shown in Table 6.2.

Table 6.1: Parameters varied in initial FE analysis

Tank Young's modulus	Soil modulus	Tank radius	Tank height	Tank wall thickness	Applied load	Applied depth-varying load
Pa	N/m3	mm	mm	mm	Pa	Pa/m depth
Default values						
2.0E10	5.0E7	1 000	1 000	10	10 000	-
Alternative values						
2.0E08	5.0E5	100	100	1	1 000	1 000
2.0E12	5.0E9	10 000	10 000	100	100 000	100 000

Table 6.2: Results of initial FE analysis (alternative conditions are in shaded bxes)

Tank Young's modulus	Soil modulus	Tank radius	Tank height	Tank wall thicknes s	Applied Load	Applied Varying Load	Simulate d fraction ($f_{\sigma,s}$)	Calculat ed fraction ($f_{\sigma,c}$)	Error ratio ($R_{s/c}$)
Pa	N/m^3	mm	mm	mm	Pa	Pa/m depth			
2.00E10	5.00E07	1 000	1 000	10	10 000	-	0.798	0.800	99.8%
2.00E08	5.00E07	1 000	1 000	10	10 000	-	0.040	0.038	105.3%
2.00E12	5.00E07	1 000	1 000	10	10 000	-	0.998	0.998	100.0%
2.00E10	5.00E05	1 000	1 000	10	10 000	-	0.997	0.998	99.9%
2.00E10	5.00E09	1 000	1 000	10	10 000	-	0.039	0.038	102.6%
2.00E10	5.00E07	100	1 000	10	10 000	-	0.997	0.998	99.9%
2.00E10	5.00E07	10 000	1 000	10	10 000	-	0.039	0.038	102.6%
2.00E10	5.00E07	1 000	100	10	10 000	-	0.806	0.800	100.8%
2.00E10	5.00E07	1 000	10 000	10	10 000	-	0.800	0.800	100.0%
2.00E10	5.00E07	1 000	1 000	1	10 000	-	0.287	0.286	100.3%
2.00E10	5.00E07	1 000	1 000	100	10 000	-	0.972	0.976	99.6%
2.00E10	5.00E07	1 000	1 000	10	1 000	-	0.798	0.800	99.8%
2.00E10	5.00E07	1 000	1 000	10	100 000	-	0.798	0.800	99.8%
2.00E10	5.00E07	1 000	1 000	10	-	1 000	0.798	0.800	99.8%
2.00E10	5.00E07	1 000	1 000	10	-	100 000	0.798	0.800	99.8%

The results show a deviation from the calculated fraction of less than 3% over several orders of magnitude of all parameters demonstrating a good agreement between Equation Equation 6.8 and finite element simulation. Neither changing the magnitude of the load nor changing it to a depth-varying load had any effect on the results.

6.2.2 CYLINDER WITH A FIXED BOTTOM EDGE

Flat bottomed tanks complicate the analysis somewhat as they have a bending moment component as well as the simple hoop stress described in Section 6.1. Ghali has shown that Poisson's effect causes this moment to increase the apparent analogous foundation modulus so an increase in tank contribution should be expected.

The error fraction from Equation 6.8 for a cylinder was found by performing FE simulations on a model of a cylindrical shell using the parameters shown in Table 6.3. As results have proved insensitive to loading, this was not varied in the FE analysis; a linearly-varying load of 10 kPa/m depth was used to simulate water pressure. Again, two sets of simulations were performed, the first with no elastic support and the second with support. Within each set of simulations one set of parameters were “defaults” and were held constant while the alternative values were changed. The alternative values ranged over several orders of magnitude but greater attention was given to the range of sizes actually found in domestic rainwater tanks (the values in the shaded boxes).

Table 6.3: Parameters varied in FE analysis

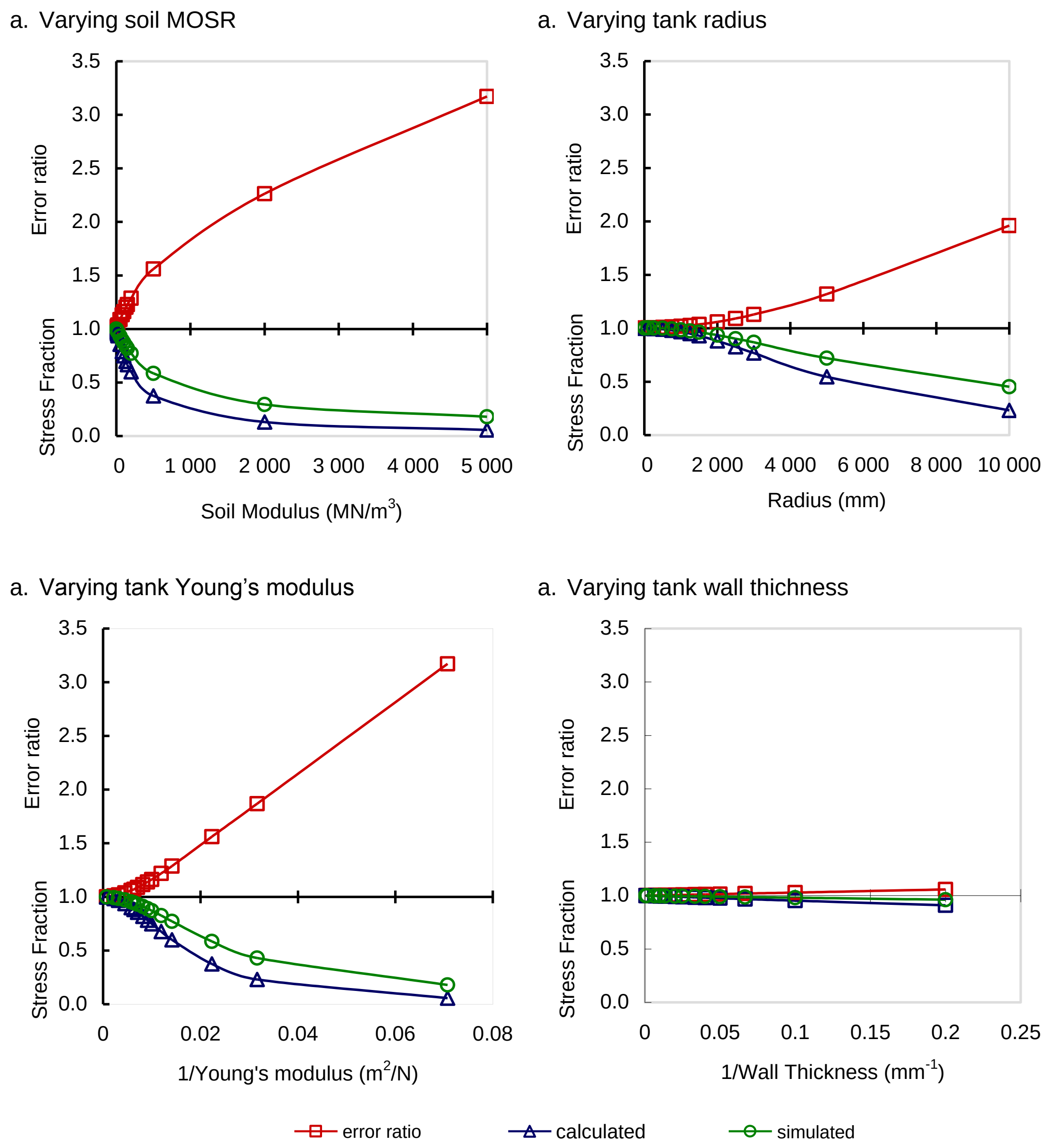
Tank Young's modulus	Soil modulus	Tank radius	Tank height	Tank wall thickness
Pa	N/m^3	mm	mm	mm
Default values				
2.5e10	5.0e7	1 000	2 000	15
Alternative values				
2.0e08	5.0e5	10	20	1
1.0e09*	2.0e6	100	200	2
2.0e09	5.0e6	250	500	5
5.0e09	1.0e7	500	750	10
7.0e09	1.5e7	750	1 000	15
1.0e10	2.0e7	1 000	1 250	20
1.2e10	5.0e7	1 250	1 500	25
1.5e10	8.0e7	1 500	1 750	30
2.0e10	1.0e8	2 000	2 000	40
2.5e10**	1.3e8	2 500	2 500	50
3.0e10	1.5e8	3 000	3 000	75
5.0e10	2.0e8	5 000	4 000	100
1.0e11	5.0e8	10 000	5 000	150
2.0e11	2.0e9		10 000	750
2.0e12	5.0e9		20 000	1 500

* HDPE
** 1:3 mortar

The FE model was constrained to be “fixed” at the base to simulate a join to an immovable base resulting in a bending moment.

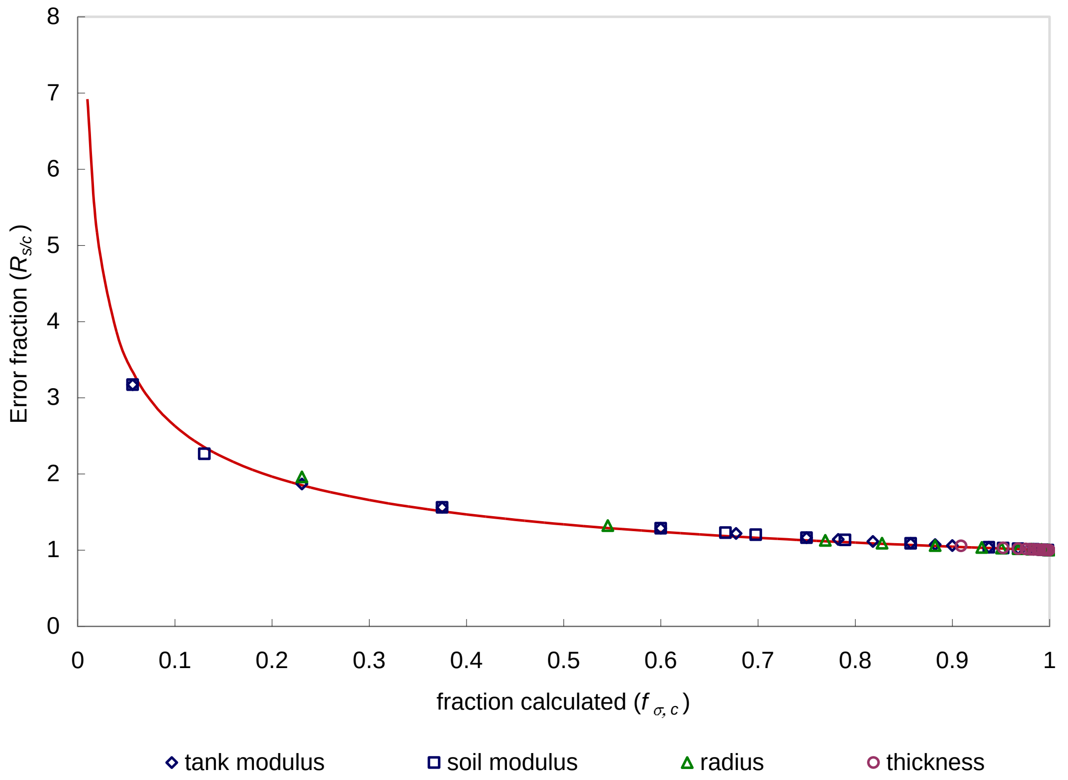
The simulated and calculated stress fractions ($f_{\sigma,s}$ & $f_{\sigma,c}$) were found and the error ratio calculated as before. The stress fractions error ratios are plotted in Figure 6.1. unlike the hoop results, the error ratios ($R_{s/c}$), are not negligible with the simulated results displaying higher stress fractions than those derived from Equation 6.8. This is not unexpected as the equation does not take bending moments or Poisson's effect into account

Figure 6.1: Simulated and calculated stress fractions for cylindrical tanks and the ratio of simulated to calculated results



The ratio is quite systematic and very close to a scaled inverse of the individual elements of Equation 6.8. In fact when the error fraction is charted against the calculated fraction, the data collapses into a single line as shown in Figure 6.2.

Figure 6.2: Error ratio based calculated value



This line corresponds to:

$$R_{s/c} = f_{\sigma,c}^{\ \ -c} \qquad \text{Equation 6.12}$$

where c is a constant. This equation can be developed into a correction that can be applied to Equation 6.8.

$$f_{\sigma} = f_{t} = \frac{p_{w}}{p_{rt}} = \frac{\sigma_{u}}{\sigma_{s}} = \frac{t_{u}}{t_{s}} = \left(\frac{kr^{2}}{E_{t}t} + 1\right)^{-c} \qquad \text{Equation 6.13}$$

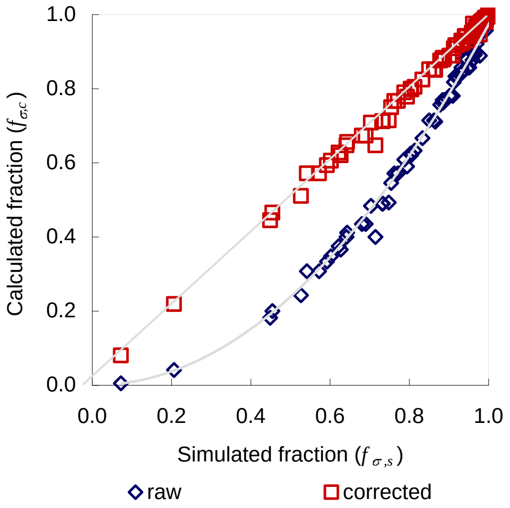
Equation 6.13 was checked against 100 simulations with parameters from Table 6.3 applied in random combinations. The combinations were slightly constrained to avoid extremely unlikely combinations by applying the following conditions:

- The ratio of height to wall thickness was constrained to between 10:1 and 1000:1
- The ratio of diameter to height was constrained to between 1:10 and 10:1

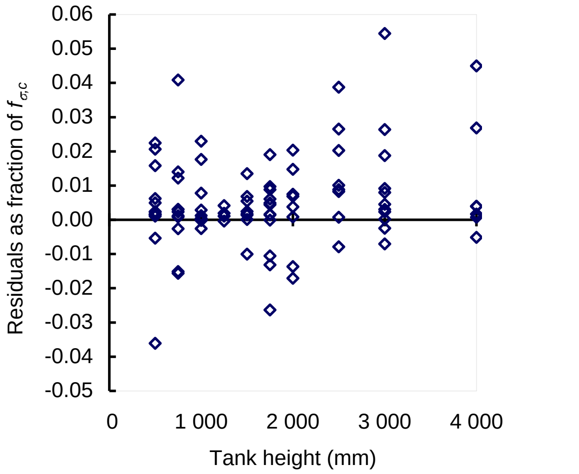
The results of the simulations and calculations are shown in Figure 6.3.

Figure 6.3: Raw and corrected values for stress fraction

a. raw and corrected calculated stress fractions compared to the simulated stress fraction.



b. Residuals of corrected calculated stress fraction from simulated fraction compared to tank height.



For cylindrical tanks, a constant c of 0.467[1] gives the best least squares fit with the data and gives a R^2 of 0.997, predicting the finite element results +6%,-4% across all combinations tested. The deviation also showed little correlation with tank height.

6.2.3 APPLICATION OF CORRECTED EQUATION TO WATER TANKS

Having derived Equation Equation 6.13 and using the correction factor of c = 0.467, it can now be used to calculate the thickness (and therefore material) reduction afforded by soil to typical sizes and shapes of water tanks.

The US Defence Department publishes a table of moduli of the subgrade reaction to be used in pavement and road design, as well as a description of how to perform an on-site estimate [Unified Facilities Criteria, 2004]. The table is reproduced below (converted from "English" units).

[1] This correction factor is extremely close to 0.5, so the R.H.S. of Equation Equation 6.13 could be simplified to

$$\sqrt{\frac{kr^2}{E_t t}+1}$$

Table 6.4: Modulus of the subgrade reaction of soils

Moisture content (%)	1-4	5-8	9-12	13-16	17-20	21-24	25-28	>28
Soil Type	Modulus of the subgrade reaction (in N/m^3)							
Silts and clays, LL* greater than 50 (OH, CH, MH**)		4.8E7	4.1E7	3.4E7	2.7E7	2.0E7	5.0E6	7.0E6
Silts and clays, LL less than 50 OL, CL, ML)		5.4E7	4.8E7	4.1E7	3.4E7	2.7E7	2.0E7	1.4E7
Silty and clayey sands (SM, SC)	8.1E7	6.8E7	6.1E7	5.4E7	4.1E7			
Sand and gravelly sands (SW, SP)	9.5E7	8.1E7	6.8E7					
Silty and clayey gravels (GM, GC)	1.1E8	9.5E7	8.1E7	6.8E7				
Gravel and sandy gravels (GW, GP)	1.4E8	1.2E8						

Notes:
* LL refers to the liquid limit
** The two letter codes refer to the US Unified Soil Classification System (ASTM D 2487)

The figures in Table 6.4 can be used to define the values for k for “loose” (k = 1E7 N/m^3) “medium” (k = 5E7 N/m^3) and “firm” (k = 1E8 N/m^3) soils. When these are used in Equation 6.13 with the dimensions and thicknesses from Table 3.1, the rather disappointing results in Table 6.5 are revealed. In none of these cases is the material reduction larger than 20% and in “medium” soil not a great deal higher than 10%.

Table 6.5: material savings for walls of cylindrical tanks by using soil support

Capacity (m^3)	Dimensions H:D (m)	Calculated thickness (mm)*	Material saving		
			Firm soil (k = 1E8 N/m^3)	Medium soil (k = 5E7 N/m^3)	Loose soil (k = 1E7 N/m^3)
1	1.2:1.1	3.5	15%	9%	2%
2	1.4:1.5	5.5	18%	10%	2%
5	1.8:1.8	9.5	15%	9%	2%
10	2.0:2.5	13.8	19%	11%	3%

* No safety factors are included in these figures

6.3. FINITE ELEMENT ANALYSIS OF DOMED BOTTOM TANKS

Although cylindrical underground tanks can be found (e.g. the Sri Lankan underground brick tank [Heijnen & Mansur, 1998] and the (backfilled) Brazilian plate tank [Gnadlinger, 1999]), a primary advantage of building a tank underground is that there is, in fact, no need to constrain the design to a cylinder. Shapes can be used that are more material efficient than cylinders and designs that will be dominated by membrane stress rather than bending moments when under load can be considered at little or no extra labour cost or organisational complexity. The soil usually forms a good, solid formwork against which to work and, if the soil is coherent enough, almost any shaped hole can be dug (e.g. bottle-shaped tanks in loess soil in Northern China [Zhang et al., 1995]).Few soils are, however, coherent enough to support an overhang so the most commonly applicable underground designs will have their widest point at ground level and tend to narrow towards the bottom. In practice the most practical shape is that of a cylinder with an inverted dome (antidome) at the bottom as the antidome can be fairly simply approximated by using a stake and taut string to define its radius.

When comparing tanks with a domed bottom to cylindrical tanks, it is useful to define a tank of equivalent volume and aspect ratio. This can simply be achieved by using an "equivalent depth" which is the overall depth of a dome-bottomed tank whose volume is the same as a cylindrical tank of the same diameter

$$\pi r^2 h_\mathrm{c} = \pi r^2 \left(h - r \right) + \frac{1}{2} \times \frac{4}{3} \pi r^3$$

Where: h is the overall dome-bottomed tank depth; r is the tank radius; h_c is the depth of a cylindrical tank of the same volume and radius.

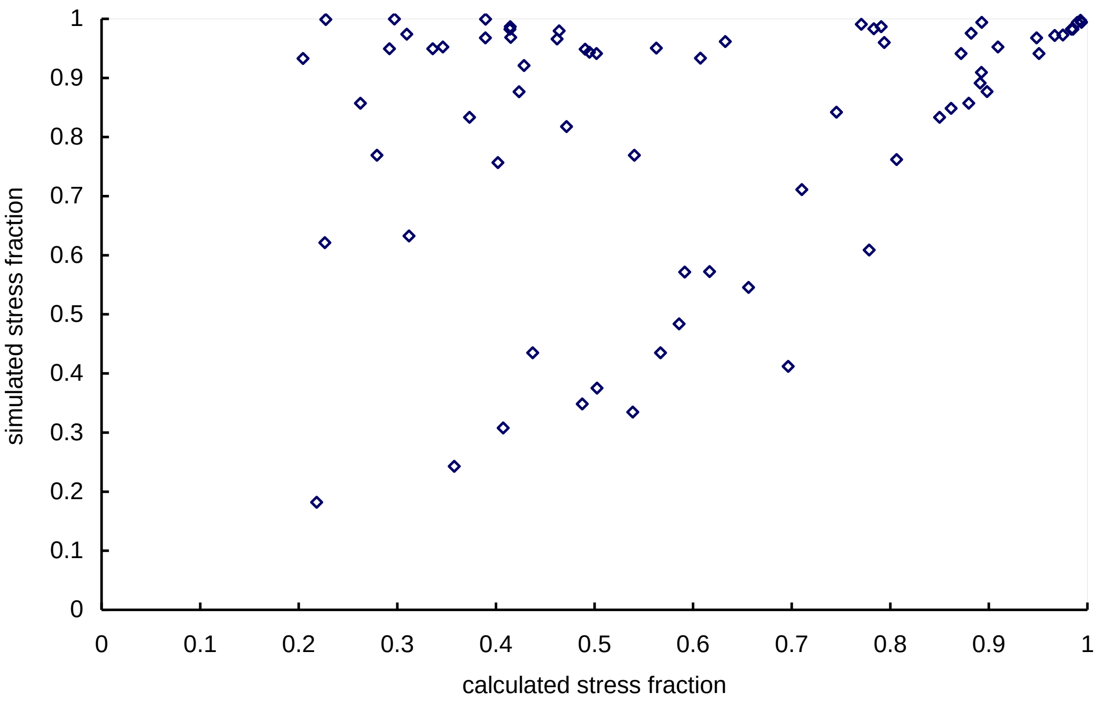
This expression neatly simplifies to:

$$h = h_\mathrm{c} + \frac{r}{3} \qquad \text{Equation 6.14}$$

The concept of an "unsupported" tank has little meaning in the context of a domed bottom tank as such a tank must be supported across its entire lower surface to avoid substantial point loads and is therefore unlikely to be built above-ground. In this case the stress fraction was redefined from "the ratio of supported to unsupported maximum stress" to "the ratio of maximum stress developed with soil support to that developed with a very soft support of 10 000 N/m^3". 10 000 N/m^3 was chosen as it allowed stable FE analysis and was 2 orders of magnitude below the smallest MOSR used in the "supported" simulations.

The analysis shows a considerable scatter with higher simulated stress fractions than those predicted by Equation 6.8 and little discernable pattern save for an apparent lower limit on the ratio of simulated/calculated peak stresses.

Figure 6.4: Simulated and calculated stress fractions for dome bottomed tanks



There are two potential reasons for this:

- The domed bottom also gives rise to pressure-based forces that are not strictly horizontal but have vertical components
- The soil support results in a shear component on the walls of the tank that in some cases can be the most significant stress

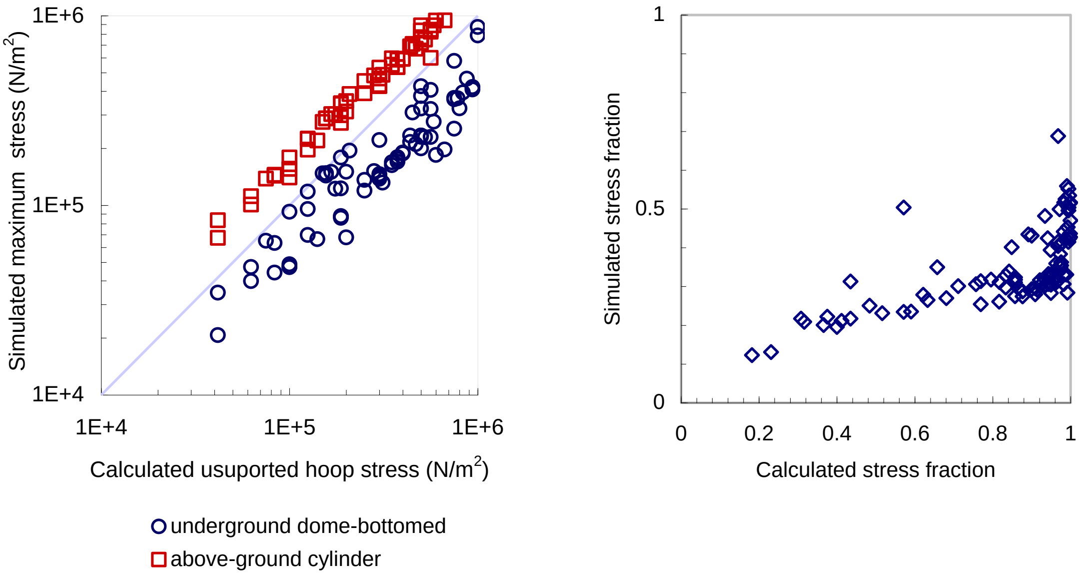
These reasons make it unlikely that a simple correction to hoop theory will be possible.

As the comparison between an "unsupported" and "supported" dome-bottomed tank has little meaning, it is more useful to compare the dome-bottomed tank with an above-ground cylindrical design of the same volume and aspect ratio. Figure 6.5 shows comparisons between the maximum stress developed in an above-ground cylindrical tank and in an underground dome-bottomed tank. The reduction in maximum stress is significant with half the scenarios simulated exhibiting stress reductions greater than 68%, and 90% of the scenarios showing stress reductions of more than half. These reductions are, in fact primarily due to a lack of stress concentrations in the tank rather than soil support. These benefits are actually much larger than the benefits of soil support and show that *shape optimisation* is the real benefit of building tanks underground rather than stress reduction through soil support alone. The actual stress reduction depends on tank geometry and soil support and is a very weak function of the stress fraction calculated using Equation 6.8. Further investigation may reveal an analytical solution, however the confounding factors mentioned above make this a challenge.

Figure 6.5: Comparisons between unsupported cylindrical tanks and underground dome-bottomed tanks

a. Simulated maximum 1st principle stress in unsupported cylindrical tank and underground dome-bottomed tank compared with calculated unsupported hoop stress

b. Fraction of simulated maximum principle stress of an above-ground cylindrical tank developed in an equivalent underground dome-bottomed tank compared with calculated hoop stress fraction



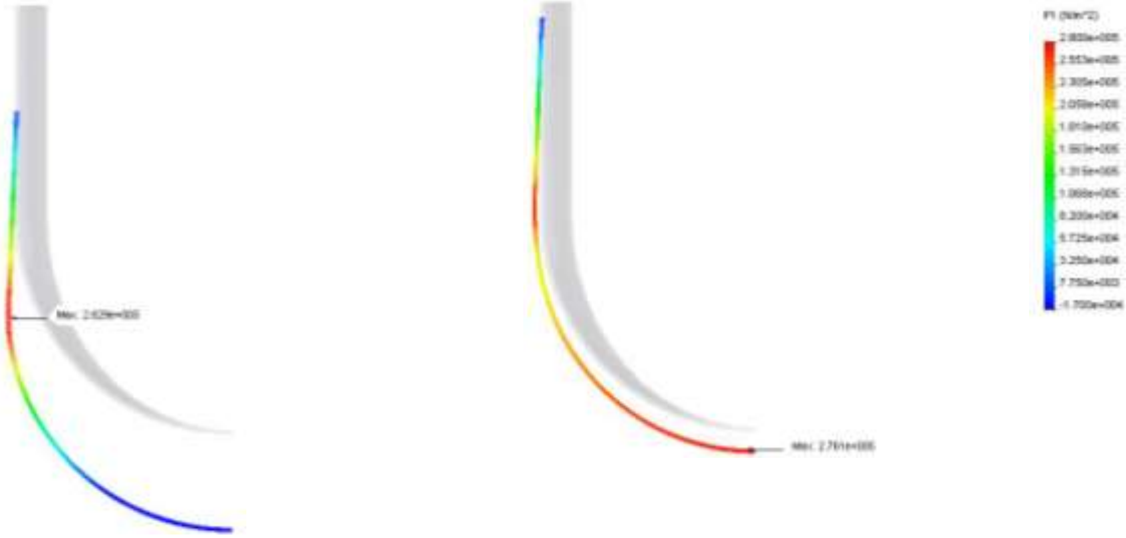
6.4. WEIGHT EFFECTS

It is often assumed that the most important stresses in a water tank will arise from water *pressure* (e.g. [Watt, 1993; Manning, 1967; Ghali, 2000]), however when soil support is considered there are a number of stresses resulting from the soils support of the *weight* of the water. Further analysis of the dome-bottomed tank shows that stresses on the outside of the walls in an underground tank can have the additional component of shear with the soil and a resultant tension within the tank wall, akin to the skin resistance load transfer found in pile foundations [Bowles, 1996]. In piles, the skin resistance forms a part of the load transfer to the soil and is highest at the top of the pile and reduces with depth until the load is transferred or the end of the pile is reached – with the balance being transferred by the reaction at the end of the pile. Long piles have a transfer that reduces in a roughly parabolic shape but shallow piles are very close to linear in their load transfer. Underground tanks represent a *very* shallow pile so the linear transfer that is modelled by an elastic foundation is likely to be a good approximation, however using the same soil modulus for forces normal to the surface and in shear may be incorrect as it assumes perfect transfer of load in shear which is unlikely.

Figure 6.6: 1st principle stresses developed in a dome-bottomed underground tank with soil support

a. with no shear load transfer

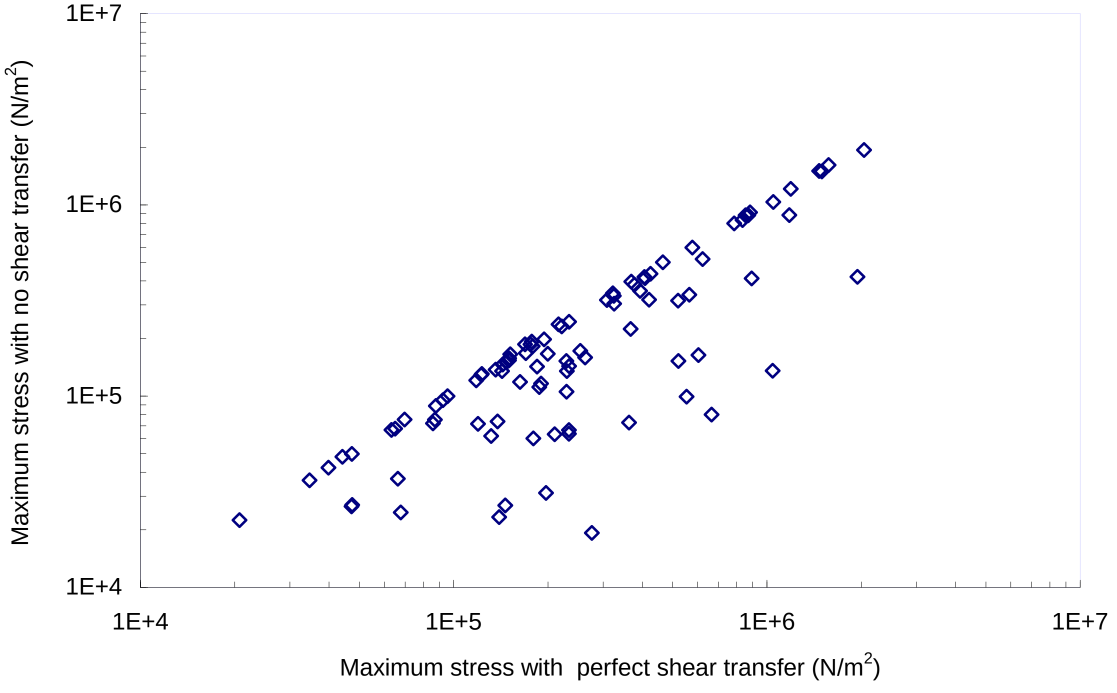
b. with perfect shear load transfer



Notes: E = 2.5E10, k= 1E8, Ø = 1.5m, depth = 1.5m; wall thickness = 15mm.
Displacement is magnified 2000 times; the greyed shape shows a segment of the tank before displacement

Figure 6.6 shows the first principle stresses on one side of a cross section of a domed-bottomed tank under a simulated water load. In Figure 6.6a, shear load transfer (i.e. load transfer tangential to the surface) is absent and the very bottom of the tank shows only compression from direct transfer of the load to the soil. Tensile stresses are developed further up the dome where hoop stress becomes dominant and the stress increases with diameter and resultant hoop stress until the maximum stress is reached at the join between the antidome and the vertical part of the tank. The maximum bending moment is also developed near to this location. Figure 6.6b shows the same tank with perfect shear transfer. In this case, the weight of the water is transferred to the soil along the total wall length and the weight is transferred from the bottom of the tank by putting the walls into tension verically. The weight transfer causes the maximum tensile stress to appear at the very bottom of the antidome while the pressure-based stress near the dome-cylinder join is also present and is little changed from the no-shear value. Depending on the tank's geometry, its elastic modulus and the soils MOSR, the pressure-based stress can be larger or smaller than the weight-based stress at the bottom. Figure 6.7 shows that over the range of tank sizes and moduli simulated there is considerable variation in maximum stress depending on whether shear transfer is dominant or absent. Of course, a real soil will give results between these two extreme cases. The figure also shows that perfect shear transfer always yields an equal or higher maximum stress than simulations without shear load transfer, so results from simulations with perfect shear transfer are likely to be conservative.

Figure 6.7: Comparison of maximum 1st principle stress of simulated dome-bottomed tanks with and without shear load transfer to the soil

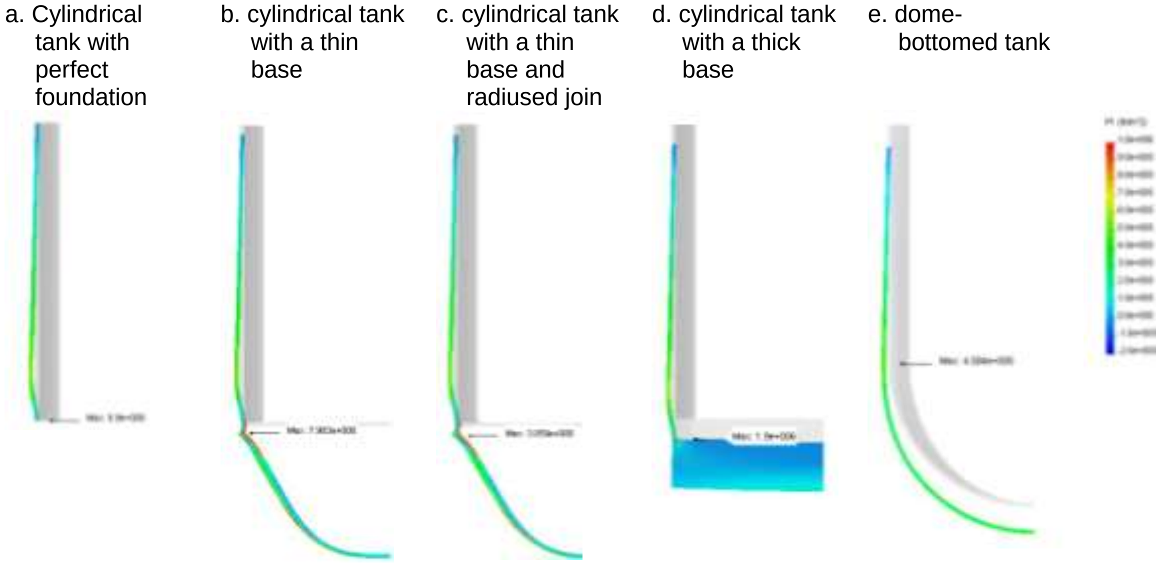


Shear load transfer is evident in several failures involving horizontal cracking in a few experimental tanks in Sri Lanka. The cracks are shown in Figure 6.8 and are further discussed in Section 8.3.1.

Figure 6.8: Horizontal cracks developed in an underground tank in Sri Lanka

The weight based stress found with the dome-bottomed tank will also be found in a cylindrical tank with an imperfect foundation. A number of scenarios are illustrated in Figure 6.9 which shows the effects of considering weight-based stress on the walls and floor of a cylindrical tank.

Figure 6.9: Comparison of displacement and first principle stress developed by different tank geometries with perfect shear transfer to soil under water load

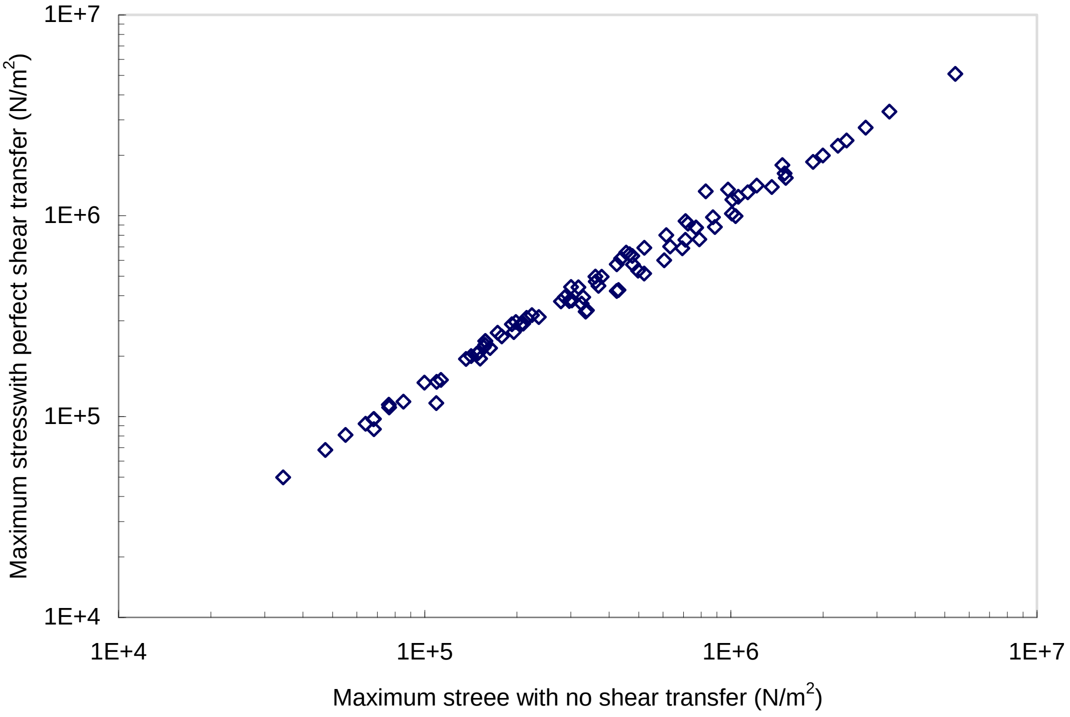


Notes: E = 2.5E10, k= 5E7, Ø = 1.8m, depth = 1.8m; wall thickness = 25mm.
Displacement is magnified 2000 times; the greyed shape shows a segment of the tank before displacement

Figure 6.9a shows a cylindrical underground tank with a perfect foundation and no vertical movement. Figure 6.9b shows a tank with the base the same as the wall thickness as would be the case with a dome-bottomed tank. Such a tank shows a significant stress resulting from the bending moment where the wall meets the base caused by the base bending downwards under the weight of the water – an effect usually neglected in analysis of cylindrical tanks. A radiused join (Figure 6.9c) reduces this stress however it is still very high. A good foundation (Figure 6.9d) reduces the bending, however the stress at the base remains much higher than shear weight unloading to soil. For comparison, a dome-bottomed tank of the same volume and diameter is shown in Figure 6.9e with all stresses lower than those for the cylindrical tank. It is also worth noting that in this case, the pressure-based stress at the dome-cylinder interface is higher than the weight-based stress at the bottom of the dome.

When the 100 scenarios used in the previous analysis are simulated with and without shear load transfer but with a thick foundation, the results (shown in Figure 6.10) show that unlike the case of the dome-bottomed tank, the stress at the base of the tank wall is so large that it dwarfs the weight-based tensile stress resulting in near identical maximum stresses in all scenarios simulated.

Figure 6.10: Comparison of maximum 1st principle stress of simulated cylindrical tanks with and without shear load transfer to the soil



6.5. CONCLUSIONS

Underground tanks do provide a significant material economy over similar capacity and diameter above-ground designs but this is not as was previously surmised due mainly to the soil support which in an average soil can only provide about 10% of the necessary reaction. The main material-saving advantages to a soil supported tank are:

- The ability to use more material efficient shapes
- The ability to use shapes which reduce bending moments
- The removal of the need for an expensive foundation

An analytical solution can be used to determine the material saving of a cylindrical tank with a perfect foundation and good agreement has been found between this and finite element analysis, however it is likely that in the field vertical tensile forces will be present in any underground tank due to skin resistance between the tank wall and the surrounding soil making the analysis more complex.

Cylindrical underground tanks with poor foundations and thin bases will experience significant bending moments at the joint between the base and the wall caused in part by deformation of the base which can be partially mitigated by radiusing or greatly thickening the base, however the stresses developed still far exceed those found in a dome-bottomed tank.

Further material may be saved through a reduction in necessary safety factors and by removing some material types entirely (such as steel mesh in a mortar tank). The effectiveness of the soil as a formwork is qualitatively investigated in Chapter 7.

7. TANK DESIGNS

Under the DFID project several reduced-quality, low-cost tanks were designed and field tested. The experience with these designs can be used to test the cost reduction strategies developed in Chapter 4. The next two chapters describe this process.

This chapter presents a number of low-cost designs created for the DFID project. The processes leading up to the designs are discussed along with which strategies were used. Successful designs were tested against both the “minimum” and “desirable” criteria described in Section 3.1 and also initial cost. Some less successful designs are also described and lessons drawn.

Chapter 8 considers the outcomes of field testing the designs. actual built cost are presented as well as problems that came to light and changes made to the designs to correct or alleviate those problems.

7.1. NOTES ON MEASUREMENT TECHNIQUES

7.1.1 LEAKAGE

The first of the minimum criteria for any tank is that it does not loose water at too high a rate. To test tanks for this, an apparatus was made to test leakage by measuring the water level in the tank. The apparatus is designed to be able to detect a leakage of <1 mm/day (corresponding to a leakage of between 1 and 3 l/day) over a time-frame of 6 hours. Thus a resolution of 0.25mm was required. The device is a refinement of one designed by Colin Oram for use in Uganda.

DESCRIPTION

The apparatus is pictured in Figure 7.1 and was based on a buoyant weight which would follow the water level as it fell through leakage. The weight itself was a bucket of water so that any lowering of the water-level in the tank by evaporation would be compensated for by similar evaporation in the bucket. The water level in the bucket could also be changed to raise and lower the initial "zero" height compensating for slight variations in initial height of the tank water. The bucket was suspended by a string from a lever arm which multiplied the movement by 20 times so a .25mm reading would correspond to 5mm of movement at the arm. The lever arm was suspended by two strings from an adjustable fame allowing free rocking movement with no static friction. The frame itself was seated on three adjustable screws so it could be made level on most surfaces.

Figure 7.1: Leak testing apparatus

METHOD OF USE

Setting up

- The apparatus was set up on top of an opening in the tank with the three feet resting on firm foundations. The device was levelled using the screws.
- An initial estimate of the distance of the water from the top of the tank was taken and string cut to an appropriate length so that the bucket would float with roughly half of its volume immersed.

- The string was tied to a hole on the short end of the lever arm
- The other end of the string was tied to the bucket handle
- The bucket was quarter filled with water.
- The bucket was lowered into the tank water.
- The water level in the bucket was adjusted until the end of the long end of the lever arm was opposite a zero mark.

Readings

- The vertical distance of the long end of the lever arm from the zero point was periodically measured with a ruler
- The testing was ended after 6 hours or when the marker had moved 150mm whichever came first
- The readings were divided by 20 to obtain the actual height reduction and then multiplied by the tank's cross sectional area for the volumetric reduction over the period of the test. This was then scaled to give a leakage in litres/day.

7.1.2 FINANCIAL MEASURES

All tanks were costed as they were being built, in terms of materials used and labour. Labour was also divided into skilled-specialist labour and labour that could be provided by a household. As the tanks were prototypes some over-estimations were bound to occur so the actual costs were refined by discussion with a contractor on the basis of "cost to build five tanks of the type prototyped". For sizes of tank not built in the prototyping stage, these costs were then scaled assuming the following:

- Wall thickness, surface area and materials for each tank were based on stress calculations attaining a specified design stress.
- Hole-digging labour was considered directly proportional to hole volume i.e. no penalty for digging particularly deep holes.
- Wall building/plastering time was considered proportional to surface area

- Labour was rounded up to the nearest full day.

All costs given in the chapter are quoted with and without using household labour to reflect the range of opportunity costs of such labour.

7.2. BELOW-GROUND DESIGNS

Below ground designs are the mainstay of low cost tank design. The implication is that if built in stable soil, the ground itself can be made to take 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. This is true of materials with a low Young's modulus such as plastics but *not* true for tanks lined with cementitious materials. When underground tanks were analysed theoretically in Chapter 6, it was found that, in the case of a mortar tank, the soil supplied rather little support, however significant advantage *is* derived from the use of advantageous shapes.

7.3. TUBE TANK

7.3.1 DESCRIPTION

Low-density polyethylene (LDPE) is a low-cost waterproof material that is widely used throughout the world. It is available in many low-income countries as a rolled tube in a range of thicknesses with a flattened width of approximately 900mm producing an opened-out diameter of 573mm. The cost of this tube (in 2001) was £0.38/m in Sri Lanka (SL), £0.54/m in Ethiopia (Et) and £0.40/m in Uganda (Ug). The open diameter results in a volume of just over 0.25m^3 per metre length. The cost is therefore only £1.66, £2.33 and £1.75 (in SL, Et & Ug respectively) per m^3 of storage for the tube itself.

LDPE has a Young's modulus of between 1E7 N/m^2 and 2E7 N/m^2. Using the higher of these values and the open tube diameter in Equation 6.10 reveals a potential soil contribution of 80mm even in loose soil. If this is used in Equations 3.2-3.4, it is found that the soil contribution alone can sustain a LDPE tank to a depth of over 100m before the 7E6 N/m^2 yield stress of LDPE is reached. This means that any practical depth of tank can be made with the polyethylene forming a waterproof barrier only.

Initial designs for a tube tank were made 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 original design (see Figure 7.2) 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 7.2: Original tube tank design

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 as it has a very small diameter
- 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

A redesign of the tube tank was carried out under the DFID project intended to address these shortcomings. The new design (see Figure 7.3) includes the following changes:

- The tube has been folded in two, eliminating the tied joint, making the hole 40% larger and consequently, easier to dig. One end of the tube is at the inlet, the other is at the outlet

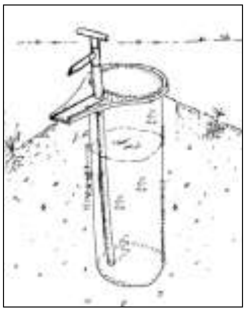
- The design was moved underground to remove the cost of the parapet wall. Instead a precast concrete cover is used reducing the cost and allowing centralised manufacture. The cover itself is similar to the arrangements found on some handpumps and so should be familiar to use. It is made using similar casting techniques to pit-latrine covers (sanplats).

- The tube itself is easily replaceable by using a retaining ring and binding wire to hold the tube in place.

- The seal between the filter basin and the wall was improved by introducing a partly inflated bicycle inner tube

- A variant folds the tank into a "L" shape reducing the depth of the excavation and replacing the vertical hole with a trench this allows simpler digging and higher capacities but it does add the problem of covering the excavation.

Figure 7.3: Modified tube tank design.



General arrangement of tube tank Mk 2

Retaining ring

Overflow

Horizontal folded tube tank being filled

With these modifications, cost reduces to £17 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 about 15% can be made by reducing skilled 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 volume is determined by hole depth so the deeper a household digs, the larger the store. Any 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.

7.3.2 BREAKDOWNS

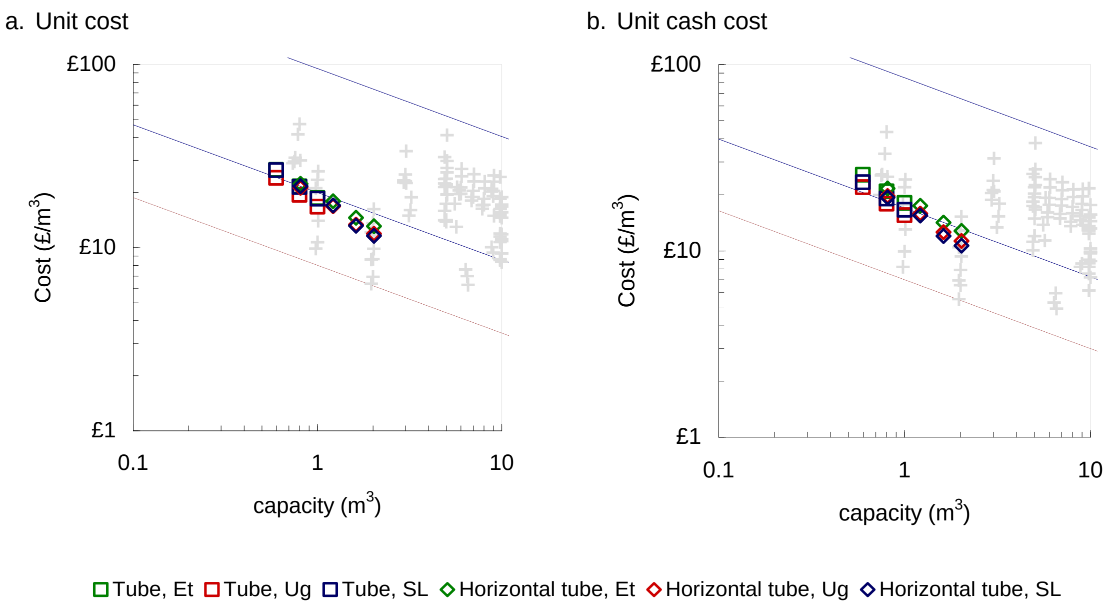
Table 7.1 Strategies used – tube tank

Strategy	How it is used
Functional separation	Soil fo structure, LDPE tube for waterproofing
Workshop manufacture	cast concrete platters, standardised handpump
Labour division	Largest part of the labour component - the hole - can be dug by householder prior to tank delivery
Design for reduced maintenance cost	tube can be easily replaced
Design to enhance rapid implementation	pre-manufactured parts can be quickly assembled on a prepared site

Table 7.2: Criteria met – tube tank

Criteria	How it is met
No excessive water loss	No water loss detected – polythene proved waterproof
Sufficient structural strength	Soil provides all strength needed
No danger to users	Vertical tank is covered by concrete slab
No entry to light/vermin	Entry sealed by filter / filter sealed by bicycle inner tube
Ventilated	Vented through filter cover
Won't taint water	Polyethylene does not flavour water
Low cost	In the low-normal or low-cost range
Durable or easy to maintain	Tube is easy to remove to service, clean or replace
Ease of entry/exit of water	Large (40cm) filter opening for entry; pump for exit
Overflow arrangement	Tank overflows through a gap in the entry to a spillway drawing water away from the hole
Easy to clean	Must be cleaned with the tube out
Resistant to large forces	??
Look attractive	??
Other	Well suited to workshop manufacture and rapid installation

Figure 7.4: Costs – tube tank



7.4. THIN LINING OF CEMENT PASTE

Neat cement is about 40% stronger and has only half the stiffness of as 1:3 mortar. It is also almost completely impermeable, making the material ideal for the construction of very thin shells to line pits. Initial experiments in Sri Lanka of lining a 1m depth pit of Ø1.5m with as thin a layer of cement paste as could be reliably applied (approx 3mm), revealed leakage of less than 10 litres/day and the surface showed no cracks. A larger pit of 3m depth was then tried, however the cement paste was applied without due care and cracks developed upon curing. The leakage from this pit was measured as 80 litres/day, however no additional cracking was observed as a result of filling with water. Revisiting the tank 12 months after the initial build found it full of water after recent rains, however it was not possible to measure the leakage at that time.

While the thin shell cement lining technique has the potential to result in material economies of up to 50% when compared to conventional mortaring, it requires great care in application. This may make it unsuitable to unsupervised (free market) construction at a local level. The technique was therefore abandoned in favour of direct application of lean mortars.

7.5. DIRECT APPLICATION OF MORTAR

The relative benefits of small mesh volumes to above-ground "ferrocement" structures were discussed in Section 3.2.4. It is also common practice to put mesh in underground tanks when lining them with mortar. Typically this takes the form of a layer or two of chicken wire. As discussed, the strength added to mortar by small volume fractions of mesh is very small.

Moreover, There are a number of problems inherent in using mesh on a concave surface such as the inside of a hole.

Although the soil provides little support against hoop stress (though more than a couple of layers of mesh), intimate contact with the soil is extremely important to avoid point loads on the tank. Mesh introduces an over-constraint to the geometry. 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 sheet forms bubbles, losing contact with the wall, if it is too tight it will stretch tight across the wall rather than following the curve, resulting in voids.

During the product development phase of the DFID project, it was found that directly applying mortar to the walls of an excavation was a simple technique to apply in the field. A layer of 1cm could be applied easily and with reasonable quality control. As with many other cementitious tanks, waterproofing was provided by a thin layer of cement slurry applied while the mortar is green. Several tanks were built using this method to depths of up to 2.5 meters with no visible cracking and no discernable leakage.

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.

7.5.1 CALCULATING THE MORTAR THICKNESS

A key part of the design of the underground mortar tanks was material reduction by the use of cement by reducing section thickness and concentration of cement in the mortar to an optimised minimum. The tanks incorporated a domed bottom, so an analytical solution was not possible, however finite element analysis can be used to calculate design curves for the tanks.

Still [2006] has derived a relation for mortar strength and sand:cement ratio (assuming optimal water content) based on data from Than [1991]. A brief summary of results are shown below in Table 7.3.

Table 7.3: Properties of mortar

Mortar mix (Sand:Cement)	Tensile Strength (Mpa)	Compressive Strength (MPa)	Elastic Modulus (GPa)
3:1	3.0	40	30
5:1	2.1	22	22
8:1	0.7	10	10

Notes: Strength Data derived from Than [Thanh, 1991]
Modulus calculated from the relation $E_c = 4.73\ f_c'^{\,0.5}$ [Neville, 1995]

Using FE analysis and elastic foundation soil simulation as described in Chapter 6, four sizes of tank were modelled with a range of wall thicknesses and the elastic moduli appropriate to each mortar mix. The maximum stresses from the simulations are charted as the solid lines in Figure 7.5. The dotted lines correspond to half the tensile strengths from Table 7.3 representing a safety factor of 2. From these graphs, the thickness of each mix of mortar required for each tank type and each soil type can be determined by the intersection of solid and dotted lines.

For an optimum cost of tank it is necessary to examine the relative cost of different mortars which depend on the cement:sand ratio and the ratio of cement to sand cost. The relative costs of the mortar mixes considered are shown in Table 7.4 and are relatively insensitive to location, due mainly to the extreme difference in cost between cement and sand.

Table 7.4: Cement:sand ratios and relative costs of mortars in study countries based on 2001 prices.

Country	Cement cost (£/kg)	Sand cost (£/kg)	Cement:sand cost ratio	Relative cost of mixes (1:3 = 1)		
				1:3	1:5	1:8
Uganda	0.050	0.0018	27.36	1.00	0.71	0.52
Ethiopia	0.057	0.0020	27.63	1.00	0.71	0.52
Sri Lanka	0.110	0.0016	70.40	1.00	0.68	0.47
Mean				1.00	0.70	0.49

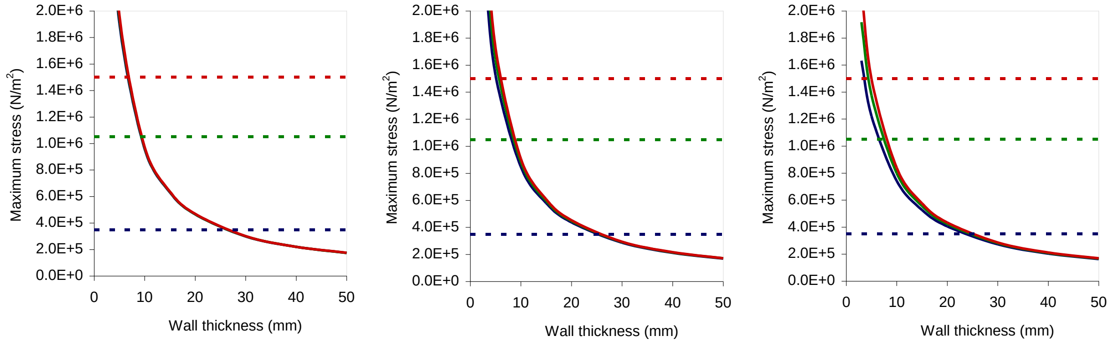
Using the relative costs of mortar and the thicknesses determined with Figure 7.5, it is possible to establish the thickness and mixture that result in the least cost. These are detailed in Table 7.5. and summarised in Table 7.6.

In field testing, it was found that the minimum thickness that could be reliably applied was 10mm. Moreover In the interests of clarity of communicating to tradespeople and quality control, thickness has been rounded up to the nearest 5mm. The 1:5 mix tends to be the cheapest for all tank sizes – or very close to the cheapest so for simplicity, this mix has been selected for all designs.

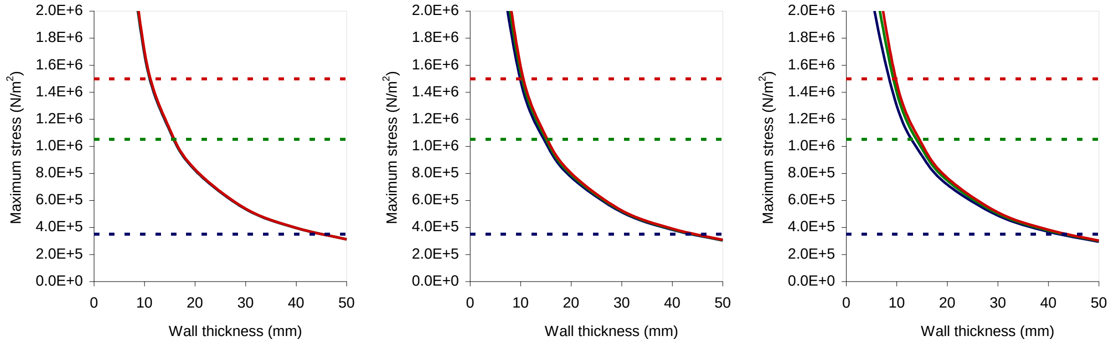
Figure 7.5: Stresses in underground mortar tanks for 3 mortar mixes

Loose soil (*k*=1E7) Medium soil (*k*=5E7) Firm soil (*k*=E8)

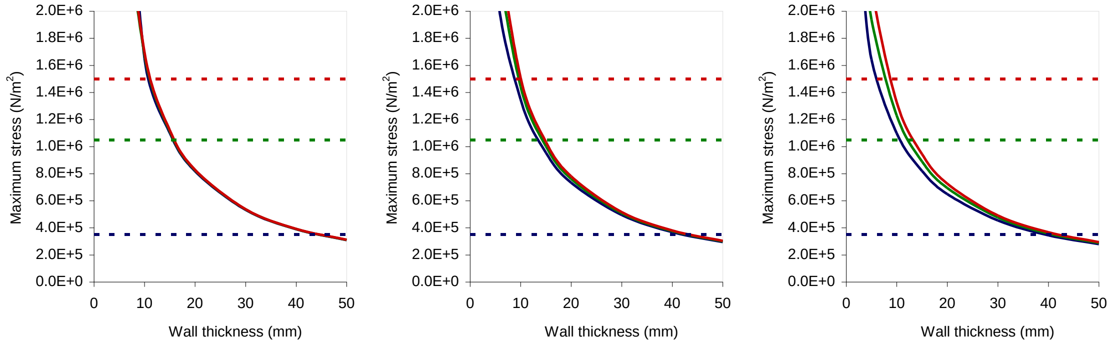
Ø = 1.5m; depth = 1.5m



Ø = 1.5m; depth = 2.5m



Ø = 2.0m; depth = 1.9m



Ø = 2.0m; depth = 2.9m

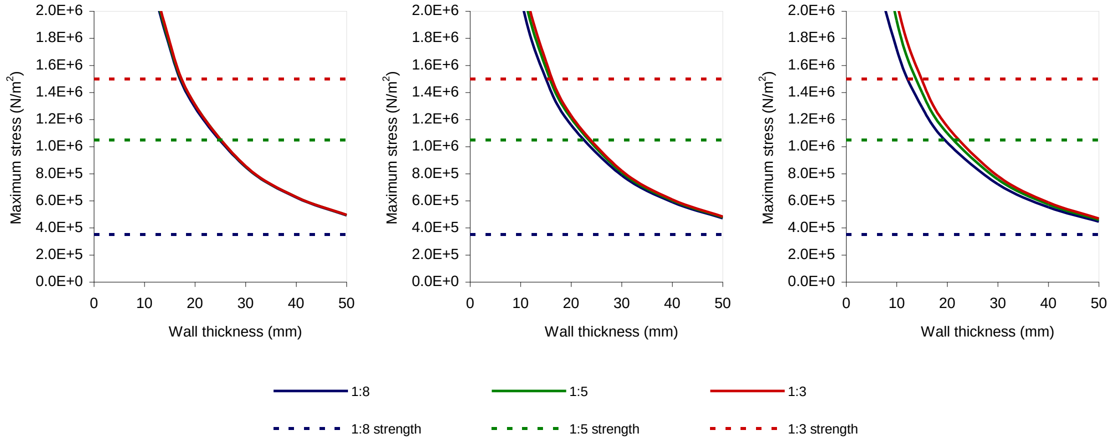


Table 7.5: relative costs of using different mortar mixes for underground tanks

a. Ø = 1.5m; depth = 1.5m

Soil type	Cement:sand ratio	Required thickness (rounded up to next 5mm)	Relative cost
Loose	1:3	10	1.00
	1:5	10	0.70
	1:8	25	1.23
Medium	1:3	10	1.00
	1:5	10	0.70
	1:8	25	1.23
Firm	1:3	10	1.00
	1:5	10	0.70
	1:8	25	1.23

b. Ø = 1.5m; depth = 2.5m

Soil type	Cement:sand ratio	Required thickness (rounded up to next 5mm)	Relative cost
Loose	1:3	15	1.00
	1:5	15	0.70
	1:8	45	1.47
Medium	1:3	10	1.00
	1:5	15	1.05
	1:8	45	2.21
Firm	1:3	10	1.00
	1:5	15	1.05
	1:8	45	2.21

c. Ø = 2.0m; depth = 1.9m

Soil type	Cement:sand ratio	Required thickness (rounded up to next 5mm)	Relative cost
Loose	1:3	15	1.00
	1:5	15	0.70
	1:8	45	1.47
Medium	1:3	10	1.00
	1:5	15	1.05
	1:8	45	1.47
Firm	1:3	10	1.00
	1:5	15	1.05
	1:8	40	1.96

d. Ø: 2.0m; depth = 2.9m

Soil type	Cement:sand ratio	Required thickness (rounded up to next 5mm)	Relative cost
Loose	1:3	20	1.00
	1:5	25	0.88
	1:8	>50	>1.23
Medium	1:3	20	1.00
	1:5	25	0.88
	1:8	>50	>1.23
Firm	1:3	15	1.00
	1:5	20	0.93
	1:8	>50	>1.63

Table 7.6: Optimum thickness and mixture for underground tanks

Soil type	Loose soil		Medium soil		Firm soil	
	Mix	Thickness (mm)	Mix	Thickness (mm)	Mix	Thickness (mm)
Ø:1.5m; depth:1.5m	1:5	10	1:5	10	1:5	10
Ø:1.5m; depth:2.5m	1:5	15	1:5	15	1:5	15
Ø:2.0m; depth:1.9m	1:5	15	1:5	15	1:5	15
Ø:2.0m; depth:2.9m	1:5	25	1:5	25	1:5	20

7.6. BELOW-GROUND MORTAR TANK WITH ORGANIC COVER (“THATCH TANK”)


Ring beam with trench


Shuttering for pump and overflow mounting


cover under construction


Finished tank with cover in place

7.6.1 DESCRIPTION

As the cost of the below-ground part of a tank is reduced, the cost of the cover of the tank becomes dominant. 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 in order to comply with minimum water quality requirements.

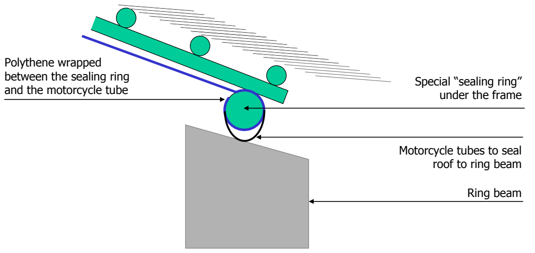
- The organic material must not fall into the tank and contaminate the water
- Runoff from the organic roof will be of low quality and so must not be allowed to enter the tank
- The roof must provide a good barrier to vermin entry, especially as some creatures can make their homes in thatch.
- The wooden supports must not be exposed to the humid atmosphere inside the tank which will make them liable to rot
- 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 a good 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. These details are illustrated in Figure 7.6a-7.6c

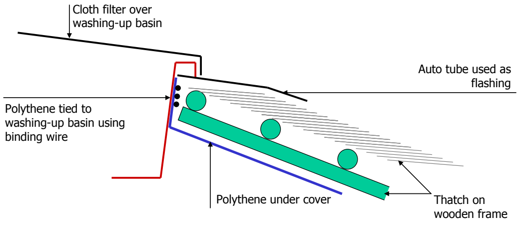
Below-ground tanks also need care with avoiding floodwater ingress and also with overflow arrangements. The design uses an overflow employing an upwardly facing elbow connected to an outflow pipe which leads either to a nearby slope or to an infiltration pit. Stormwater ingress is avoided by digging a channel around the ring beam to a width appropriate for the runoff. These details are illustrated in Figure 7.6d.

Figure 7.6: Essential features of the thatch tank

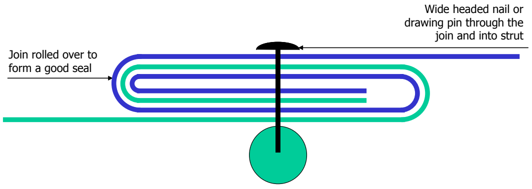
a. bottom seal



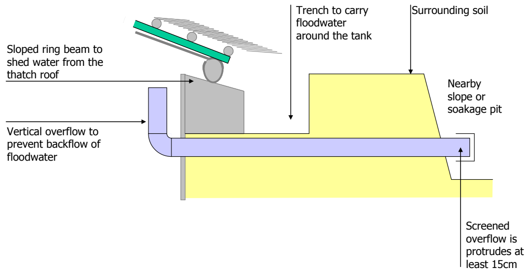
b. top seal



c. polyethylene joining



d. overflow and drainage



The overall combination of mortar-lined pit and low-cost roof yields a tank that uses very little material but is quite householder-labour intensive.

BREAKDOWNS

Table 7.7 Strategies used – thatch tank

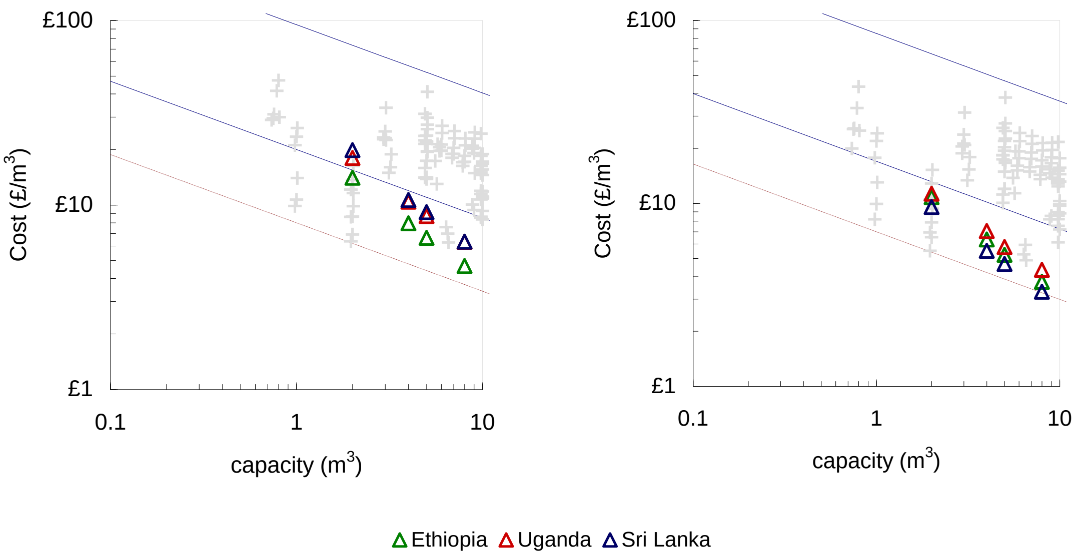
Strategy	How it is used
Functional separation	soil: supporting structure, Mortar partial structure and waterproofing LDPE: roof sealant, local round-wood and thatch roof structure and shade
Labour division	Hole digging forms a large part of the labour component Roof structure can be done with household labour (in some places) – but trained persons must perform the sealing operation
Staged construction	Can be upgraded to Dome tank

Table 7.8: Criteria met – thatch tank

Criteria	How it is met
No excessive water loss	No water loss detected
Sufficient structural strength	No observed cracks in any prototype
No danger to users	Tank is sealed by cover but strength is determined by cover build
No entry to light/vermin	Entry sealed by filter / all entrances covered, filter basin sealed with inner tube
Ventilated	Vented through filter and overflow
Won't taint water	Some flavour from hydrating cement but improves with time
Low cost	In the low-cost range
Durable or easy to maintain	Underground section durable and easy to access, cover uses local techniques so easy to maintain
Ease of entry/exit of water	Large (60cm) filter opening for entry; pump for exit
Overflow arrangement	Tank overflows through dedicated pipe
Easy to clean	Easy access with cover removed
Strength for large forces	Optimised design will likely break under excessive force – but repairable
Look attractive	Not really
Other	Staged construction – Can be upgraded to dome tank

Figure 7.7: Costs – thatch tank

a. Unit cost b. Unit cash cost



7.7. PARTIALLY BELOW-GROUND TANK WITH FERROCEMENT DOME ("DOME TANK")


Lining the pit


Fixing the frame for the dome


first layer of mortar for the dome


Finished tank

7.7.1 DESCRIPTION

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, taking advantage of the economies to be found there, but with some of the tank protruding. Partially-below-ground tanks are somewhat more expensive than purely underground tanks though, as the above-ground section can cost 3-4 time more per unit volume than the underground section. The thatch tank can be upgraded to form a 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[1]. The

[1] The dome was designed in conjunction with N.U.K. Ranatunga of the Sri Lankan Water Supply &. Drainage Board drawing on his experience with the pumpkin tank

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 more expensive than the Thatch Tank but will require less maintenance.

7.7.2 BREAKDOWNS

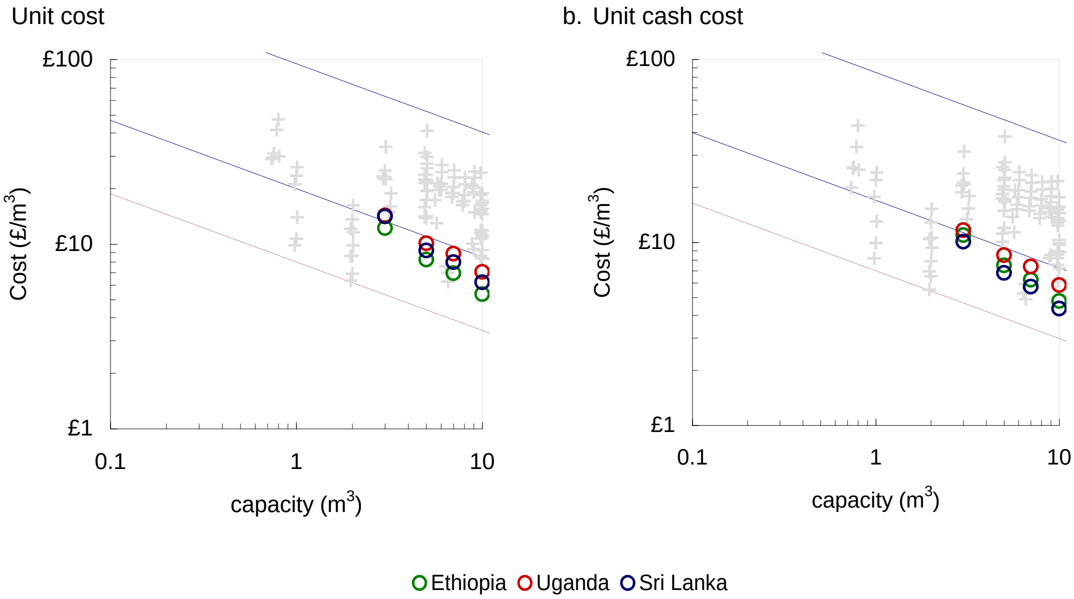
Table 7.9 Strategies used – dome tank

Strategy	How it is used
Functional separation	soil: supporting structure, Mortar partial structure and waterproofing
Labour division	Hole digging forms a reasonable part of the labour component
Staged construction	Can be upgraded from Thatch tank

Table 7.10: Criteria met – dome tank

Criteria	How it is met
No excessive water loss	No water loss detected
Sufficient structural strength	No observed cracks in any prototype
No danger to users	Tank is sealed by ferrocement dome
No entry to light/vermin	Entry sealed by filter / all entrances covered, filter basin sealed with inner tube
Ventilated	Vented through filter
Won't taint water	Some flavour from hydrating cement but improves with time
Low cost	In the low-cost range for most sizes
Durable or easy to maintain	FC and mortar construction durable and easy to access
Ease of entry/exit of water	Large (60cm) filter opening for entry pump for exit
Overflow arrangement	Tank overflows through pump
Easy to clean	Easy access through large inlet hole
Strength for large forces	Optimised design will likely break under excessive force – but repairable
Look attractive	Users say yes
Other	Staged construction – Can be upgraded from thatch tank

Figure 7.8: Costs – dome tank



7.8. 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 and there are geometric constraints of height and stability. The principle of functional separation allows some scope for cost reduction. Waterproofing can be done by either mortar or a liner.

7.9. 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 is often 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.

7.10. WORKSHOP MANUFACTURED TIMBER STRUCTURE WITH POLYETHYLENE LINER ("CRATE TANK")


lid with hole for inlet


delivering the crate to the household


Internal configuration


Finished tank in use

7.10.1 DESCRIPTION

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 quite 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 of the waterproof membrane is similar 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. Like the tube tank, the tube is slightly longer than the space in which it sits so the top is never subject to a vertical load from the weight of the water. The outlet and overflow can be on any side 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.

7.10.2 BREAKDOWNS

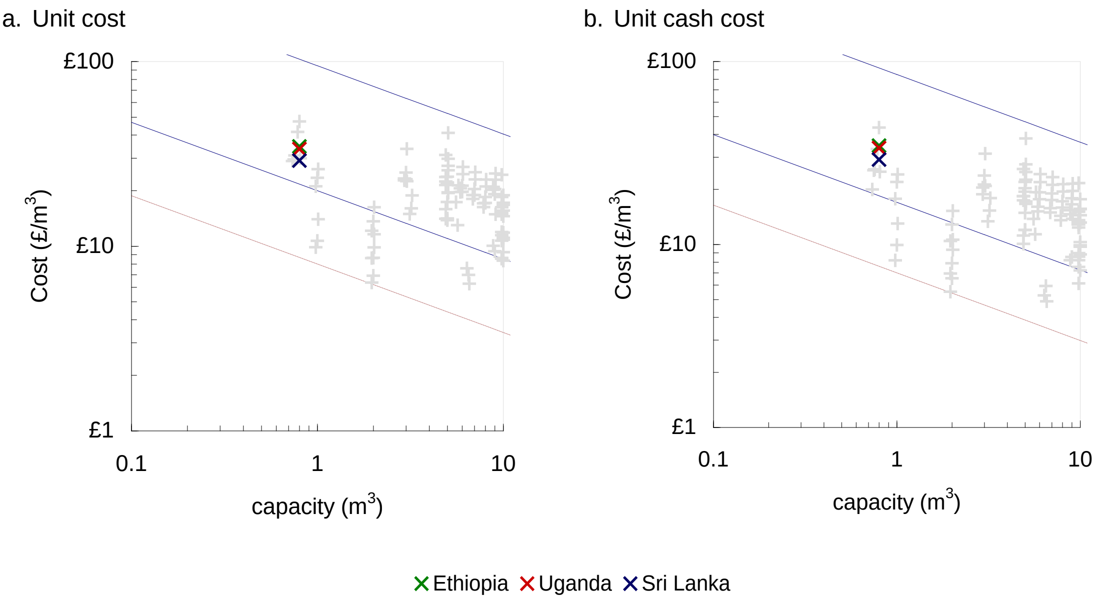
Table 7.11 Strategies used – crate tank

Strategy	How it is used
Functional separation	Wood crate: supporting structure, LDPE waterproofing
Workshop manufacture	Can be made centrally and delivered to site
Special circumstances	Small footprint to fit into urban spaces

Table 7.12: Criteria met – crate tank

Criteria	How it is met
No excessive water loss	No water loss detected
Sufficient structural strength	No excessive deformation observed when filled with water
No danger to users	Tank is stable when full and empty
No entry to light/vermin	Entry sealed by filter / all entrances covered, filter sealed with inner tube
Ventilated	Vented through filter and overflow
Won't taint water	Polyethylene does not flavour water
Low cost	In the low end of the normal cost range
Durable or easy to maintain	Tube can be replaced (though not easily)
Ease of entry/exit of water	Large (40cm) filter opening for entry tap for exit
Overflow arrangement	Tank overflows through dedicated pipe
Easy to clean	Not especially
Strength for large forces	Unknown – though wood structures usually fare well in quakes
Look attractive	Not really
Other	Small footprint for urban areas; easily transported

Figure 7.9: Costs – crate tank



7.11. WATTLE AND DAUB STRUCTURE WITH PLASTIC LINER ("MUD TANK")


Bamboo frame


Detail of access hole


Finished $5m^3$ tank


Mud blocks under test

7.11.1 DESCRIPTION

A simple way of producing an above-ground tank with some of the economy of a below-ground tank is to use earth construction. Earth technologies have been used in building for millennia and such techniques are often the mainstay of housing for the poor [Houben & Guillaud, 1989; Norton, 1997]. Wattle and daub is a widespread practice for building from earth, particularly when householders build their own homes. The technique uses unmodified mud to fill a frame structure made from roundwood or bamboo. 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 5 mud blocks from different sources were tested for tensile strength using a flexing beam test based on BS 1881: Part 118 [British Standards Institution, 1983] and it was found that the maximum tensile stress (at the bottom edge of the beam) lay in the range of 729 kN/m^2 to 894 kN/m^2 a with a standard deviation of 62 kN/m^2 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. Assuming a safety factor of 2 and so using a maximum tensile stress of 365 kPa in Equations Equations 3.2-3.4– shows that the required wall thickness needs to be 85mm for a 2m high, 2m diameter tank like the $5m^3$ prototype. Wattle and daub construction tends to create thicker walls than this though and the material cost is purely in household labour, so the walls were specified in the range of 150mm to 200mm which also gave a more realistic safety factor of 4-5.

Initial tests used cement paste as a liner, however, as discussed in Chapter 6, mud has a Young's modulus several orders of magnitude lower than cement, so the structure expands slightly under load causing the cement lining to crack resulting in leakage and damage to the mud walls. A plastic sheet liner proved much more satisfactory but large sizes are difficult to produce. For the testing some specialised large sheeting was obtained, however this was expensive and rare so not a good option for supplying the rural poor. It did prove the structure was able to take the pressure of water however and if the sealing problem can be reliably solved, wattle and daub water-retaining structures should be viable.

7.11.2 BREAKDOWNS

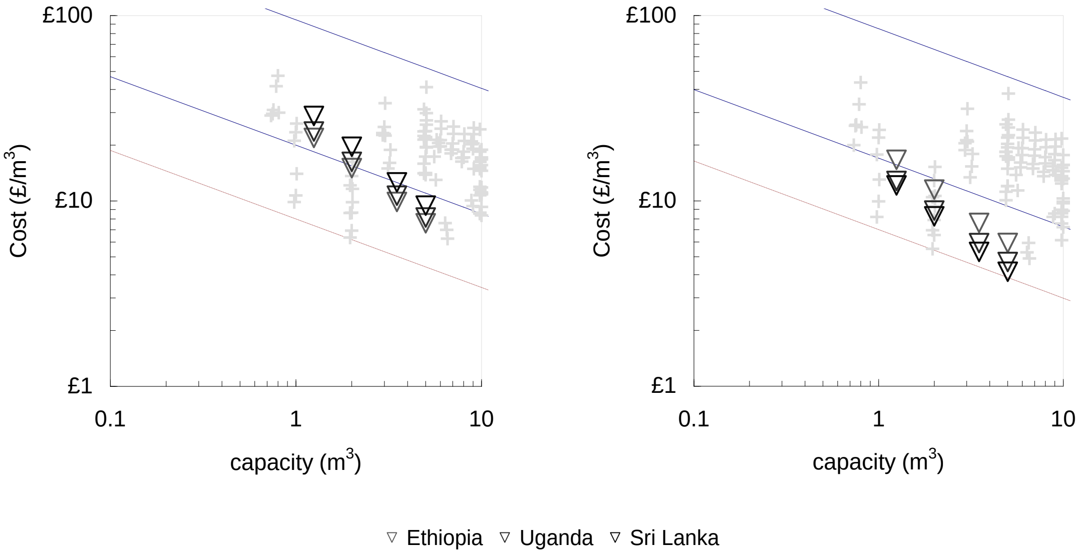
Table 7.13 Strategies used

Strategy	How it is used
Functional separation	soil: supporting structure, LDPE waterproofing
Labour division	Wattle and daub structure can be built by household

Table 7.14: Criteria met – crate tank

Criteria	How it is met
No excessive water loss	No water loss detected with plastic liner, cement liner leaks. ANY leaks damage the structure and must be avoided
Sufficient structural strength	No excessive deformation observed when filled with water. Leaking tanks damaged structure at point of leakage by washing material away, but structure held until tank was empty
No danger to users	Tank is stable when full and empty. Leak based failure does not cause explosion (so far)
No entry to light/vermin	Entry sealed by filter / all entrances covered, filter basin sealed with inner tube
Ventilated	Vented through filter and overflow
Won’t taint water	Polyethylene does not flavour water
Low cost	In the low end of the normal cost range or in low-cost range but much of the cost is household labour
Durable or easy to maintain	Tube can be replaced (though not easily), structure needs maintenance but similar in nature to adjacent dwelling and can be done at same time
Ease of entry/exit of water	Large (40cm) filter opening for entry tap for exit
Overflow arrangement	Tank overflows through dedicated pipe
Easy to clean	Not especially
Strength for large forces	Likely poor
Look attractive	Not really (for householder) Architects like it though…
Other	Large use of household labour and gatherable materials give the tank the lowest *cash* cost of all designs

Figure 7.10: Costs – mud tank



7.12. CONNECTED DRUMS[1]


Connectors in bottom barrel

Cover being refitted with a bracket


Plumbing for filter


Assembling barrels on-site

7.12.1 DESCRIPTION

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 7.15. Initial

[1] The connected drum tanks were designed in conjunction with Bisrat Woldemariam of Water Action, Ethiopia

cleaning of the drums was done by using a locally produced caustic soda, however there is still some uncertainty about its effectiveness.

Table 7.15: 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 high compared to others and thus the material needed 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.

7.12.2 BREAKDOWNS

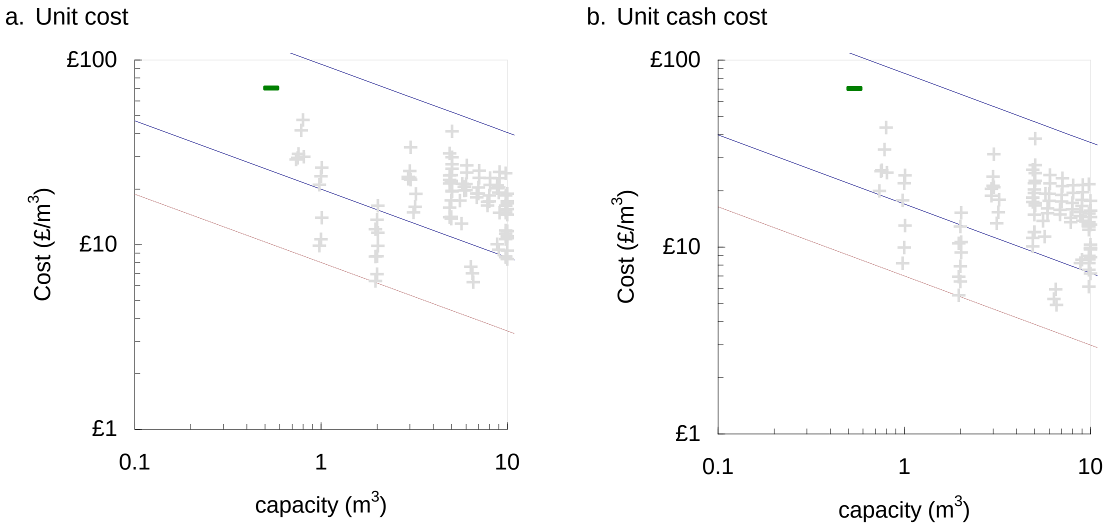
Table 7.16: Criteria met – crate tank

Criteria	How it is met
No excessive water loss	No water loss detected
Sufficient structural strength	No excessive deformation observed when filled with water.
No danger to users	Tank is stable when full and empty.
No entry to light/vermin	All entrances covered, filter basin sealed with inner tube
Ventilated	Vented through filter and overflow
Won't taint water	Slightly uncertain at prototype stage
Low cost	In the normal cost range
Durable or easy to maintain	Uncertain at this stage – though drums have been in use for several years
Ease of entry/exit of water	Large (40cm) filter opening for entry tap for exit
Overflow arrangement	Tank overflows through dedicated pipe
Easy to clean	Top and overflow drums able easy to clean – filtered drum impossible (but not necessary)
Strength for large forces	Should be good
Look attractive	OK
Other	Very small footprint and transportable; Well Suited to workshop production and rapid implementation

Table 7.17 Strategies used

Strategy	How it is used
Use of existing containers	Many households have existing drums
Workshop production	All connections suited to workshop production

Figure 7.11: Costs – drum tank



7.13. SEVERAL CONNECTED JERRY CANS

Many households already have 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 was developed to 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.

7.14. CASCADE OF WATER JARS

Many Asian households use ceramic 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.

7.15. CONCLUSIONS

Figure 7.12 shows a summary of the costs of the designs and the equivalent unit costs are shown in Table 7.18 along with the equivalent unit costs of some benchmark existing designs from Table 3.2. The new designs are mainly in the low-cost region, especially when household labour is removed from the costs. The designs presented in this chapter show that the methods discussed in Chapter 4 such as quality reduction and functional separation, are a viable method of capital cost reduction with reductions on current practice of up to 50% and up to 70% if household labour is free. The effects on longer-term cost will be discussed in Chapter 8. Minimum criteria discussed in Section 3.1 are maintained and several desirable criteria are possible – though meeting these often make the tank more expensive.

The most promising designs are:

- Thin-shell cement lining as found in the underground cement tank with an organic roof and in the partially below-ground tank with a ferrocement dome

- 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

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 cement slurry lining

Special circumstances can also be accommodated at little cost

- Tall thin structures such as the crate tank and vertically stacked drums can be built for crowded urban areas

Figure 7.12: Cost comparison of tanks developed during the prototyping phase

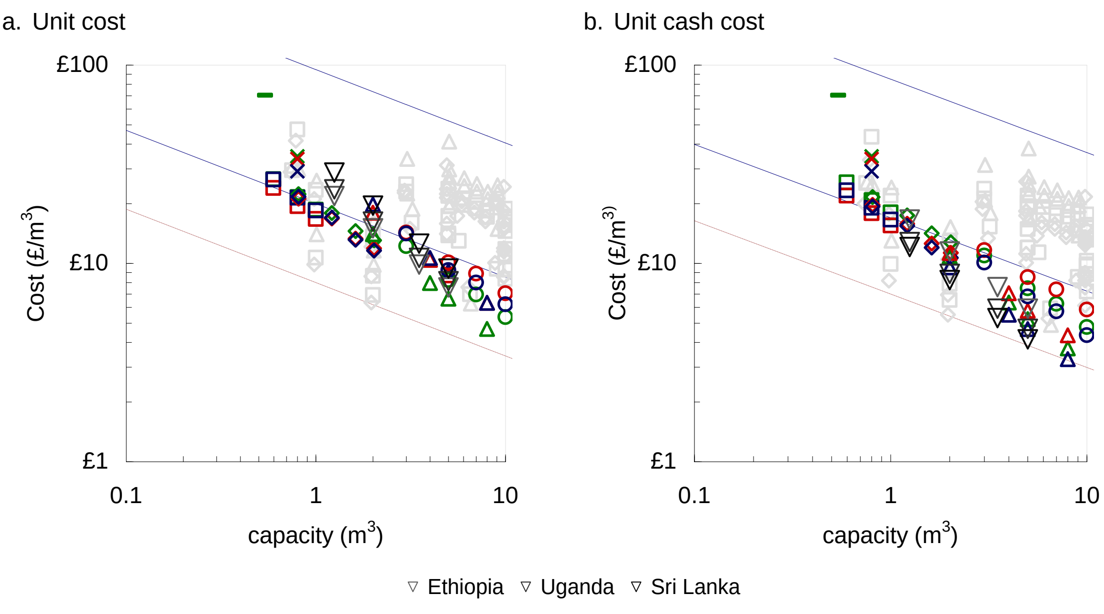


Table 7.18: Equivalent unit costs of rain-tanks (based on Ugandan material and labour costs)

Tank type	Tank cost	Tank capacity (m^3)	Simple Cost per m^3	Equivalent unit cost
Moulded plastic	£470	25	£19	£94
Drum tank	£72	0.5	£140	£88
Open frame ferrocement	£220	10	£20	£60
Pumpkin tank	£100	5	£20	£42
Plate tank	£94	10	£9	£26
Thai jar	£28	2	£14	£23
Dome tank	£50	5	£10	£21
Mud tank	£28	2	£14	£19
Tube tank	£18	1	£18	£17
Tarpaulin tank	£39	5	£8	£16
Thatch tank	£38	5	£8	£16

Generally larger designs fare better, mainly because smaller designs can be dominated by fixed costs such as pumps which cannot easily be removed without causing water quality problems. There are, however good designs available in the range of small tanks so the goal of extending the range of designs available in the low-cost range has been achieved.

Table 7.19 and Table 7.20 show the labour fractions and householder labour fractions respectively. The emphasis on minimising materials in favour of labour and particularly household labour has increased the viability of several designs, when household labour is discounted, notably the thatch and mud designs which have very high householder labour fractions. These designs only just fall into the low-cost range when labour is fully costed, but are well into the low-cost range when household labour is removed.

Table 7.19 Labour cost as a fraction of the total cost, for selected rainwater tanks

Tank type*	Ethiopia (variable material cost & low labour cost)	Uganda (high material cost & low labour cost)	Sri Lanka (low material cost & medium labour cost)
Mud tank	63%	55%	80%
Thatch tank	40%	45%	70%
Dome tank	35%	35%	60%
Tube tank	20%	30%	50%
Pumpkin tank	35%	25%	50%
Open frame ferrocement	25%	25%	35%
Drum tank	20%	25%	60%
Tarpaulin tank	20%	20%	33%
Thai jar	10%	20%	30%
Plate tank (Brazil)	15%	15%	30%
Moulded plastic	<5%	<5%	10%

Table 7.20: Household labour fractions (based on Uganda data)

Tank type	Unskilled household labour (as a fraction of total cost)	Unskilled household labour (as a fraction of total labour)
Thatch tank	20%	80%
Open frame ferrocement	14%	60%
Tube tank	9%	80%
Dome tank	9%	65%
Drum tank	9%	30%
Pumpkin tank	6%	65%
Tarpaulin tank	4%	70%
Thai jar	4%	50%
Mud tank	3%	85%
Plate tank	2%	50%
Moulded plastic	1%	50%

8. EVALUATION OF NEW DESIGNS

The more successful tanks described in Chapter 7 were field tested in Ethiopia, Uganda and Sri Lanka for a wet and a dry season. The experience gained from these field trials allowed:

- actual bills of materials and costs to be determined
- the designs to be modified were necessary based on user feedback

This and subsequent experience has also allowed an estimate of maintenance costs to be made and net present value analysis as described in Chapter 5 to be conducted. The NPV analysis allows:

- “present values” based on different rational and subjective discount rates to be determined
- the internal rate of return to be found
- the crossover discount rate against benchmark designs to be established

8.1. TANK TYPES FIELD TESTED

A number of designs were selected from those developed in the prototyping phase for wider testing in the field. Overall 4 designs were selected for extensive testing and a further 3 for more limited field-testing.

Table 8.1: Tank types installed in each location

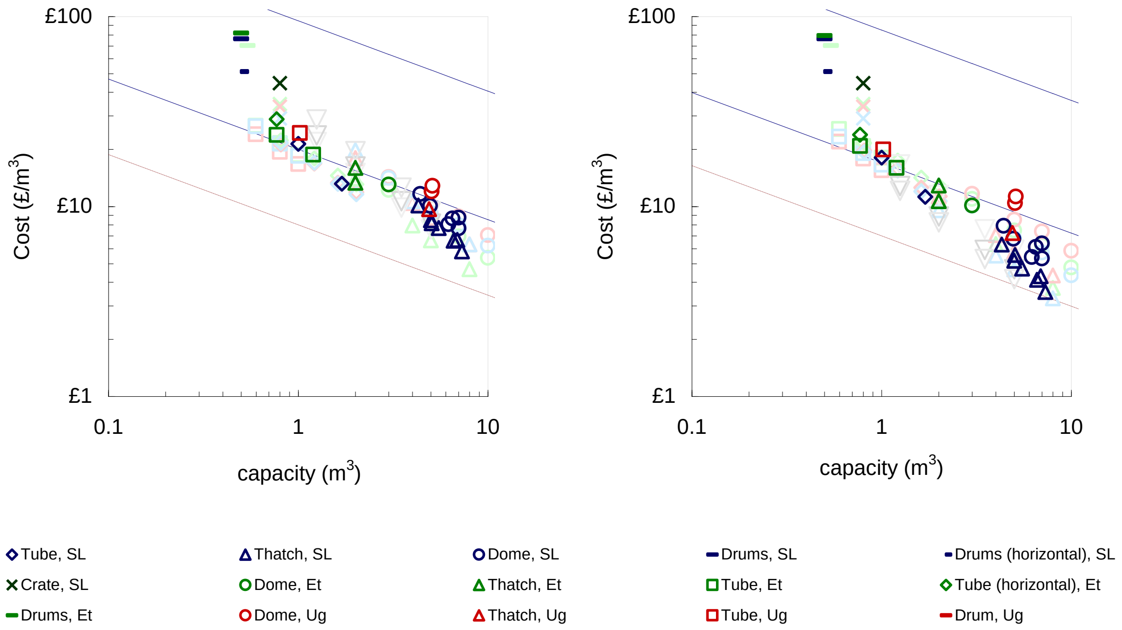
Type	Volume (m3)	Ethiopia			Uganda		Sri Lanka			Total
		Addis Ababa (urban)	Arerti (peri-urban)	Alaba (peri-urban)	Kampala (Urban)	Kibengo (Rural)	Colombo (Urban)	Aranayake (Rural)	Ambanpola (Rural)	
Dome	3			5				1		38
	5				13	6		2	7	
	6.5-7							4		
Thatch Roof Ferrocement	2		1	5						27
	5					7		4	7	
	7							3		
Drum	0.5	15		5	4	1	14			39
Tube (vertical)	0.8		4	5		4		5		19
	1.7							1		
Tube (horizontal)	0.8		3							3
Mud	0.8							1	2	3
Crate	0.8						6	1		7

8.2. FINAL CONSTRUCTION COSTS

Figure 8.1: Costs of tanks based on field construction

a. unit costs

b. cash costs



Note: faint points represent estimates from prototyping from Chapter 7, bright points represent costings from field trials.

Costs of tanks varied based not only by country but often by region of the country, particularly in the case of the drum tank which was strongly dominated by the local cost of drums. Figure 8.1 shows the varying costs of construction. Final bills of materials however did not vary greatly and were very close to the estimates in Chapter 7. Most tanks fall into the “low cost” range as compared to existing technologies with the exceptions being designs primarily to fulfil particular niches in crowded urban areas such as the crate and drum tanks, and smaller tube tanks which are dominated by fixed costs.

8.3. PROBLEMS ENCOUNTERED AND DESIGN MODIFICATIONS

8.3.1 DOME TANK

PROBLEMS

The dome tank had very few problems and was well received. Several were built in fairly loose soil in Sri Lanka and developed horizontal cracks indicating that weight-based stress was a problem. The field trials were carried out before the analysis contained in Chapter 6 was performed. A wall thickness of 10mm of 1:5 mortar was specified for the underground tanks whereas subsequent analysis showed this should have been 15mm. In addition, investigation of the bills of materials reveals it is likely that weaker 1:8 mortar was used. The finite element analysis shows that 10mm of 1:8 mortar in this situation is likely to develop a maximum stress of 1.6MPa which is more than double the tensile strength of 1:8 mortar. So the breakage, while problematic, is consistent with the FE model.

Three solutions to this problem were tried:

- Specifying a higher (1:3) cement:sand ratio for the underground section
- Adding chicken mesh to the underground section
- Raising the wall thickness from 1cm to 2cm

Unfortunately local politics meant that a permanent cure had to be found quickly so all these remedies were done at once. So far all have proved successful; however it is unclear which was most effective. According to the calculations in Section 7.2.3, it is likely that the thicker wall would have solved the problem.

In some circumstances, the water has a “shut in” taste to it as the overflow through the pump and the use of a plastic basin prevents adequate ventilation. The inlet was also cited by one or two users as a potential danger to children who could climb on the dome and fall through the inlet hole.

DESIGN CHANGES

The dome tank required very few changes. The ventilation problem was solved by abandoning the basin and inner tube seal in favour of tying the fabric over the inlet hole directly which had the side-benefit of reducing the cost. The danger to climbing children is also partially solved by the use of a tied-on filter which can also include a newly designed conical steel frame which has the dual benefit of making the inlet filter partially self-cleaning and making it impossible to fall in unless the filter is untied and removed. The filter has been described in a paper delivered to the 12th IRCSA conference in Delhi [Martinson & Thomas, 2005]

8.3.2 THATCH ROOF TANK

PROBLEMS

The thatch tank has had 4 main problems.

- The organic roof can deteriorate if it is made incorrectly or not maintained. This is particularly true in Southern Uganda and Sri Lanka where there were few thatched houses. The design was modified in these cases to use banana leaves in Uganda and Tar sheet in Sri Lanka. These modifications have thus far been successful.

- Seals were not applied correctly. There seems to have been some confusion about the functions and necessity for the seals on the cover. This has in some cases resulted in large openings into the tanks allowing entry to lizards, frogs and other wildlife. These were corrected in a number of cases with good results. Instructions for these tanks stress the necessity and function of these seals but it is likely that training is required if the seals are to be successful.

- Overflows were not placed correctly allowing ingress of floodwater through the overflow.

- Several sites had no trench to divert floodwater, again allowing floodwater into the tank. Others had a trench that became filled with soil. Modifying the trench by lining it with mortar encourages smooth flow of floodwater around the tank, which prevents erosion and reduces the tendency for the trench to fill with soil. It also highlights the need for the trench.

Sites with appropriate overflow and floodwater drainage arrangements worked well and water was being used for high quality purposes. Sites with poor overflow and floodwater arrangements tended to be used for secondary water.

Corrections being made to seals in Uganda

Good drainage practice on thatch tank in Ethiopia

Good top seals and tar sheet cover used in Sri Lanka

DESIGN CHANGES

Lining the trench with mortar improves water flow around the tank

The ring beam can be made to stand proud of the ground by about 10cm using bricks as a temporary formwork, – this should go some way to reducing floodwater ingress problems and simplify overflow arrangements.

The thatch can be replaced by more durable – yet inexpensive materials such as tar sheet or other local materials. The flexibility of materials should present few problems so long as the material can protect the polyethylene from UV light and wind-born dust and effectively keep light out of the tank.

8.3.3 TUBE TANK

PROBLEMS

The tube tank also suffered from poor installation. Tubes were sometimes installed in rocky ground or in some cases in holes that were too large causing the polyethylene tube to take the full pressure of the water resulting in failures of tubes. Some tube tanks were made without overflow or slabs installed without proper seating and were damaged. Other tube tanks that were installed carefully and correctly fared better. Tube tanks in Ethiopia used a thin polyethylene as thicker material was not available and several failed after one season. There were a number of problems with the tube, mainly in handling and installation. The tubes are however an inexpensive component and easily replaced so can be considered disposable after a year or so.


Tube tank in use in Sri Lanka


Tube tank made without overflow in Sri Lanka


Mould developed in Ethiopia for making platters which reduced time to make a platter from 1 day to 1 hour

DESIGN CHANGES

Several modifications were made to the platter that reduce the cost including the removal of the basin, inner-tube seal and steel bracket, replacing them with having the cloth and tube tied directly to an upstanding ring in the slab. The steel mould developed by Water Action in Ethiopia has also reduced the slab cost.

8.3.4 BARREL

PROBLEMS

The barrel tank had few problems. It was, however fairly expensive in most instances compared to the other designs, the costs dominated by welding. The barrels themselves suffered from a lack of any economies of scale as they are used in quantity, so while one barrel is relatively competitive, a number of connected barrels become less so. Some tanks suffered from rusting as they had not been painted inside but those that were painted fared well. Some users complained of a “gas” taste to the water, which faded with time. The filters worked well and rarely overtopped.


Barrel tanks being delivered by truck (Picture: B. Woldemariam,)


Horizontal variant in use in Sri Lanka (Picture T. Ariyananda)

DESIGN CHANGES

Some households had roofs that were too low to accommodate the standard vertical design. A horizontal design was developed for these households that also reduced the cost, as it did not incorporate as much welding.

8.3.5 CRATE TANK

Horizontal crate tank in Sri Lanka

Vertical variant in use in Sri Lanka

Fewer crate tanks were tried than other designs due to uncertainty about their longevity. Crates were tried in urban areas, particularly in Sri Lanka in both vertical and horizontal configurations. Several tanks failed due to water pressure pulling out nails, however these tanks were made with fewer nails than initially specified. Later tanks were modified and stood up to the water pressure well. The horizontal design, however suffered problems with the waterproof insert puncturing on the inside of the tank and with water rotting the boards. The tanks were also not particularly cheap when manufactured under field conditions.

8.3.6 MUD TANK

Completed mud tank in Sri Lanka

Twin Mud tanks in Sri Lanka (one built under the project – the other by the householder)

The mud tank was tried in a few households in Sri Lanka. Only smaller tanks were tried in this instance as no reliable method of joining plastic sheet in the field was found and it was felt best to prove the design with smaller capacities before larger models were tested in the field. One tank failed when the bamboo reinforcing sheered off at the base. Others, which used timber for reinforcing rather than bamboo, are standing and holding water. The mud walls are liable to washing from rain, however they are easily repaired and a larger roof will reduce this effect. The tanks are very inexpensive, however and if they prove reliable over a period and the problem of larger tanks can be solved, could reduce the cost of ownership considerably.

8.4. USER OPINIONS AND REPLICATION

After using the tanks, householders were asked their opinions about them and to rank the designs as well as their other water sources using a pair-wise ranking exercise. The number of times a particular design was selected as the "winner" in the exercise was counted and the count divided by the total number of pair-wide trials of which that design was a part. This resulted in a score for each design at each site with the total adding up to one. The results are in Table 8.2. The larger below-ground designs fare best, followed by the above-ground designs. This was borne out by a further exercise whereby participants were offered a flat subsidy that would cover a tube tank but could upgrade to any other design by contributing themselves. Of those that upgraded, most upgraded to Dome tanks or thatch tanks with a view to further upgrading to dome tanks in the future. There have also been reports of replication of dome, thatch and mud tanks from surrounding households.

Table 8.2: Users rankings of designs

Type	Fraction of votes						comments
	Ethiopia		Sri Lanka				
	Arerti (peri-urban)	Alaba (peri-urban)	Colombo (Urban)	Aranayake (Rural)	Ambanpola (Rural)	Average score	
Dome		0.8		1.0	1.0	0.9	Well received
Mud				0.8	0.5	0.7	Well liked as water is cool – but not seen as "permanent"
Drum		0.6	0.7			0.6	Well received but could be expensive, seen as too small
Thatch		0.3		0.6	0.8	0.6	Mainly seen as a step to the dome tank
Crate			0.3			0.3	Difficult to clean and maintain, seen as too small
Tube	0.5	0.3		0.4		0.3	Still some unresolved problems, seen as too small, concerns about children falling in
Tap stand	1.0	1.0	1.0			1.0	
Well			0.0	0.2	0.3	0.2	
Pond	0.2				0	0.1	
River	0.0	0.0		0.0		0.0	

Replication and adoption into programmes has occurred with several of the designs

- According to a paper presented to the 12th IRCSA conference by Weerasinghe The Thatch Tank has been adopted for agricultural programmes in Sri Lanka [Weerasinghe et al., 2005]
- At the same conference Nakanjakko, described the use of dome tanks in southern Uganda [Nakanjakko & Kamugasha Karungi, 2005]
- During the field trials several mud tanks were made by householders in the locations where trials were being carried out
- Thatch tanks were also built by householders in Sri Lanka – mainly as a step towerd ownership of dome tanks
- There were reports of intentions to build dome tanks by some householders in Southern Uganda
- Drum tanks were featured in the Sri Lanka section of the UNEP brochure on Rainwater and the millennium development goals [Khaka et al., 2005]

8.5. TANK MAINTENANCE AND NET PRESENT VALUE ANALYSIS

The experience of using the tanks over a period of time has allowed a good estimate of the required maintenance to be made. Designs such as the dome tank and the drum tank require little maintenance, however the other designs require maintenance over their lifetimes. The estimated maintenance tasks and costs are detailed in Table 5.2.

Table 8.3: Servicing costs for highly reduced-quality tanks

	Servicing task	Frequency (years)	HH Time (days)	Cost in Sri Lanka	Cost in Uganda
Thatch tank	Inspect and repair thatch cover	1	2	£1.64	£1.00
	Replace cover plastic	2	1	£1.54	£4.93
Tube tank	Replace tube	1	1	£4.85	£4.45
Mud tank	Inspect and repair mud walls	1	2	£1.64	£1.00
	Inspect and repair thatch cover	1	1	0.82	0.5
	Replace tube	3	1	£4.85	£4.45
Crate tank	Replace crate	5		£24.77	
	Replace tube	3	1	£1.45	

Having established the maintenance costs over the life of the tank the net present value can be determined using the methods outlined in Chapter 5. Investment cost was taken as the reported field cost modified to take design changes into account and the life of the tank is assumed to be 20 years. The scenario used is the same as in Section 5.3.3; a house in southern Uganda with a roof size of 34 m^2 containing an average of 7.8 persons each demanding 20 litres/day. Net present value is calculated using 20 years discounted benefits and costs with benefits accruing from time saved collecting water and costs being the investment and maintenance costs of the design being considered. Internal rates of return are also shown as is the crossover rate. The crossover rate was developed in chapter 5 and is the rate which, if applied to both the new designs and to a benchmark design give the same NPV. If the discount rate being used by the customer is lower than the crossover rate, the higher investment – lower maintenance design is more attractive, if the customer is using a higher discount rate (either objectively or implicit), the lower cost – higher maintenance design will be more attractive. A discount rate of 5% p.a. is taken to simulate objective discounting as used by a water provision agency and a discount rate of 50% p.a. is taken to simulate subjective discounting by a poor household. As in Chapter 5, the benchmark used is the Sri Lankan pumpkin tank which is a well designed and evolved ferrocement design whose costs are at the bottom of the "normal" range. The investment cost for this design is scaled according to the cost:capacity sensitivity of $V^{0.65}$ as discussed in Section 3.3 and the design is assumed to be free of maintenance costs.

Table 8.4: Internal rates of return and crossover rates for the new designs

		Pumpkin tank*		Dome tank	Drum tank	Thatch tank	Mud tank	Tube tank	Crate tank
Capacity (m^3)		5	1	5	0.5	5	0.8	0.7	1
Investment cost	SL	£68	£24	£40	£26	£33	21	£17	£35
	Ug	£105	£37	£55	**	£41	19	£18	**
Equivalent unit cost	SL	£24	£24	£14	£41	£12	£24	£21	£35
	Ug	£37	£37	£19		£14	£22	£23	
NPV at 5% p.a. discount	SL	£513	£684	£541	£552	£787	£606	£426	£579
	Ug	£248	£394	£298	£325	£434	£357	£239	£315
NPV at 50% p.a. discount	SL	£68	£90	£96	£67	£99	£80	£87	£69
	Ug	-£22	£32	£28	£30	£36	£42	£42	£27
IRR	SL	100%	238%	170%	178%	201%	245%	311%	153%
	Ug	39%	94%	75%	108%	95%	163%	170%	91%
Crossover discount rate	SL	[redacted]		BFA	WFA	3%	WFA	68%	WFA
	Ug			BFA	WFA	1%	21 %	23%	WFA

* Existing FC design used as a benchmark standard
** where Ugandan cost data is unavailable, investment cost is assumed the same as Sri Lanka
BFA = better than benchmark for all discount rates
WFA = worse than benchmark for al discount rates

As expected, smaller tanks give higher returns, however the smaller reduced-quality tanks do not fare so well against the ferrocement design. This is again indicative of the high fixed cost some of these designs carry.

- The dome tank is better at all rates due to its low cost and low maintenance
- The drum tank is worse at all rates despite low maintenance, due to its high cost
- The crate tank combines a high cost with high maintenance and so is worse at all discount rates
- Despite the need for maintenance, the thatch tank has a crossover discount rate of less than 5% due to the concentration of maintenance in the cover which is quickly, easily and cheaply maintained despite the frequency needed.
- The relatively high investment cost of the tube tank results in a fairly high crossover rate, particularly in Sri Lanka where cement is comparatively cheap. In Uganda, where cement is more expensive, the design has a crossover rate of 23% making it attractive to poor households but not to a water agency.
- The mud tank shows a similar trend with NPVs less than the pumpkin tank for all discount rates in Sri Lanka and a crossover rate of 21% in Uganda. In this case, the reason for this difference is the large amount of labour in servicing the tank which is comparatively expensive in Sri Lanka. The design also suffers from its small size. Results are quite different when a scaled 3 m^3 tank is considered; the crossover rate in Sri Lanka is 34% and in Uganda, 10%.

8.6. CONCLUSIONS

Many of the designs have proven themselves in the field and are being taken up by users and water supply agencies. The effectiveness of the underlying strategy of quality reduction is also demonstrated with some designs where the quality has been reduced slightly displaying lower cost than competing designs with no difference in servicing needs. Other designs where the quality reduction involved less durable materials and high household labour at the expense of higher servicing costs remain attractive when this servicing is considered as a cost discounted over time. The attractiveness is, however, dependent on how much the future is discounted and is often sensitive to labour opportunity cost. NPV analysis over a range of discount rates reveal a crossover rate where the lower-cost, higher maintenance design becomes more attractive than the higher cost, more durable design. The crossover rate for many designs is between that used by water agencies and subjective discounting used by households so reduced-quality designs will be more attractive to poor householders than those agencies.

9. INFORMED CHOICE AND THE ROOFWATER HARVESTING LADDER

While reduced-quality designs have been demonstrated to be lower in overall cost than more durable designs, their use in formal water programmes may cause resentment – especially when it is being offered by a “rich” agency. If householders are exposed to trade-offs, this can form the basis of a dialogue that can result in them choosing a technology that matches their needs and abilities. The sanitation ladder is a method of introducing different qualities of sanitation systems to a community to instigate such discussions of where on the ladder a community is – where they want to be and how to get there. As introduced in Section 4.1, a similar ladder approach can be used for rainwater harvesting systems, although there must be two dimensions to climb rather than one (quality and quantity). This chapter deals with how such a ladder can be constructed and how it can be used.

9.1. DECISION FACTORS

WATER SUPPLY ORGANISATIONS

The most important factors to a water supply organisation are cost (primarily up-front cost, since maintenance is typically done by the household), and the direct benefit from the water collected by the system (and knock-on benefits in saved time and/or additional water use in the home). The benefit side of the equation itself has two components; how much each household can gain from its DRWH system and how many households can be served with systems. The second of these will depend on the total amount of money available to build systems and the cost of each system. In general, choosing a smaller or lower-quality system will only slightly reduce delivery per-system yet substantially increase the number of households servable.

HOUSEHOLDERS

The most important factors to a householder may be different. Up-front cost is of course important where there is a significant household contribution (or the tank is bought outright by the household), however in recent years many tropical DRWH systems have been so heavily subsidised that their beneficiaries have had little interest in their cost. The actual amount of water that can be delivered by the system is difficult for householders to predict, so tank size is used as the main measure of water delivery potential. Householders, though, often fail to realise that doubling the size of a tank does not double the water it can deliver in a year. Prestige is also important to householders – a visible water tank can be a considerable household asset and a large, high-quality tank can enhance the householders' standing in their community.

While there is considerable crossover in these goals, for a fully informed choice, householders should be exposed to information on:

- How much water can be gained from a range of system sizes and how this water will be distributed over the year
- The effects of different water management strategies on the water gained from the system
- A range of designs of different quality
- The trade-offs of tank cost and system coverage or household contribution
- Any cash or labour contribution expected from the household

Systems of different sizes and qualities should be clearly presented along with a number measures of water provision. A community or household can decide on the solution that is best for their needs.

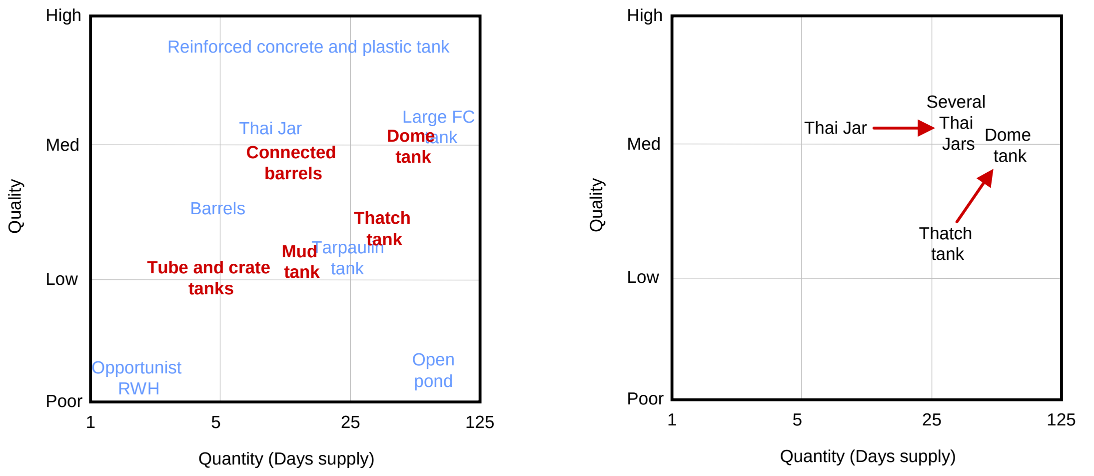
9.2. THE LADDER APPROACH

The "sanitation ladder" is a tool for working with communities in participative planning of on-site sanitation projects. It was invented by Uno Winblad and popularised as part of the WHO Participatory Hygiene and Sanitation Transformation (PHAST) toolkit [WHO/AFRO, 2001]. It basically consists of a catalogue of designs of incremental quality and cost from which a project manager, a community or individual can select an appropriate well designed sanitation system to suit local conditions and the available funds. Such “ranges” are, in fact, the norm in consumer products and usually form the basis of consumer choice.

Rainwater harvesting systems are very amenable to this product range approach, as uncertainties about geology and topography don’t exist; the water simply falls from the sky. They are however, slightly more complex than sanitation systems as there are two dimensions, *quantity* of water provision (a function of tank volume – but not just the volume itself) and *quality* of construction. An increase in either of these dimensions will increase cost. It is in fact the quality aspect that is predominant in the sanitation ladder whereas roofwater harvesting systems are often dominated by the question of quantity (usually, in the form of raw tank volume) with a certain construction quality taken as read.

Figure 9.1: New designs and the roofwater harvesting ladder

a. How new design map onto the ladder b. Paths “up” the ladder



Note: the faint blue titles correspond to exiting designs, bright red titles, new designs.

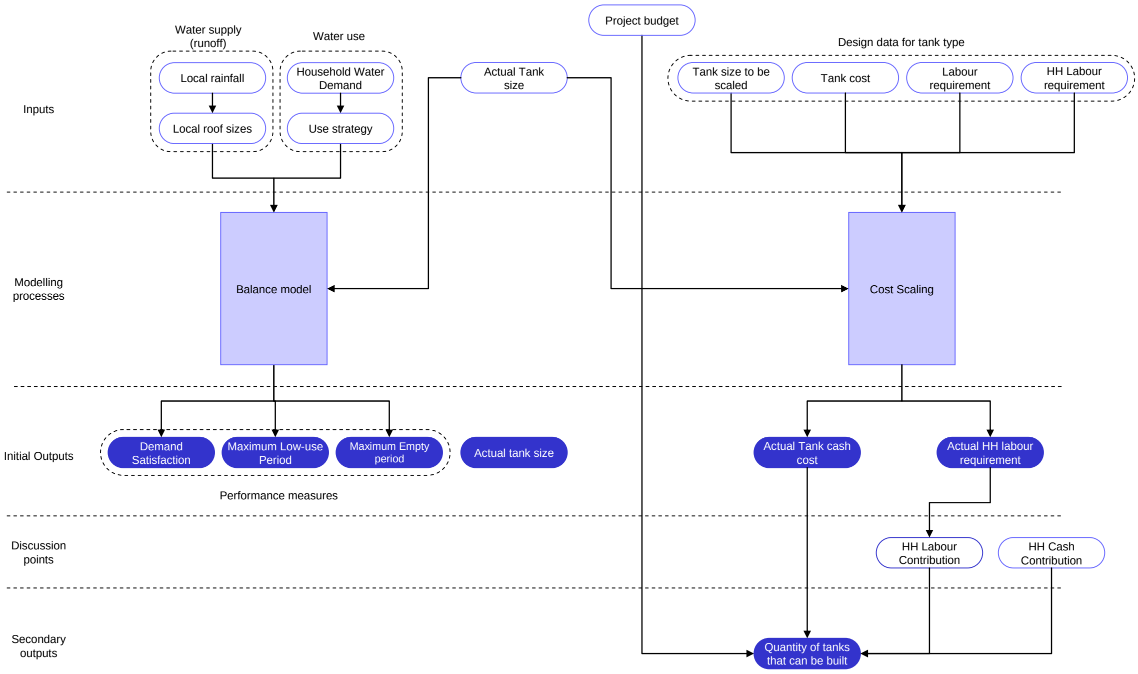
Figure 9.1 shows how the new designs map onto the roofwater harvesting framework introduced in Chapter 4. This framework can be seen as a kind of two dimensional ladder which “climbs” in quality and quantity dimensions. A currently used method of climbing the ladder is incremental adoption such as the gradual building of a series of Thai jars – thus moving up the ladder in quantitative terms. This works well and a small jar does a great deal to demystify DRWH and encourage further adoption. It does, however suffer from a lack of economies of scale and may ultimately be a fairly expensive option in the long-term. A new form of incremental adoption has been developed under this research, where a householder can start with a relatively cheap but low-quality tank such as the thatch tank and then upgrade this tank to a dome design – moving upward slightly in quantity terms but mostly upward in quality terms.

The two-dimensional nature of DRWH service provision makes the analogy of the "ladder" slightly clumsy as one must "climb" from one corner to another rather than just straight up as in the sanitation ladder, however the term has been retained due to its familiarity to water and sanitation personnel.

9.3. INPUTS AND OUTPUTS

In order for the community to make an truly informed choice among technologies, they will need information about how different sizes of system behave, as well as the costs and trade offs involved in different designs. Figure 9.2 shows the steps to produce such information.

Figure 9.2: Steps to produce a roofwater harvesting ladder



The size/performance relationships will be the same regardless of actual tank design so can be presented once for all designs. The actual performance measures presented are, *demand satisfaction*, *maximum empty period* and *maximum low use period*. These measures are intended to replace the simplistic and misleading *tank size* measure and are described in Section 2.4.2. To produce the performance outputs, information on rainfall, average roof size and a breakdown of average household water demand are needed.

The tanks themselves must also be presented, either as a picture, or better with example tanks built and in use. Cost information should also be presented either as a direct cost or a regular payment (if full cost recovery is a goal) or as a number of systems that can be built (if a fixed donor-led grant is being used), or some combination of the above. Section 3.3 discusses the calculations necessary to scale an existing tank design if a range of bills of materials are not available.

9.4. PREPARING THE LADDER

Having calculated the water delivery of various tank sizes and having arrived at the scaled costs and labour fractions, the final ladder can be prepared for presentation. Figure 9.3 is an example of a completed ladder of three tanks for a project with a fixed budget. RWH Ladders can, of course, have more tank choices; sanitation ladders tend to have about seven.

The top rows are generic descriptions of tanks of various sizes. It is possible to present here the results of more than one usage strategy, e.g. a fractional demand and a variable demand. These figures should not change regardless of tank type.

The following rows are dedicated to tank type. They should include:

- A picture of the tank (ideally, the community should also have access to examples of the actual tanks)
- Cost if cost recovery is being sought or number that can be built if they are to be subsidised (or given free) from a fixed budget.
- Any cash contribution required of the householders
- Any household labour contribution required of the householders

The table can either be presented "as is" or can be altered during discussions of trade-offs with the community, e.g. if the household cash contribution were raised, would it be possible to afford better or larger tanks, and what impact would this have on the overall water picture? Is it better to have smaller-higher quality designs or larger, lower quality designs? Is a mix of designs appropriate – how will these be apportioned? Discussions can also be had on moving from one quality to another or the relative merits of adopting technologies incrementally ("climbing the ladder"), either by starting small or by using tanks such as the thatch tank as an interim measure.

Figure 9.3: An example of a roofwater-harvesting ladder

		1,000 litre	2,000 litre	5,000 litre	10,000 litre
Based on 40 litres per day	Demand satisfaction	61%	68%	79%	94%
	Max empty period	163 days	151 days	113 days	51 days
Based on variable demand*	Demand satisfaction	66%	77%	86%	92%
	Max empty period	135 days	112 days	37 days	0 days
	Max low-use period	83 days	98 days	159 days	147 days
Design 1: Pumpkin tank					
	Number of tanks that can be built by the project	1,000	680	410	280
	Labour contribution (per tank)	6 days	8 days	13 days	19 days
	HH cash contribution (per tank)	0	0	0	0
Design 2: Dome tank					
	Number of tanks that can be built by the project	2,000	1,360	820	560
	Labour contribution (per tank)	6 days	8 days	13 days	19 days
	HH cash contribution (per tank)	0	0	0	0
Design 3: Mud tank					
	Number of tanks that can be built by the project	3,000	2,040	1,230	840
	Labour contribution (per tank)	9 days	14 days	24 days	35 days
	HH cash contribution (per tank)	0	0	0	0

* based on nominal demand (40 l/d). if tank is more than 2/3 full, 80 litres/day; if it is less than 1/3 full, 20 litres per day; otherwise, 50 litres per day.

PART TWO: HEALTH AND WATER QUALITY INTERVENTIONS

10. HEALTH RISKS FROM DRWH SYSTEMS

Apart from cost, the largest concern for water supply professionals is the effect of roofwater harvesting systems upon public health. Initial concerns involve health risks from users drinking and washing in the water, but as systems become more widespread there is the possibility of them affecting the community as a whole through a rise in the number of viable containers for disease carrying mosquitoes.

Part Two of this thesis is concerned with the health impacts from roofwater harvesting systems, primarily with concerns about water quality. This Chapter discusses water quality in general terms. The risk pathways of roofwater harvesting systems are discussed and a literature review of water quality and vector-based health impacts is given. Chapter 11 carries this further and discusses the effects of water quality interventions on RWH systems. Chapter 12 considers one of the more promising of these interventions; namely “first-flush” diversion, whereby the dirtier first part of the storm runoff is separated. particular emphasis is given to quantifying the necessary volume that needs to be discarded. Chapter 13 considers the effect of discarding the first flush on the overall water delivery of the roofwater harvesting system.

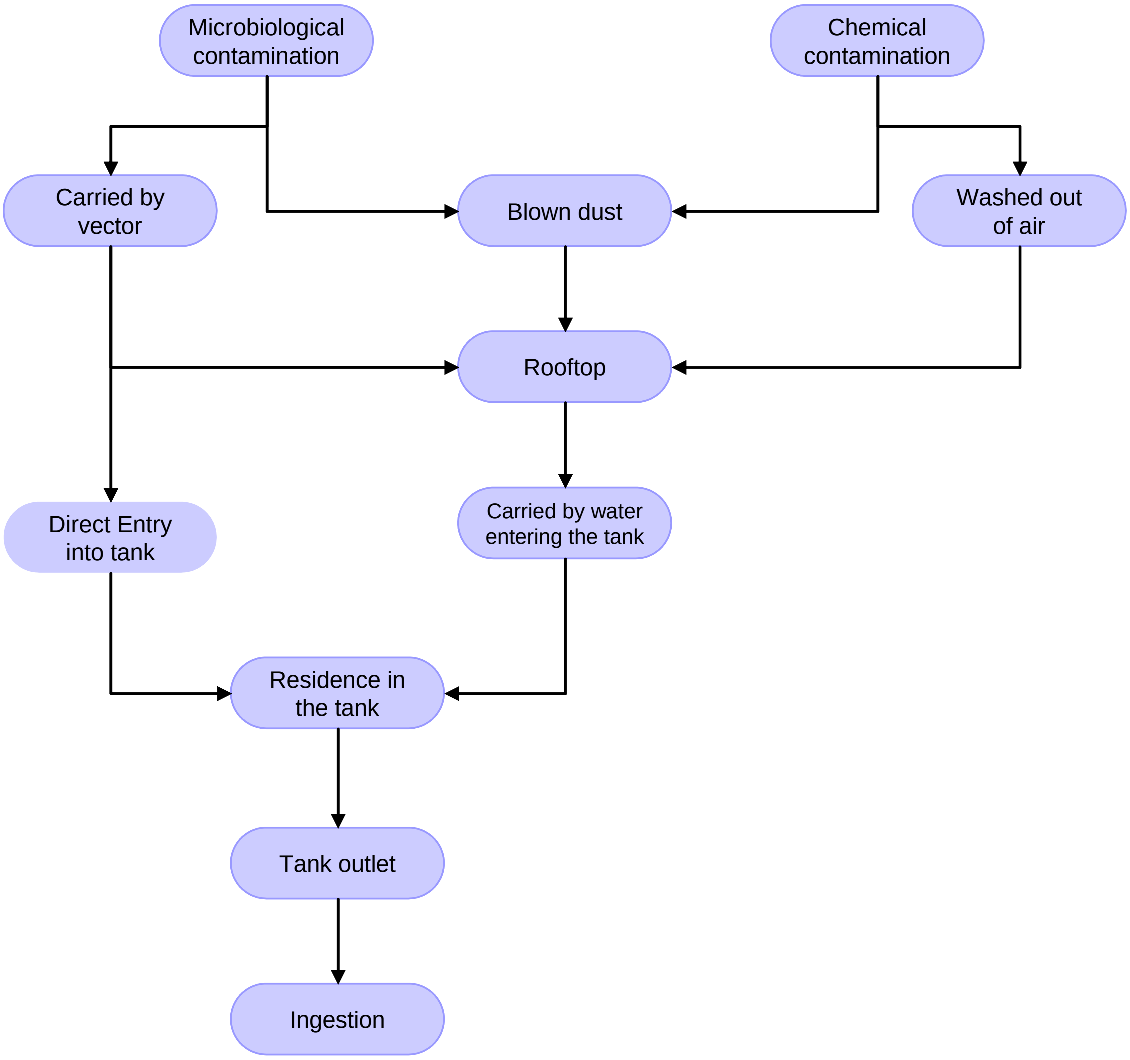
Roofwater harvesting systems generally have a good record as far as public health is concerned, however there is a very small database upon which to draw actual health cases due to their rarity and the strong likelihood of under-reporting. A number of studies have been done on the water quality itself using indicators and several key pathogens, some of these include measurements at different stages along the contamination path. A number of reviews done over the years [Latham & Schiller, 1987; Lye, 1992; Vasudevan & Pathak, 1998; Gould, 1999; Lye, 2002;

Sinclair et al., 2005; Meera & Ahammed, 2006], however most of these reviews report findings without reporting any the circumstances that may have contributed to those results or the contamination paths that are important to roofwater harvesting. This chapter specifically investigates the contamination paths and the changes that take place in the water quality as it flows through these paths.

10.1. CONTAMINATION PATHS IN DRWH

When considering the water quality of a roofwater system, it is useful to observe the path a contaminant must follow in order to enter a human being. The usual paths available are shown in Figure 10.1.

Figure 10.1: Contamination paths for roofwater harvesting



The paths are complex and generally represent a hostile environment for microbiological contaminants and present a number of barriers to some chemical contaminants. Means of enhancing these contaminant- reduction processes are discussed in Chapter 11. The primary means of tank contamination is through water washed in from the roof, although direct entry either via a vector (such as a rat, lizard or insect) or via an accident (such as spillage of raw sewage into the tank) is often the reason for outbreaks of reported disease. This is mainly because such direct entry paths “short circuit” the complex path usually followed by incoming contaminants.

Material washed in from the roof can come from several sources:

1. By far the largest contribution will come from material that has accumulated on the roof or is blown onto the roof during the storm itself. The accumulated material may have been blown onto the roof by the wind, stirred up by passing vehicles, dropped from overhanging trees or deposited by an animal (or person) with access to the roof.

2. If the roof is made of decayed materials, the roof itself can contribute to the dirt load. This is particularly true of low-quality roof materials such as thatch or tar sheets, though asbestos sheeting and galvanised iron (particularly if it is rusty) can also add material to incoming water.

3. Passage of water along uncleaned gutters may add further debris. If allowed to enter the tank, this usually organic material will form a good source of nutrition for bacteria and insects and may itself harbour some microbiological contamination.

Residence in the tank provides opportunities for water purifying processes such as sedimentation and bacterial die-off to take place, increasing the water quality over time. If, however, the tank is poorly designed, built or maintained, the storage time may provide further opportunities for pollution to enter the water. If light is allowed to enter the tank, (particularly if it is open-topped) an active ecosystem may develop in the tank resulting in stagnant water of very poor quality.

10.2. BIOLOGICAL CONTAMINATION

10.2.1 BACTERIAL WATER QUALITY BY INDICATORS

There have been a large number of studies measuring various indicator bacteria, such as heterotrophic plate counts (HPC) total coliforms (TC) or thermo-tolerant (faecal) coliforms (FC), in rainwater systems [Bunyaratpan & Sinsupan, 1984; Haeber & Waller , 1987; Fujioka, 1987; Bambrah, 1997; Ariyananda, 2001; Abo-Shehada et al., 2004; Ariyananda, 2005] etc. Most have shown indicator bacteria exist in rainwater tanks in some quantity but that this quantity has large variations both from system-to-system and over time with readings changing several orders of magnitude within a few days.

There is a growing body of evidence, though, that the number of detected indicator bacteria is not well correlated with pathogens [Rinehart et al., 1985; Crabtree et al., 1996] resulting in both false positives and false negatives. Inspection of the tables in Section 10.2.1 also shows that for the largely or *animal* sourced pathogens most likely to be found in Rainwater tanks, indicators developed to detect risk of *human* faecal contamination are often not considered appropriate by authorities such as the WHO or the UK Environment Agency. It is likely that in rainwater tanks, the detected coliforms merely indicate the presence of opportunistic environmental bacteria such *Bacillis* species [Coombes et al., 2005] which are not considered dangerous and there is even some evidence that they form an important part of the beneficial biofilms that line the walls of rainwater tanks [Lücke, 1998; Spinks et al., 2003]. There is also strong evidence that *E.coli* species can survive and multiply in tropical soils [Hardina & Fujioka, 1991; Byappanahalli & Fujioka, 1998]. In view of the potential for environmental bacteria to produce false positives, relaxed *E.coli* water quality criteria have been proposed [Fujioka, 1993; Krishna, 1994], however in view of the possibility of false negatives it may not be wise to rely solely on these measurements. A survey of potential risk pathways such as overhanging trees, an excess of vermin, access by pets etc. and verification of appropriate system design and construction is likely to be easier and may be as accurate as indicator measurement.

10.2.2 CHANGES IN BACTRIA WITHIN DRWH SYSTEMS

While of uncertain value from a public health monitoring perspective, indicator organisms still have value as a research tool as they show a *risk* of bacteriological contamination and will give some indication of how this risk changes throughout the system.

Several studies have measured the water quality at various points along its path [Bunyaratpan & Sinsupan, 1984; Wirojanagud, 1987; Coombes et al., 2000; Handia, 2005]. These give a good indication of the changes that take place. The results from three studies are shown in Table 10.1 and show a clear increase in bacteria from rain collected from the roof, followed by a reduction in bacteria collected from the tank outlet.

Table 10.1: Results from three studies showing changes in indicator bacteria at locations along the water path in rainwater harvesting systems

Location	Rain	Roof runoff	Tank outlet	Source
TCs				
NSW*	0	359	18-200	[Coombes et al., 2005]
Zambia**	0-27	30-TNC***	0-30	[Handia, 2005]
Thailand****	93%	100%	60%	[Wirojanagud, 1987]
FCs				
NSW*	0	135	<1-20	[Coombes et al., 2005]
Zambia**	0-5	15-TNC	0-6	[Handia, 2005]
Thailand***	33%	32%	30%	[Wirojanagud, 1987]

* range of averages from 3 sites
** range from 5 sites
*** Too Numerous to Count
**** % of samples not meeting criteria (FC<1, TC<2.2 cells/ml by MPN)

The reasons for these changes become clear when the path of contamination is investigated. In microbiological terms, rainwater itself is of excellent quality with indicator counts generally very low. Material and organisms which accumulate on surfaces are washed off by the rainfall and are then transported into the water store. Roofs are elevated and as a result tend to exhibit cleaner water than ground catchments [Duncan, 1995] and this study is focussed on roof-water harvesting precicely because of the possibility of roofs providing water of drinking quality. When roof and ground-based runoff are mixed high levels of contamination are often found; e.g. Zhu [2004] found high levels of indicator bacteria in RWH systems in northern China where a courtyard is used as a catchment. The high levels were attributed to access to the courtyard by stock animals. Moreover, when an underground tank is insufficiently protected or open at ground level allowing ground runoff into the tank, increased levels of bacteria are to be found [Abo-Shehada et al., 2004]

The roof itself (particularly if it is made from steel) is an extremely hostile environment for faecal bacteria which have evolved to live in a warm, wet, low-oxygen environment. The dry heat typical of a metal roof under bright sunlight will effectively kill many of these pathogens. This effect is borne out by the usually lower microbiological indicator levels from metal roofs as compared to other roof types [Yaziz et al., 1989; Vasudevan et al., 2001b]

These organisms are then washed into the tank where changes take place in the tank. and over time the contamination levels reduce due to settlement, and die off within the tank. Tank die-off is further discussed in Section 11.4 and provides an explanation both for the changes in indicator bacteria between roof and tank outlet and also for the noisiness of indicator and pathogen data.

10.2.3 PATHOGENS AND ILLNESSES ASSOCIATED WITH DRWH

A number of epidemiological studies have been reported in the literature. Those where the alternative is untreated or ineffectively treated water tend to report consumption of rainwater harvesting as a protective feature, whereas several other studies have shown consumption of DRWH to be detrimental. Table 10.2 shows a summary of these studies which are mainly in high-income counties. The only study in a low-income country [Shier et al., 1996] is fairly tenuous.

Table 10.2: epidemiological studies reported in the literature

Location	Illness	Notes	Source
South Australia	Gastroenteritis	Diary study of 1016, 4-6 year old children. Found no increase in odds for rainwater consumption (odds ratio 0.84). Postulated that immunity may play an important part in reducing illness in regular rainwater users. Only 40% of tanks had a screened inlet, 65% of gutters had been cleaned recently, 42% of tanks had never been cleaned.	[Heyworth et al., 2006]
Ghana	Diarrhoea	Study focussed on boreholes but found significant seasonal variations in illness by season. Rainy season morbidity was lower than dry season reflecting a increased use of "traditional water sources – including rainwater. It is likely that only opportunistic roofwater harvesting was practiced, although this is not mentioned in the study	[Shier et al., 1996]
South Australia	Cryptosporidial diarrhoea	Questionnaire study of 51 cases of Cryptosporidial diarrhoea. Results suggest a protective effect of rainwater consumption ($P<0.005$)	[Weinstein et al., 1993]
Hawaii	Leptospirosis	Statistically significant ($P=0.003$) association between Leptospirosis and consumption of rainwater. Other significant factors were presence of skin cuts ($P=0.008$), contact with cattle ($P=0.05$), and handling animal tissues ($P=0.005$).	[Sasaki et al., 1993]
New Zealand	Campylobacteriosis	Rainwater consumption associated (odds ratio 2.20) with an elevated risk of Campylobacteriosis in questionnaire study of 621 cases. Other signigicant factors wer consumption of chicken (odds ratio 3.85 - 4.52), consumption of raw dairy products, and contact with puppies and cattle	[Eberhart-Phillips et al., 1997][1]

For a bacteria to infect a human it must pass through the same paths indicated in Figure 10.1, it must then survive in the tank and be ingested in sufficient quantity to cause infection. Table 10.3 shows a number of potential pathogens that can be transported by water to humans. Of these the human-specific pathogens may safely be ignored as in most cases it is unlikely that

[1] Original paper could not be obtained, however a good summary is given in [Sinclair et al., 2005]

human faecal matter will find its way into the tank. Human defecation on sloping roofs is uncommon and even on flat roofs is unusual. However there are extreme circumstances (e.g. in shanty towns) in which human faeces are actually thrown onto roofs – roofwater harvesting is not viable in the land of the “flying toilet”. Where open defecation is practiced, a possible path is via faecaly contaminated dust blown onto roofs; however it is unlikely that live bacteria will survive the faces drying out before it is light enough to be blown by wind. This is borne out by lack of reports of such diseases as hepatitis, dysentery, cholera etc. which are transmitted via *human* faecal matter. Similarly livestock-based pathogens can be discounted as it is unlikely that livestock will soil the roof.

Table 10.3: Possible pathogen transport routes (Modified from Lücke [1998])

Pathogen type	Example	Entry into tank	Survivability in tank water
Pathogens specific to humans	*Salmonella Typhi, Salmonella Paratyphi*	Sewerage accident	Low
	Shigella		Low
	Viruses pathogenic to humans		Low
	Vibrio cholerae group	Impossible	Low
Pathogens adapted to domestic animals	*Enterohaemorrhaging E.coli*	Accident involving feaces from livestock	Low
Pathogens adapted to birds of small animals	*Salmonella Typhimurium*	Possible	Low
	Campylobacter		Low
	Giargia		Good (cysts)
	Cryptospiridium		Good (oocysts)
Pathogens that also operate as opportunistic bacteria	*Pseudomonas aeruginosa*	Small	Moderate
	Entobatreiaceae (such as Yersinia)		
	Mycobactreium		
	Legionella		
	Aeromonas		

This leaves the main danger bacteria of roofwater harvesting, namely those bacteria that have the opportunity to enter the systems such as *Salmonella* species and organisms that can survive in the low-nutrient environment of the tank, primarily cysts such as *Giardia* and opportunistic bacteria such as *Legionella*.

Several pathogens have been isolated to roofwater tanks however their results are variable with tanks alternately showing negative and positive results and wildly differing counts over time. As mentioned in Section 10.2.1, most pathogens found show little correlation with indicator bacteria. There are a handful of reported cases of illness associated with RWH systems. The low number of outbreaks is partially because well-maintained RWH systems tend to give fairly clean water and partially because outbreaks are confined to one RWH system and do not become widespread as with centralised water supply, so outbreaks tend not to be reported unless they involve a large number of people or take place on commercial premises [Koplan et al., 1978].

SALMONELLA

Salmonella is an anaerobic bacteria of the family Enterobacteriaceae with over 2000 subspecies only some of which are infective to humans. They are flagellated so have the ability to move in tanks in response to stimuli. Survivability in surface waters is a few hours or days [EA, 2002] and in rainwater tanks in the order of 3 days [Holländer et al., 1996]. The most important species from a water born disease perspective is *S. Typhi* which causes typhoid, however this species is restricted to human hosts and no cases have been associated with rainwater consumption in the literature. *S. Typhimurium* and *S. Enteritidis* are also infective to a wide range of animals including birds, reptiles and small mammals [WHO, 2004]. Thermo-tolerant coliforms are considered reliable indicators for salmonella [WHO, 2004].

Several studies have isolated samples of these species from rainwater supply. [Rinehart et al., 1985; Nevondo & Cloete, 1999; Simmons et al., 2001], however other have failed to find any [Thurman, 1995] or only one or two cells/sample [Wirojanagud, 1987; Holländer et al., 1996]. The effective dose for a healthy individual is millions of cells, except for *S Typhi* which only requires a few cells to be infective [WHO, 1996].

Non-Typhoid salmonella infections are rarely from a water source, food contamination being much more common; however some rainwater sourced infections have been reported. Table 10.4 shows the reported out breaks, most cases are of salmonella, delivered via the faeces of a bird or small mammals with access to the roof surface or to the tank. All cases were non-fatal and tend to come from examples of bad practice.

Table 10.4: Salmonellae outbreaks reported in the literature

Location	Bug	Notes	Source
Trinidad	*Salmonella arechevalata*	*S. arechevalata* isolated from rainwater supply but not tank. Strong associations with use of water from the tank and infection. Postulated as originating from "dried and fresh bird droppings" on the roof of the building.	[Koplan et al., 1978]
Queensland	*Salmonella saintpaul*	*S. saintpaul* isolated from the rainwater tank and strong associations with use of water from the tank and infection. Presumed route was either via washed in mouse faeces or via "several large frogs" which had entered via an uncovered inlet or some combination of these factors	[Taylor et al., 2000]
New Zealand	*Salmonella typhimurium*	*S. typhimurium* isolated from the rainwater supply but the contamination path was not identified	[Simmons & Smith, 1997]

CAMPYLOBACTER

Campylobacter bacteria of the family Spirillaceae and is the most important water-borne pathogens in high-income countries [EA, 2002] and among the most important wordwide [WHO, 2004], Campylobacteriosis cases are, however described as "self limiting" and pass within days [WHO, 2004]. *Campylobacter sp.* are motile by a single flagellum and are micro-aerobic requiring 3-6% oxygen for growth [WHO, 1996]. They are also capnophilic requiring increased levels of carbon dioxide. *Campylobacter sp*. are also thermophylic and so prefer temperatures of <30°C [Hurst & Crawford, 2002]. Survivability in environmental water is in the order of several days, but can be several weeks in cold water Not all species are pathogenic and *C. jejuni*, is the most important pathogenic strain, however *C. coli*, *C. laridis* and *C. fetus* have also been associated with disease but less frequently. The pathogenic strains can also are also infective in animals including birds and rodents [WHO, 2004]. The WHO considers thermo-tolerant coliforms to be an appropriate indicator for *Campylobacter* [WHO, 2004], however the UK Environment Agency does not [EA, 2002] but do consider them a reliable indicator of treatment effectiveness.

Campylobacter sp. Have sparodically been isolated from rainwater supplies [Bannister et al., 1997; Albrechtsen, 2002] and also been missed in large samples [Holländer et al., 1996; Simmons et al., 2001]. There is no indication in the positive samples of the level of contamination, however an infective dose is about 1000 organisms.

There are a few reports of gastroenteritis associated with *Campylobacter*, which is carried by birds, however the cases reported are either from immuno-compromised subjects (such as a chemotherapy patient) or have rainwater listed alongside several other possible causes (such as the consumption of poultry).

Table 10.5: Campylobacteriosis reported in the literature

Location	Bug	Notes	Source
Queensland	*Campylobacter jejuni*	*Campylobacter jejuni* not isolated from rainwater supply, however *E.coli* were detected. Strong associations with drinking rainwater and contamination path postulated to be from rodent or bird faeces washed into the tank.	[Merritt et al., 1999]
Queensland	*Campylobacter fetus*	*Campylobacter fetus* isolated from rainwater supply. Immunocompromised (cancer/chemotherapy) patient had repeated outbreaks. Boiling tank water cured problem.	[Brodribb et al., 1995]

PROTOZOA; GIARDIA AND CRYPTOSPORIDIUM

Protazoans are parasitic microorganisms that are an important cause of disease in humans and animals. The main cause of infection is through ingestion of their environmental stage called a cyst or an oocyst. These are hardy dormant stages that can survive for weeks to months in fresh water [WHO, 2004]. On ingestion the cyst "hatches" and infects the host. Cysts and oocysts are also extremely resistant to water treatment processes such as chlorination and UV sterilisation. They are, however very susceptible to desiccation [Robertson et al., 1992] and are therefore unlikely to survive on a roof for a long period. Being a dormant stage, cysts and oocysts are also non-motile and will settle out in a quiescent environment such as a water tank. Medema et al [1998] found a mean settling velocity for oocysts of 0.35 μm/s and for cysts of 1.4 μm/s (equivalent to 1m settling in about 8 days!). They did, however find that cysts and oocysts readily adsorb to suspended particles and should therefore settle more quickly. The two main types of protozoa of relevance to rainwater harvesting are *Giardia* and *Cryptosporidium* which are also found in amphibians, birds and mammals [WHO, 1996]. The WHO considers thermo-tolerant coliforms to be an ureliable indicator for cysts and oocysts [WHO, 2004]

Giardia was found by Crabtree et.al. [1996] (1–3.8 cysts/100 litres) but not found in other studies [Bannister et al., 1997; Simmons et al., 2001; Albrechtsen, 2002]. The infective dose is 10-15 cysts [EA, 2002]

Cryptosporidium was found in "traditional" cisterns (6-60 oocysts/l) but not cement tanks in Jordan. Traditional cisterns were mainly underground dugouts from rock and seepage from cesspits was thought to be the cause [Abo-Shehada et al., 2004]. Other studies have also isolated varying loads of oocysts from rainwater supplies Crabtree et.al. [1996] (1-70 oocysts/100l in

48% of samples) Albrechtsen [2002] (ND-50 cells/l from 6/17 of samples), Simmons [2001] Others studies [Bannister et al., 1997] have looked for *Cryptosporidium* but not detected it The infective dose is highly variable and ranges from 10 to 1000 organisms [EA, 2002]

One outbreak of *Giardia*/*Cryptosporidium* has been reported. This was the result of poor sanitation practice where the outfall from a leaking septic tank entered an underground rainwater tank.

Table 10.6: Giardia reported in the literature

Location	Bug	Notes	Source
Victoria	Giardia/ Cryptosporidium	*Giardia* and *Cryptosporidium* isolated from rainwater supply. Supplied by leaking septic tank uphill from water tank	[Lester, 1992]

LEGIONELLA

Legionella is a genus of bacteria of which there are about 40 species, about 18 of which cause disease in humans [EA, 2002]. *L. pneumophila* is the major species of health interest as it causes Legionnaires disease, however other species can cause less severe illnesses. . They are motile and aerobic [Sussman, 2002]. *Legionella* are opportunistic bacteria that live in a wide range of environmental water sources but require warm water to multiply (>25°C but <45°C) and are quickly killed above 55°C [WHO, 2004]. Stored rainwater has been shown to be within this range in hot countries [Gumbs & Dierberg, 1985; Ariyananda, 2005] so it is likely that many tanks in these countries contain *Legionella* species. Legionalla have also been shown to occupy Protazoans and can be resistant to many water treatment process by this route. The infective dose by ingestion for Legionella sp is very high (10^5-10^6 cells/ml) [Broadhead et al., 1988] but in an aerosols can be as little as a single cell [WHO, 1996] so water used in showers or flushing is a potential hazard though drinking is largely safe. The WHO does not consider thermo-tolerant coliforms to be an appropriate indicator for Legionella [WHO, 2004]

Several studies have isolated Legionella species in roofwater supplies Broadhead [1988] (mean – 266 cells/ml in 8/10 samples) Ruskin et.al. [1992] (both in US virgin island – a hot country), others have failed to find it [Holländer et al., 1996; Simmons et al., 2001; Albrechtsen, 2002] (all from temperate countries). Albrechtsen did find some non-*pneumophila Legionella*, however.

Two reported cases of Legionnaires associated with rainwater supply could be found in the literature (Table 10.7) – on one, the connection is fairly tenuous and the other, aerosols from a roofwater-supplied fountain was presumed to be the cause.

Table 10.7: Legionnaires reported in the literature

Location	Bug	Notes	Source
Sweden	*Legionella midadei*	Single subject diagnosed with *legionella micdadei* infection. Reported drinking water from a rain barrel one day before infection. Original barrel had been cleaned, Legionella was not isolated from barrels in the area.	[Back et al., 1983][1]
US Virgin Islands	*Legionella pneumophila*	Several incidences of Legionnaires' in visitors to a hotel. *L. pneumophila* and several other Legionella species isolated to rainwater system. Hyperchlorination resulted in no further outbreaks	[Schlech et al., 1985]

OTHER PATHOGENIC AND POTENTIALLY PATHOGENIC BACTERIA

While the four categories detailed above form the largest proportion of reported bacteria and reported illness, several other potentially pathogenic bacteria have been found in tanks but with (thus far) no reported ill effects. Considering the potential for underreporting of rainwater sources infection, it is likely that at least some of these have also result in illness. Other Pathogens have been indicated in epidemiological investigations but have not been isolated from supplies (often because they have not been looked for). These two categories of pathogen are detailed in Table 10.8

Table 10.8: Other pathogens associated with rainwater harvesting supplies

Pathogen	Motility	Respiration	FCs an acceptable indicator	Notes
Leptospira	Motile	Aerobic	No	• Shed in the urine of contaminated animals including rats. • RWH indicated as a high risk factor in epidemiological study in Hawaii [Sasaki et al., 1993]
Aeromonas	Motile	Aerobic	No – but *Aeromonas* can itself be used as an indicator	• Emerging pathogen implicated in gastroenteritis cases. • Present in most fresh and brackish water environments. • Found in several studies [Rinehart et al., 1985; Wirojanagud, 1991; Simmons et al., 2001; Albrechtsen, 2002]

[1] Original paper could not be obtained, however a good summary is given in [Sinclair et al., 2005]

Pathogen	Motility	Respiration	FCs an acceptable indicator	Notes
Mycobactreium	Non motile	Aerobic	No	• Emerging pathogen implicated in gastroenteritis cases. • Ubiquitous environmental bacteria found in soil and environmental waters. • Resistant to treatment and good viable in environmental water and water distribution systems • Found in RW tanks in two studies [Tuffley & Holbeche, 1980] [Albrechtsen, 2002]
Pseudomonas	Motile	Aerobic	No	• Causes a number of symptoms but rarely fatal. Mainly affects wounded areas or compromised patients. • Found by several studies: [Rinehart et al., 1985; Krishna, 1989; Holländer et al., 1996; Albrechtsen, 2002; Coombes et al., 2005]

10.3. Chemical and Physical Contaminants

As rainwater is the result of a natural distillation process, the chemical quality of rainwater is usually good and contains relatively little in the way of dissolved minerals and almost no suspended solids. Most contamination is added once the rain has hit the roof and washes off any accumulated matter. There are several barriers in the path of chemical contaminants and other barriers can be installed which will be discussed in Chapter 11. Once the rain enters the storage tank processes such as precipitation, sedimentation adsorption and biofilm activity act to change the chemical content of stored rainwater.

10.3.1 ACIDITY

As the rain has fallen through the atmosphere it is usually saturated with oxygen and carbon dioxide. The CO_2 tends to form mild carbonic acids (H_2CO_3) in rainwater so the natural pH is slightly acidic at around 5.6. As` a result of pollution and a few natural phenomenon such as volcanoes, modern rainwater usually also contains trace amounts of sulphates which form mild sulphuric acid (H_2SO_4) and raise its acidity a couple of pH points [Olem & Berthouex, 1989; Quek & Forster, 1993; Appan, 1999]. The pH changes little on impact with the roof and transport to the tank, however, residence in a tank, particularly one made of concrete or mortar significantly raises the pH of the as the soft, acidic rainwater reacts with the cement and absorbs calcium making the stored water more alkaline in the range of pH 6-10 [Olem & Berthouex, 1989; Fujioka et al., 1991; Thurman, 1995; Ariyananda, 2005; Handia, 2005].

10.3.2 SUSPENDED SEDIMENT

The largest component of roofwater pollution is in the form of particles washed from the roof. Such sediment is suspended in the water and is measured using turbidity which is a measure of how cloudy the water is. In rural areas of low-income countries, the suspended material is usually non-toxic but it can carry microorganisms and organic material with it and presents an aesthetic problem. High levels of turbidity can also protect microorganisms from light-based disinfection such as UV lamps or SODIS and can absorb chlorine. In urban areas the sediment can contain heavy metals and particulates from exhaust emissions which are of greater concern. These are discussed in Section 10.3.3.

Roof runoff can have very high turbidity, especially the initial runoff [Yaziz et al., 1989] At the tank outlet, however, rainwater harvesting systems tend to produce water with turbidity within the WHO guidelines [Bannister et al., 1997; Simmons et al., 2001; Handia, 2005; Ariyananda, 2005]. The main reason for this is that the waters of the tank form a simple stilling chamber and most suspended mineral matter settles out within 24 hours and will form a sludge at the bottom of the tank that may be periodically removed if it becomes problematic The sludge itself has also been linked to cleaning processes within the tank [Lücke, 1998; Spinks et al., 2005]. Organic material is less dense than water and so floats to the top [Deltau, 2001] and eventually saturates with water and settles [Spinks et al., 2003].

Filtering can be effective in removing the larger material which includes most organic matter such as pollen leaves and buds. It is, however less effective in removing very fine mineral. The incoming stream of suspended matter also reduces over the course of the storm and so can be significantly reduced by first flush devices. These are further discussed in Section 11.3.

10.3.3 HEAVY METALS AND URBAN POLLUTANTS

There have been a number of studies, primarily interested in roofs as a non-point pollution source that have measured the contaminants of rain and roof runoff [Thomas & Greene, 1993; Quek & Forster, 1993; Bucheli et al., 1998; Zobrist et al., 2000; Uba & Aghogho, 2000; Chang et al., 2004; Kim et al., 2005; Athanasiadis et al., 2005]. A number of these studies show that roof runoff can have levels of some trace metals, particularly, Lead and on one occasion near to a major highway, cadmium and chromium above those considered toxic by the WHO for drinking water, and sometimes above local authority’s regulations for pollution discharges. Most of these studies conclude that roofwater is a contributor to urban runoff pollution.

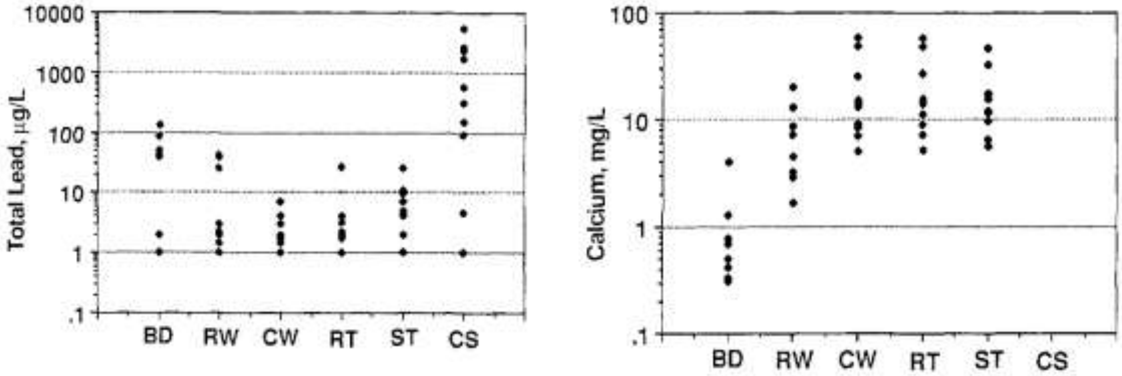
Unlike the other contaminants describes above where the roof was always a significant source of contamination, the roof can either be a source or a sink for trace metals. If leaded petrol is used and exhaust fumes exists in high concentrations in the atmosphere, scouring of the air can be a significant source of pollution and the roof may actually play a part in reducing the concentrations [Quek & Forster, 1993]. Conversely slightly acidic rain may corrode the roof and leach minerals from roof material itself or from flashing [Olem & Berthouex, 1989; Forster, 1999; He, 2002] leaching from metal roofs and flashing tends to raise zinc, copper and sometimes lead levels. Tiled roofs can also contribute copper and zinc via leaching of captured minerals [Quek & Forster, 1993; Chang et al., 2004]. Some tests have also been carried out on organic roofs and these can release chemicals that have been used to preserve the roofing material such as arsenic, however long-term exposure to the elements reduces the concentrations available. [Chang et al., 2004]. Almost all studies report initial concentrations are higher than those in subsequent runoff – even when the source of the contamination is the atmosphere itself, however when the roof material is a source of contamination, some concentration of metals remain throughout the storm [He, 2002].

Rather like the high levels of microbiological contamination in roof runoff, the chemical roof runoff data is potentially alarming as there is a significant fraction that has chemical contamination above the WHO guidelines for drinking water. Unfortunately, a number of these studies are being quoted as a source for chemical quality in “rainwater harvesting systems” – e.g. Lye [2002] quotes Forster [1999] as a source for those interested in chemical quality in his review of microbiological quality. Several other authors have reported roof runoff results as sufficient to show the non-potability of captured rainwater [Uba & Aghogho, 2000; Adeniyi & Olabanji, 2005]. However the roof runoff only supplies a part of the process. A number of other studies have also measured the levels of these contaminants at various points along the system [Sharpe & Young, 1982; Gumbs & Dierberg, 1985; Olem & Berthouex, 1989; Wirojanagud, 1991]. These authors have found that the chemical content of rainwater is significantly changed by being introduced into a tank. Figure 10.2 shows the results from one such study [Olem & Berthouex, 1989]. The contamination load (in this case lead) starts much higher than the WHO recommended level in rainfall delivered from the atmosphere, the roof acts as a slight sink, reducing the load in this case (a similar diagram for Zinc raises the concentration) and shows a significant reduction inside the tank where the level drops below the WHO guidelines. The level actually rises again when the soft-rainwater comes into contact with the household plumbing and leaches lead from pipes and solders.

It is the slightly acidic and “soft” (i.e. reactive) nature of rainwater that is thought to be the cause of the changes. When stored, the rain reacts with the walls of the tank leaching calcium (note the rising levels throughout the system shown in Figure 10.2b which reduces the acidity of the water causing dissolved metals to precipitate out and settle quickly to the bottom of the tank. Metals washed into the tank as suspended matter are also settled quickly due to their high density. The sludge at the bottom of the tank shows a highly elevated level of lead from all the settled material.

Figure 10.2: Concentrations of minerals at various points in the rainwater harvesting system from a system in Kentucky [Olem & Berthouex, 1989]

a. Lead (WHO limit: 10 μg/l) b. Calcium



Key: BD is the bulk deposition; RW if roof runoff; CW is water from the tank (taken at 50cm below the surface; RT is water from the tap, ST is water from the tap after standing in the plumbing for 8 hours; CS is tank sludge.

Another possible route to reduction in trace metals in stored roofwater is via biofilms action. Spinks et. al. [2003; 2005] noted similar reductions even in tanks with a plastic liner, which cannot leach calcium and show similar pH to the incoming roofwater [Coombes et al., 2005]. The proposed removal mechanism is that tank sludges and tank wall biofilms play a role in absorption of heavy metals, evidenced by elevated heavy metal content in the sludge and wall biofilms.

While it is often not practical to filter these minerals as they contain a substantial dissolved phase, it has been found that these pollutants decrease over the length of the storm so the introduction of first-flush mechanisms can substantially reduce the pollutant load delivered to the tank. Such mechanisms are discussed in Section 11.3.

10.3.4 ASBESTOS

Asbestos roofing is a common material in some countries and asbestos gutters can sometimes be found, so there are concerns about the risk of health problems from drinking water that has run-off via such materials.

There have been a number of epidemiological and animal-based studies of asbestos in drinking water distribution pipes in centralised water supply systems. The animal studies involved dosing with large amounts of asbestos and results were mainly negative, though some inconclusive results have been obtained. Epidemiological studies have found little to no evidence of a correlation between ingestion of asbestos and gastrointestinal cancers [WHO, 1996].

Advice from the World health organisation [WHO, 2004] and others is that while it is dangerous to *breathe in* asbestos dust, for example while cutting asbestos materials, there is no known danger from *drinking* water containing asbestos fibres. Asbestos is strongly associated with lung cancer, not with stomach cancer. Asbestos pipes continue to be used in many countries for drinking water distribution with no ill effects.

10.4. MOSQUITO BREEDING

As well as water quality considerations, roofwater harvesting systems may have wider public health implications, particularly with regard to their potential as breeding sites for mosquitoes which are an important vector for a number of diseases, notably malaria and dengue. The current expansion of dengue is a particular worry as the *Aedes aegypti* mosquito is a container breeder particularly suited to breeding in clear isolated water such as that found in a rainwater tank.

Mosquito breeding in roofwater harvesting systems has been associated with reported outbreaks of malaria and dengue in several locations [Hanna et al., 1998; Heukelbach et al., 2001]. However, when the state of systems is described, poorly designed and maintained storage, particularly unscreened [Hanna et al., 1998] and open-topped tanks [Heukelbach et al., 2001] are referred to.

Gutters are also quoted as an important breeding site, supplying up to half of surveyed *Aedes aegypti* (dengue carrying) pupae in a survey in Cairns [Montgomery & Ritchie, 2002]. The breeding sites were pools of water either caused by low gradients or built-up debris, indicating that well-installed gutters that reliably drain down and are regularly cleared of debris are necessary to avoid mosquito breeding. The “out of sight:out of mind” nature of many parts of a rainwater catchment system is seen as a particular problem. Many parts of the systems are above eye-level and so do not receive the attention they need – gutters are not cleaned and screens and covers are not checked regularly.

Considerable uncertainty exists as to how significant rainwater harvesting storage structures are to mosquito breeding. Even a well screened tank will often allow insects to enter; field experience shows that “tight fitting lids” tend not to be very tight fitting and it is not uncommon to find adult mosquitoes in rainwater tanks. Similarly mosquito eggs can be found, as they can be laid by the adult in the tank directly or in the gutters and then washed into the tank with the next rains. A fair proportion of these eggs may hatch out to become larvae. Mosquito larvae in a drinking water tank presents an aesthetic problem as finding “wrigglers” in one’s drinking water is off-putting, however the main issue from a public health viewpoint is whether adult mosquitoes emerge from tanks and increase the total population.

Mosquito larvae go through four stages (called ‘instars’) before they pupate and emerge as adults. The larvae eat bacteria and protozoans but as discussed in Section 10.2.1, these organisms are rare in well-designed rainwater tanks that don’t allow the entry of light. Laboratory studies have found that in the absence of nutrients, larvae are unlikely to develop beyond the third instar and therefore adult mosquitoes rarely develop under those conditions [Mittal et al., 2001].

Certainly, when left uncovered, rainwater harvesting storage structures form an excellent breeding ground for mosquitoes. In a survey of water stored in oil drums on Trinidad [Chadee & Rahaman, 2000], over 80% of drums showed evidence of *Aedes aegypti* mosquito breeding. Tun Lin et al [2000] found over 70% of larvae survived to adulthood in 2 rainwater tanks surveyed but makes no mention of tank type or conditions. A large multi-country pupal survey [Focks & Alexander , 2006] found that the category of “ground tanks” and “barrels” accounted for a disproportionately high fraction of pupae counted, particularly in the dry season, whereas “cisterns” which were only found in Mexico showed a near zero pupa count. The difficulty in interpreting this data lies in the categorisation of containers which lumps good practice with bad and so provides no guidance on the effectiveness of interventions to prevent mosquito breeding such as screens and covers.

The methodology used in several of these tests is pupal surveys where pupa are counted rather than adults or larvae. Pupal surveys have been shown to correlate well with epidemiological data [Focks & Chadee, 1997] and are simple to carry out. They should form a useful methodology for testing breeding in rainwater harvesting systems of varying typed and with varying interventions.

To investigate the changes in mosquito survivability, pupal surveys were carried out on 10 tanks in southern Uganda. It was found that larval survival depended greatly on tank conditions. At one extreme there are open-topped tanks with a high load of washed-in organic material where tiny worms and mosquito larvae and pupae abound. At the other extreme, tanks which are covered so that no light enters and are well screened against the entry of leaves and twigs contain a few early-instar larvae but do *not* contain pupae. This study is preliminary but does indicate that a well-designed and maintained tank may not encourage the spread of mosquitoes. The sample is, however very small and a larger study with appropriate system categorisation is required for firm conclusions to be drawn.

10.5. CONCLUSIONS

Roofwater harvesting systems have a complex and largely hostile route for entry of any foreign material. The main path for contamination to enter the system is via water washed from the roof which is an inherently cleaner place than ground-based catchment and is, therefore the focus of this research.

In terms of biological contamination, human-sourced pathogens are very unlikely to find their way into a water tank due to the complexity of the necessary path and no human-specific pathogens have been reported in the literature. Some pathogens have been reported but these have all been attributed to animal sources, environmental sources or accidents. As a result of the difference in potential pathogen source, it is unlikely there will be a good correlation between pathogens and traditional indicator bacteria in rainwater harvesting systems as indicators have been developed to look specifically for risk of *human* faecal contamination. The lack of correlation has been noted in several studies looking at pathogens in rainwater tanks.

Bacterial indicators are, however, useful in showing processes and changes in pathogen numbers (particularly for bacteria such as *Salmonella* and *Campylobacter*. Studies using indicators have shown that there are few bacteria in rain, much more in roof runoff and greatly reduced numbers in tank water. The processes reducing the number of bacteria in tank water are further discussed in Chapter 11.

Chemical quality of rainwater is variable – based mainly on location, and roof runoff can have high levels of contaminants, derived either from scavenged atmospheric material, accumulated material on the roof or leached out of the roof material itself. Physical and biological processes in the tank reduce these levels over time.

Rainwater harvesting systems provide several opportunities for mosquito breeding at various points along the system. Gutters are one of the prime candidates but larvae are often found in the tank itself. Early indications are that these larvae do not mature into adults. A large multi-country however reports large pupae numbers in rainwater tanks. No system information is given in the study and “tank” is poorly defined so it is impossible to tell whether the tanks were following good practice or not . A preliminary study has shown that system design is likely to affect adult mosquito emergence, however a much larger and better resourced study is necessary to confirm and codify this result.

11. INTERVENTIONS TO IMPROVE WATER QUALITY

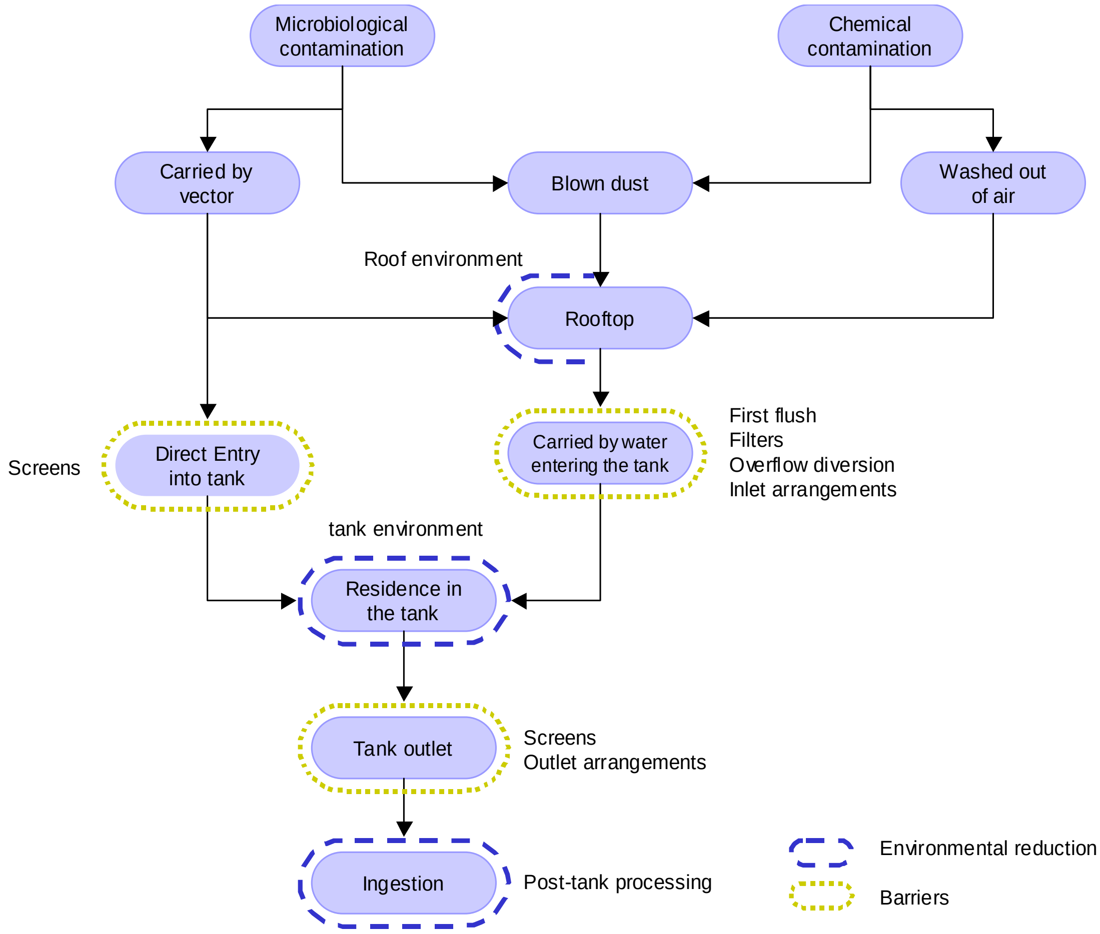
Despite the roof being higher than the ground, dust and other debris can be blown onto it, especially if the roof is near to a roadway. Leaves can also fall onto the roof from nearby trees and flying and climbing animals can defecate upon it.

The lack of correlation between indicator bacteria and pathogen content of rainwater (e.g. [Krishna, 1989; Fujioka et al., 1991; Crabtree et al., 1996; Bannister et al., 1997]) is disturbing as no simple and reliable method exists to routinely test rainwater systems for bacterial quality. Fortunately, there is a move away from routine surveillance of small household water supplies toward risk assessment and management and sanitary inspections [WHO, 1997; WHO, 2004]. Poor practice will certainly reduce the quality considerably and several studies have shown correlations between poor practice and increased bacterial activity [Rinehart et al., 1985; Dillaha & Zolan, 1985; Abo-Shehada et al., 2004]. Conversely there are a number of simple techniques that can be used to *improve* the quality of water in the system. Knowledge of the risk pathways points toward a strategic placement of interventions to reduce the risk of a pathogen or toxin entering the tank.

11.1. THE PATH OF CONTAMINATION

To decide on the best strategy to reduce contamination in roofwater systems, it is useful to observe the path a contaminant must follow in order to enter a potential host. The risk pathways were discussed in Chapter 10.1. Figure 11.1 shows these pathways, highlighting natural processes that may be enhanced to reduce risk and likely placements of interventions.

Figure 11.1: Risk reduction processes for roofwater harvesting



Of the available paths, the direct entry of contamination is seemingly the simplest route to block, indeed many of the usual pieces of advice (tight fitting lid, screens on all outlets [WHO, 1997]) are directed toward this. However it can prove very difficult in the field to totally eradicate all possible disease vectors, since the smallest of creatures such as insects will find *any* hole in the tank's defences. Larger animals, particularly mammals and birds who represent the highest disease risk to humans, can, however be excluded by ensuring all inlets and outlets are screened.

The remaining paths rely on the contaminants being washed in from the roof. It is reassuring to note that an impermeable roof (particularly one made from steel) is an extremely hostile environment for human pathogens, which have evolved to live in a warm, wet, environment. The dry heat typical of a metal roof under bright sunlight will effectively kill many of these pathogens [Yaziz et al., 1989; Vasudevan et al., 2001b], exposure to UV radiation will inactivate others and many pathogens including hardy environmental phases such as oocysts are extremely susceptible to desiccation simply from drying-out in the air [Robertson et al., 1992]. Nevertheless, It has been demonstrated that bacteria *are* washed from roofs [Yaziz et al., 1989;

Thomas & Greene, 1993; Coombes et al., 2005], originating from sheltered locations such as under leaves, from recent deposition or from deposition during the rainstorm itself [Evans et al., 2006].

11.2. INLET SCREENS AND FILTERS

The quality of water can be much improved if roof debris is kept out of the system. To accomplish this, filters can be added to a rainwater harvesting system at the inlet, outlet or both. As the water must pass from the roof to the tank inlet, the conveyance is a prime candidate for placing a filter to block any contamination from entering the tank. Investigation of the likely risk pathway shows that bacterial contaminants will have arrived on the roof attached to something solid such as wind-blown dust or animal faeces, so removing solid material should also reduce biological activity in the tank. Furthermore, removing debris also reduces the level of nutrient reaching the tank: high nutrient levels have been associated with higher bacterial levels [Rinehart et al., 1985; Krampitz & Holländer, 1999] and mosquito larvae development [Mittal et al., 2001].

11.2.1 FILTER CHARACTERISTICS

The primary measures of any inlet filter are its *hydraulic efficiency*, which is a measure of the fraction of the incoming stream that penetrates the filter (and how much is spilled), and its *particle removal efficiency* which is a measure of the fraction of the incoming particulates that are removed by the filter.

A key constraint to any filter in a rainwater harvesting system is that it should be capable of dealing with the high flows associated with high rainfall intensities. A 2mm/min peak intensity translates into a 1.7 l/s flow on a $50m^2$ roof or a flow velocity of 85 m/hr through a Ø30cm filter. A typical flow velocity for a "rapid" water treatment filter is 8-40 m/h [Droste, 1997]. As discussed in Section 2.3.1, the gutter forms a flow-limiting device, so a useful design guide is to size the filter to the same flow as the maximum gutter capacity.

Water spilling from a filter becomes unacceptable if it is allowed to become too large a fraction of the incoming stream, however a small amount of spillage can be tolerated provided it is used to keep the filter clean. All water filters will spill some water. The amount spilled depends on several factors

- The fineness of the filter material which directly affects its permeability – the finer the material, the less easily water will pass through it

- Any slope on the filter – the greater the slope, the faster water will be shed

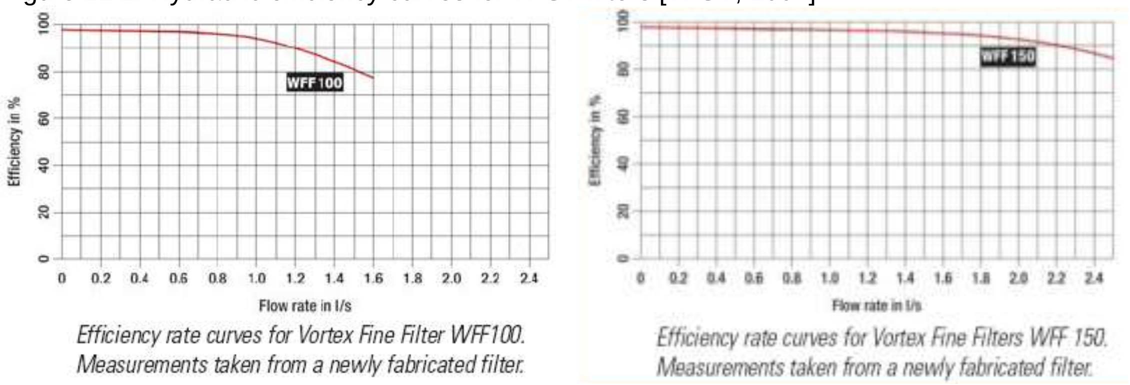
- The area of the filter – the greater the area, the more opportunity for the inlet water to pass through it

- The rainfall intensity – the heavier the rainfall and hence the greater the flow from the roof, the greater the fraction of water that will be spilled

- The existing dirt load on the filter – filters that are clogged pass water less efficiently than clean filters

Of these factors fineness of filter material, slope and area are in the control of the designer, so (finances and space willing) a filter can be designed with any desired hydraulic efficiency. Figure 11.2 shows the hydraulic efficiency curves for two commercial filters as flow rate changes. Both filters have similar designs but the WFF150 is larger than the WH100 with a commensurate improvement in hydraulic efficiency at high flow rates. The hydraulic efficiency at low flow rates is about 97% and is limited by the mesh size used (0.38mm).

Figure 11.2: Hydraulic efficiency curves for WISY filters [WISY, 2007]



It is also worthwhile noting that these efficiencies are for a new filter. As filters are used, they become blocked and hydraulic efficiency reduces. Rott and Mayer [2001] found that hydraulic efficiency dropped considerably with a median reduction of 10% and Standard deviation of 12%, after repeated testing with water mixed with a set particulate load (Table 11.1).

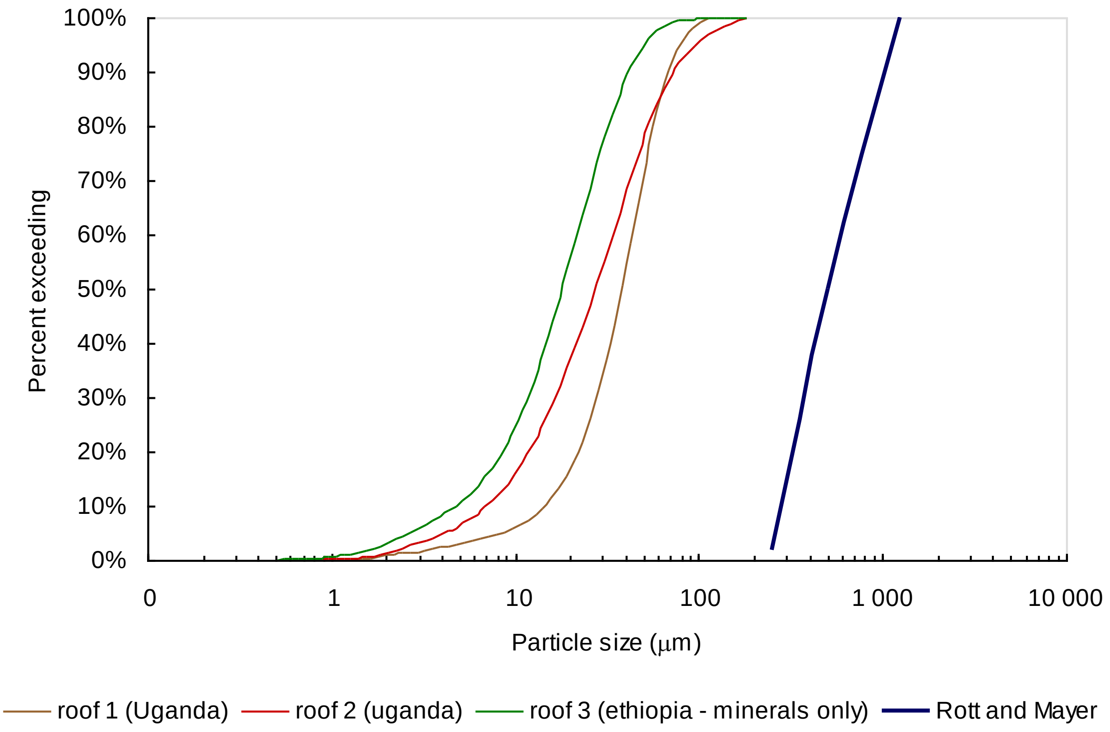
The removal efficiency of the filter is its ability to remove particulates from the incoming stream. Larger particles on roofs tend to be dropped from vegetation so are composed of leaves, sticks and pollen. When roofs are not overhung, only wind-blown particles can accumulate and this favours small particles, typically silt sized. Most filters will remove larger particles such as sticks and leaves; it is the sediment load that poses the challenge to filter designers. The effectiveness can simply be expressed as the fraction of *total* particulates removed by the filter (e.g. net reduction in turbidity) or more usefully as the fraction of a particular particle size that are removed.

Rott and Mayer examined a number of filter designs available in Germany and reported the results according to hydraulic efficiency and material removal efficiency [Rott & Meyer, 2001]. The contaminants were modelled on roof debris but repeatable analogues were found for the debris components. The challenge applied to the filters is in Table 11.1.

Table 11.1: Contaminant load used by Rott and Mayer

Contaminant	Concentration (per m^3 of water)
LDPE sheet 15µm 50mm x 50mm	10
Polypropylene balls D = 3.5 mm	100 g
Quartz sand with size range between 0.25-0.5mm	100 g
Quartz sand with size range between 0.71-1.25mm	100 g

Figure 11.3: Particle size distributions (PSDs) of mineral particles found on three roofs and particle sizes used by Rott and Mayer

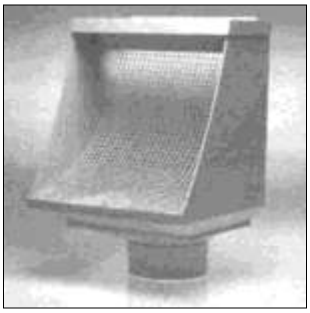


Rott and Mayer's contaminant load provided good results for the commercial filters under test with all filters removing all of the sheets and balls and mean capture rates for sand of 91% for the coarse material and 53% for the fine material. They do not, however map well onto the particle size distribution (PSD) typically found on roofs as shown in **Error! Reference source not found.**. It is extremely likely that the performance of these filters will be significantly poorer when these silt sized particles are present.

Generally, a high hydraulic efficiency and a high particle removal efficiency are the mark of a good filter. These two criteria are, however, often in competition and limited by practicalities; A fine filter material will have a good particle removal efficiency, removing large fractions of fine particulates but it will have a poor hydraulic efficiency. Increasing the surface area can reduce this conflict this to a certain extent, but will ultimately lead to filters becoming unwieldy and requiring large openings in the tank. Large filters can be designed using complex arrangements but at a high cost.

11.2.2 COARSE LEAF FILTERS

Gutter filter
(Picture: L.B. Plastics, Inc.)



Downpipe top filtering
(Picture: Leafbeater systems Pty. Ltd.)

In-downpipe filter
(Picture: 3P Teknik GmbH)

Coarse mesh leaf filter at tank entrance in Sri Lanka

The first line of defence is a coarse leaf filter. The filter can be installed anywhere from the gutter to the entrance to the tank. It need not be fine and so no problems should be encountered with flow rate through the filter and the filter itself can be removable for cleaning. The most popular positions are in the gutter, at the beginning of the downpipe, in the downpipe, in the ground before the tank and at the entrance to the tank itself. Of these, the tank entrance is by far the most common in very low cost systems. The pros and cons of each installation are outlined in Table 11.2.

Table 11.2: Pros and cons of various filter positions

Type	Pros	Cons
In-Gutter	• Prevents leaf build-up in gutter thus; • removes fire hazard • reduces mosquito breeding • avoids cleaning chore	• Can be expensive due to large areas to be covered • Poor installation can; • increase leaf build-up due to leaves catching on filter • make cleaning what isn't filtered more difficult
At downpipe	• Central location minimises filter area • Can be combined with a drop to increase efficiency • Can replace downpipe connection as gutter box • Can be self cleaning (to an extent)	• Difficult to clean due to height • If simply placed into gutter-level downpipe connection can block entire gutter
In Downpipe	• Increase in filter area due to length of downpipe available • Low space use • Wetting requirement means first flush is beneficially dumped	• Uses more than 10% of water for self cleaning action • Requires more complex design • Poor design can lead to excessive water loss • Difficult to access for cleaning • Blockages not obvious
In-line (underground)	• Removes mounting problems • Easily accessed for cleaning	• Only useful for underground tanks • Poor design can lead to ingress of stormwater into the tank
At tank entrance	• Simple and inexpensive installation • Can be as simple as a cloth over the tank inlet • Very visible	• Entrance to tank is available to accidental (or deliberate) contamination • Reduces possibility of any further filtration

11.2.3 FINE FILTERS

Combination cloth/sand/gravel filter in Sri Lanka

Filter media from a large gravel filter in Sri Lanka

Two-stage Self cleaning filter in Germany
(Picture: 3P Teknik)

Finer filtering can remove small sediment which would otherwise either be suspended in the water or settle to the bottom of the tank leaving a sludge. The techniques are well known, employing gravel, sand or fine screens but the needs of rainwater harvesting systems are unique, as in a tropical downpour flow rates can be very high – with short-term peaks of more than 1.5 l/s. This calls for either very large surface areas or coarser screens. Fine filters are often specified to use particulate media such as gravel and sand. A typical example is a filter consisting of a Ø300mm tube filled with 150mm sand on a bed of 200mm of pebbles that has been used in Sri Lanka [Ranatunga, 1999] which copes with reasonable flows but overtops at high flows.

A problem of fine filters is cleaning. As all water passes through most designs of fine filter, particles become trapped in the filter requiring periodic cleaning. If this is not carried out, the filter will eventually block and simply overflow which has resulted in filters being refilled with coarser media or simply emptied of media and bypassed [Ranatunga, 1999]. Problems with upkeep have also been noted with German filters [Deltau, 2001]. These were initially gravel filters but have evolved into largely self-cleaning, stainless steel meshes which use the first flow of water from a storm to flush the filter of debris [3P Technik, 2001] or have a continual washing action discarding about 10% of the water [WISY, 2007]. The current trend is for these meshes to be coarser [Hyatt, 2007 personal correspondence[1]], presumably driven by their continued tendency to clog.

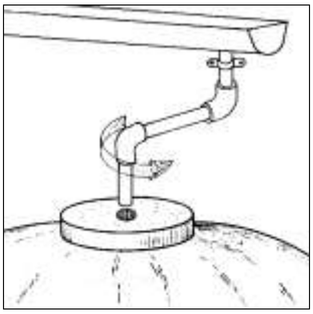
Martinson and Thomas [2005] showed that a low-cost filter can be made using similar principles to the German self-cleaning designs. A cloth is stretched over a cone-shaped frame (Figure 11.4) which allows a proportion of the water to run-off, cleaning the filter.

Figure 11.4: Low-cost, self-cleaning inlet filter

[1] Glyn Hyatt is the UK distributor for 3P a German rainwater accessories manufacturer whose range includes a series of filters some of which now include backwashing in a attempt to solve clogging.

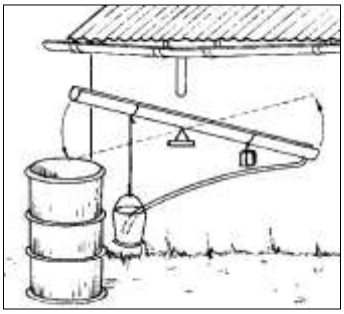
Experiments showed with a muslin cloth (1.0 mm weave – though with significant furring of threads), the filter removed 90% of the fine sand defined by Rott and Mayer and spilled about 10% or the inlet water at a flow rate of 1 l/s, a hydraulic performance roughly comparable to the smaller WISY filter. Colwell et al [2003] showed that pouring water through sari cloth can reduce bacterial load and that the effect is particularly high in old cotton sari cloth as a result of the threads having frayed, reducing the aperture size. Coarse cloth is therefore an ideal candidate for water filter use. In VLC systems there is usually a significant overflow of water and self-cleaning cloth filters of these types may be viable if suitable cloth is available locally.

11.3. FIRST FLUSH



Manual first flush device (Picture: Lee and Visscher [1992])

Simple downpipe first flush system in Sri Lanka (Picture:T. Ariyananda)



Seesaw diverter (Picture: Lee and Visscher [1992])

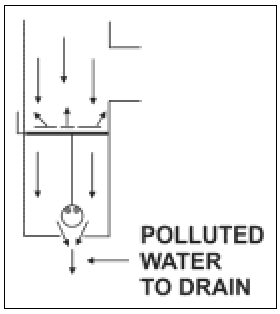


Diagram of "flow-rate" diversion system (Picture: Church [2001])

During a storm, the rain progressively washes the roof resulting in progressively cleaner water running off. Contaminants are therefore usually concentrated in the first run off and after this has passed and washed the roof the water is considerably safer. Simply removing the dirtiest part of the runoff is consequently a useful method of reducing contaminants washing into the tank. First flush systems have a number of advantages over filtration:

- They are not sensitive to particle size which is particularly important when the PSD of roof dust is considered

- They will remove dissolved contaminants as well as suspended ones which is important if trace minerals such as lead and zinc are problematic

They do however require the "throwing away" of a portion of water, though it is possible to design systems where this water may be captured and used for purposes where potable quality is not necessary. First flush systems are a popular method of improving the quality of roof runoff prior to storage, particularly in Asian countries and good results have been shown for the effectiveness of first flush devices on water quality in rainwater tanks [Ntale & Moses, 2003; Abbott et al., 2006].

There remains uncertainty as to the actual amount to be removed, with recommendations varying from fixed volumes of 20-25 litres [enHealth, 2004] [WHO, 2004]to fixed times for manual devices [Lee & Visscher, 1992] [WHO, 1997]to fixed rainfall depths of 0.4 – 2mm [Coombes et al., 2000; TWDB, 2005]. There are, in fact two parameters that define the performance of a first-flush system; the amount to be flushed and the length of time to reset the system. Chapters 12 and 13 are devoted to quantifying these parameters and investigating the effect of discarding the first flush on total material removal and water delivery of rainwater harvesting systems.

There are four methods of separating the first flush; manual, fixed volume, fixed mass and flow rate. The manual method is the simplest and widely recommended [Lee & Visscher, 1992], [Gould & Nissen-Petersen, 1999], it does, however rely on the user both being home and prepared to go out into the rain to operate the device, much reducing its usefulness.

The fixed-volume method, which relies on the water simply filling a chamber of a set size (often a length of downpipe) until it overflows is the "automatic" method usually recommended [enHealth, 2004; TWDB, 2005]. The method can be used either with or without a floating ball seal which helps in reducing mixing between early dirty water stored in the first-flush device and later clean water. However Michaelides [1987] has found that this mixing is transient. Michaelides also showed that placement of the pipe is critical to the efficiency of the mechanism. If the pipe is placed in a horizontally flowing section of downpipe mixing is greatly reduced. Fixed volume, first-flush systems are found with either automatic draining over a period of time or require manual draining. Manual draining systems have little to recommend them as they rely on user intervention which is unreliable. If they are left undrained, they will not only fail to work for the next storm, but can cause additional pollutants to be entrained into the incoming water flow from the first-flush device itself.

The fixed mass system which uses the mass of a fixed volume of water to activate the system has also been promoted, mainly in Africa but has met with little success. The devices, usually relying on a mass of water to tip a bucket or seesaw tend to be unreliable and users inevitably disable the system [Gould & Nissen-Petersen, 1999].

A newer first flush concept is to use the changes in flowrate over the course of a storm. Stormwater management designers have been using a flowrate model of first-flush for some time to reduce the large land areas required for “volumetric” facilities [Adams, 1998], however recently several companies in Australia [Church, 2001] and the USA [FloTrue, 2006] have marketed systems whereby flowrate is used for roof runoff. These systems balance the rate of water intake into a suspended hollow ball against its leakage. During rainfall, inflow will exceed leakage so that water accumulates in the ball, raising its weight and stretching its suspension until it descends into a recess, blocking the opening and allowing water into the tank. The system has the advantage of being largely self-cleaning and removes the need for any storage of the first flush water (and its subsequent drainage).

11.4. RESIDENCE IN THE TANK: IN-TANK PROCESSES

A frequently overlooked feature of rainwater storage is the effect of storage itself. As the water is stored in a quiescent condition, several processes can take place raising the water quality such as sedimentation, floatation and bacterial die-off.

Sedimentation and floatation are the result of differences in density of washed in matter to that of water in the tank. Simply put, sediment tends to be heavier than the water and will settle on the bottom given enough time and can be cleaned out from time-to-time. Vegetable matter is generally lighter and will float to the top and is can be washed out with overflow water. Eventually, this matter saturates with water and sinks to the bottom and can be dealt with in the same way as bottom sludge.

The tank is a very low-nutrient environment and not conducive to organisms evolved to live in the gut of an animal. Adverse environmental factors outstrip supportive factors due to removal of organisms from their natural environment. Sometimes there may be a short-term increase in numbers as the microorganisms take up residence. The main factors for the decline are [Droste, 1997]:

- Flocculation and sedimentation remove some bacteria
- Plant matter dies off from lack of sunlight
- Competition for food increases
- Predation increases reducing the prey micro-organisms and ultimately starving out the predators

As discussed in Chapter 10, die-off of is indicated in comparisons of indicator bacteria in rainwater tanks with those in roof runoff. As a result of near daily indicator sampling under the DFID project, Martinson and Thomas [2003] were able to show that bacterial die-off in tanks in the field conformed to exponential decay with a log reduction[1] in about 3 days. This die-off rate is similar to that found in laboratory experiments with *Salmonella enteritides*, *Yersinia enterocolitica* and *Campylobacter jejuni* carried out in Germany [Krampitz & Holländer, 1999] which showed total die-off of these bacteria in less than 8 days and that it was accelerated by high temperatures and presence of biofilms and delayed by presence of nutrients. Experiments in Australia [Spinks et al., 2003; Spinks et al., 2005] have also shown the effects of biofilms and tank sludge in reducing bacterial and other contaminants over time.

Typical die-off behavior for out-of-place microorganisms in water follows the pattern of a short period where numbers remain constant followed by exponential decline. The time needed for a defined level of die off can be calculated using the equation [Droste, 1997].

$$t = \frac{\ln\left(\frac{C_0}{C}\right)}{k_d} \qquad \text{Equation 11.1}$$

Where: t is the elapsed time; C_0 is the initial bacterial concentration; C is the concentration at time t; and k_d is a constant which depends on local factors such as UV levels, temperature and pH

The residence time in the tank can also be used to introduce more proactive measures to improve water quality. Chlorination is widely recommended as a final sterilisation for rainwater [UNEP, 1998] and methods of introduction as simple as an suspending earthenware pot suspended in the tank have been employed [Pieck, 1985]. However chlorination is not generally well liked by users [Fujioka, 1993] and the chemicals used can be dangerous if misused.

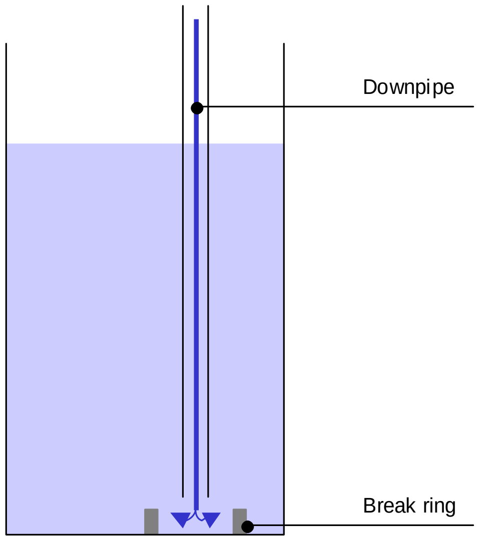
[1] A cultural aside: Engineers are comfortable with the concept of a "time constant" where an expression is given in the form of $e^{t/k}$ where t is time and k is the time constant. Environmental health has many of these exponential functions mainly of exponential decay; however these are usually expressed as a time to achieve a certain level of process such as a "log reduction", a tenfold decrease – sometimes called a T_{90}, or a "two log reduction", a 100 fold decrease or T_{99} and so on. Conversations with field workers and users indicate that they are happy talking about a "time to get the water clean enough" but often completely baffled by the concept of a time constant or decay constant. As a result of this, most such expressions will be expressed in such "amount to reach a certain level" terms unless there is an analytical reason for the more formal engineering usage. In these cases a "translation" will also be provided.

11.4.1 INLET AND OUTLET ARRANGEMENTS

In order to take advantage of this effect, mixing must be kept to a minimum by keeping the tank water calm. Water from rainfall is usually cooler than the ambient water in the tank [Kincaid & Longley, 1989] so water should be introduced from the bottom of the tank while water removal should be from the top, the reverse of the normal practice. Martinson and Lucey [2004] carried out a study of various inlet arrangements and found that radial manifolds were effective in lowering water velocity and downward pointing pipes produced a smaller zone of mixing. German best practice [Herrmann & Schmida, 1999; Deltau, 2001] is to arrange the inlet so that it goes all the way to the bottom of the tank as shown in Figure 11.5. A ring of material surrounding the inlet will break the downward flow and prevent it from resuspending settled material. With this arrangement, the incoming water will remain in a zone on the bottom of the tank and will not disturb the aged water above it.

Figure 11.5: Ideal inlet arrangement

a. Diagrammatical representation



b. Commercial example from Germany

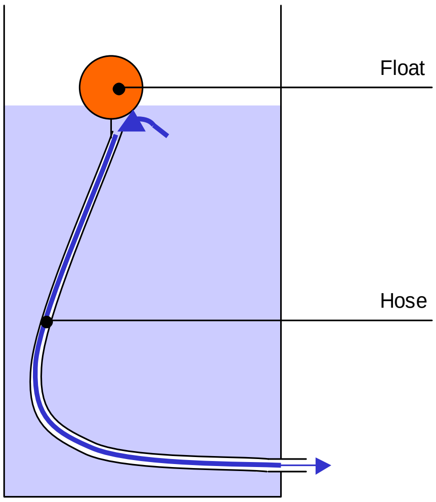
[Picture WILO GmbH]

c. Mortar ring used in Sri Lanka

The outlet to the tank is similarly important. As the newest and dirtiest water is at the bottom of the tank, it would seem advisable to take the water from near the top. To do this, German practice [Deltau, 2001] places the outlet on a flexible hose with a float at the top as shown in Figure 11.6. The float can be anything that floats; successful examples have been made from discarded mineral water bottles. To prevent entry of floating matter, the entrance to the hose should be about 2” below the surface of the water.

Figure 11.6: Ideal Outlet arrangement

a. Diagrammatic representation



b. Commercial example from Germany

[Picture WILO GmbH]

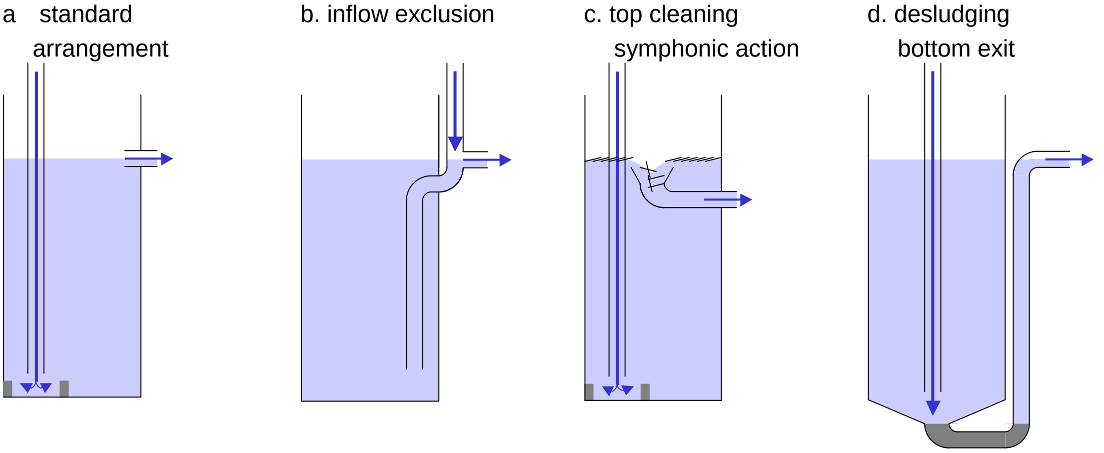
c. Floating off-take in Uganda

The very top of the tank has, however, been shown to have raised levels of indicator bacteria [Coombes et al., 2005; Abbott et al., 2006] thought to be attracted to the aerobic zone. E.Coli are aerobes, however many human pathogens found in rainwater tanks tend to be anaerobes (e.g. *Salmonella*), micro-aerobes (e.g. *Campylobacter*) or non-motile (e.g., *Shigella* and various cysts and oocysts) so this finding may not be especially significant from a health perspective. Nevertheless some environmental pathogens such as *Leptospira* and *Legionella* are motile aerobes and considering the variety of organisms that may transfer from the environment to humans via rainwater tanks, it is probably wise to ensure the outlet is well below the surface. The reported findings are limited, based on testing at only 3 or 4 heights within the water column, so at present it is unknown exactly how levels of bacteria change across the height of the water column in a tank and how these levels change with differing inlet, outlet and overflow arrangements.

11.4.2 OVERFLOW ARRANGEMENTS

The overflow from the tank can improve or protect water quality. The standard overflow shown in Figure 11.7a simply removes water from the top of the tank. If the bottom-in - top-out arrangement is used, it will be the oldest and cleanest water, which will be discarded replacing it with dirty water from the roof. A better arrangement is shown in Figure 11.7b where the overflow water bocks any incoming water, preventing it from mixing with the water stored in the tank. This is probably the best arrangement for tanks of less than $2m^3$.

Figure 11.7: Overflow arrangements



For larger tanks with well-designed inlets and outlets, the inlet water will take some time to reach the outlet so dirty inlet water will have a lower impact and the overflow can therefore be used to perform cleaning tasks. In an area where most material entering the tank floats to the top, the arrangement shown in Figure 11.7d may be used. In this configuration the overflow acts to accelerate the water into the overflow pipe by symphonic action causing the surface of the water and any floating matter to be drawn into the overflow, cleaning the top of the tank. German practice is to include these symphonic overflow arrangements [Deltau, 2001]. Figure 11.7c shows an arrangement where the overflow water is taken from the bottom of the tank. This means that the overflow water will come from the mixing zone containing the dirtiest water and will also carry any resuspended settled matter with it. A tank with an overflow of this design avoids desludging but may need to have any floating matter skimmed from the water surface periodically. A fairly complex bottom-overflow manifold arrangement is currently being marketed in New Zealand [Connovation, 2006] but evidence of its overall effects is currently inconclusive [Abbott et al., 2006]. An undergraduate project attempting to design a simple single-pipe arrangement has been undertaken [Rose, 2006] but preliminary results have been disappointing.

11.5. Conclusions

Water quality can be enhanced by good practice. Appropriate inlet and outlet and overflow arrangements should enhance in-tank removal processes and reduce the need for maintenance. There is, however some uncertainty as to the ideal location for the outlet, partially due to low resolution results and partly due to the use of indicator bacteria that do not necessarily follow the behaviour of pathogens when in the quiescent, nutritionally barren water of a rainwater tank.

Inlet treatment can greatly reduce the contamination in a tank. Leaf filters are useful and can be placed in various locations, fine filters can be used but suffer from trade-offs of hydraulic efficiency and particle removal which may make effective filters expensive. They are also untried against a realistic particle size challenge in a laboratory. First-flush may be more promising as it is insensitive to particle size and also effective against dissolved matter, however the trade-offs of hydraulic efficiency and material removal have been hitherto unknown. Chapters 12 and 13 are devoted to answering these unknown factors.

For use in a low-cost system in a low-income country, there are several criteria that should be met for good inlet treatment design:

- As user intervention has been shown to be problematic, the device should be easy to clean or largely self-cleaning
- It should not block easily (if at all) and blockages should be obvious and easy to rectify
- It should not provide an entrance for additional contamination
- The cost should not be out of proportion with the rest of the system – user surveys as part of the DFID project indicate that people in southern Uganda will only spend about 5% of the cost of the system on inlet treatment, users in Sri Lanka will spend closer to 10%.

12. QUANTIFYING THE FIRST FLUSH PHENOMENON

As discussed in Chapter 11, first-flush diversion (whether to waste or to a separate buffer store) is increasingly recognised as a useful intervention to reduce both suspended and dissolved contaminate loads in rainwater systems. It is becoming more widely practiced as its benefits are better publicised. Such first-flush systems rely on the initial rainfall in a storm to wash the roof before water is allowed into the main store. While there is almost universal acceptance that this is beneficial, there is no agreement on just how much water should be diverted, or whether such diversion should be based on volume, depth or rainfall intensity.

The following two chapters discuss the first flush phenomenon with particular reference to roof-runoff. This chapter discusses the theoretical basis for first flush and governing equations derived from urban drainage research. Parameters are also derived for suspended solid and dissolved solid wash-off of roof catchments. Chapter 14 discusses the effects of first flush upon the water yield from rainwater harvesting systems.

12.1. THE THEORY OF FIRST-FLUSH

The underlying principle behind first flush diverters is that the rain washes off dust and other debris from the roof as a storm progresses, so that the runoff stream becomes progressively cleaner. Therefore, if one diverts the first part of a storm away from the water storage, the remainder will be much cleaner and the need for subsequent treatment may be reduced or even eliminated. Unlike filtering, this is true for both suspended and dissolved material. While this is generally agreed to be "a good thing", little is know *quantitatively* about first flush, particularly in regard to roofwater harvesting.

Figure 12.1a shows the typical behaviour with the contaminant level starting at L_0 and falling with accumulated rainfall until the end of the storm. As the contaminants will accumulate in the tank, integrating the levels over the overall rainfall gives the overall quantity of contaminant delivered to the water store[1].

Figure 12.1: Accumulated contamination in roof runoff

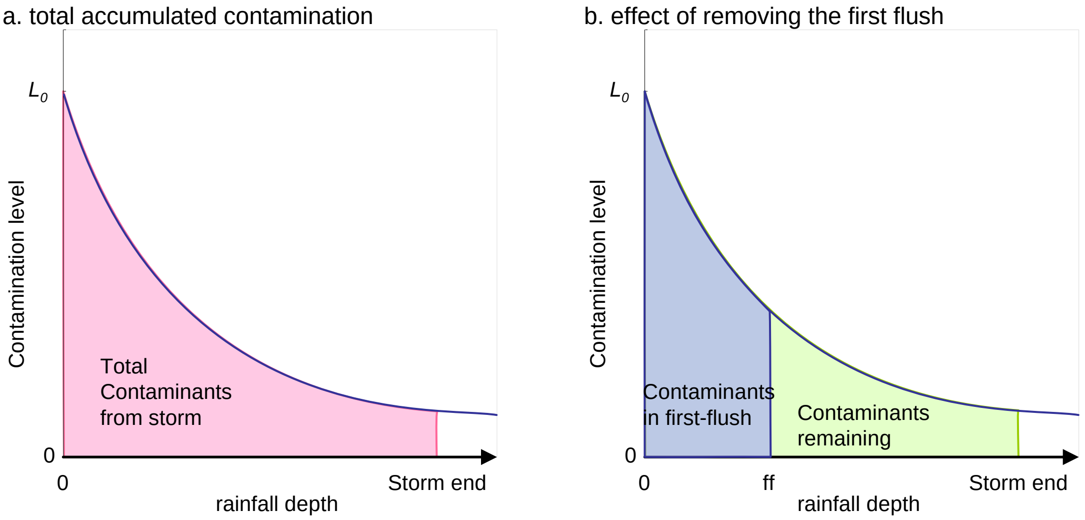


Figure 12.1b shows the effect of removing the initial runoff. The area under the curve representing the contaminants in the first-flush depth contains the majority of the total contamination delivered by the storm, however the first flush depth is less than half of the total rainfall. The effectiveness of any first-flush depends on how fast the contamination washes off and a balance struck between removing contaminants and delivering water to the store.

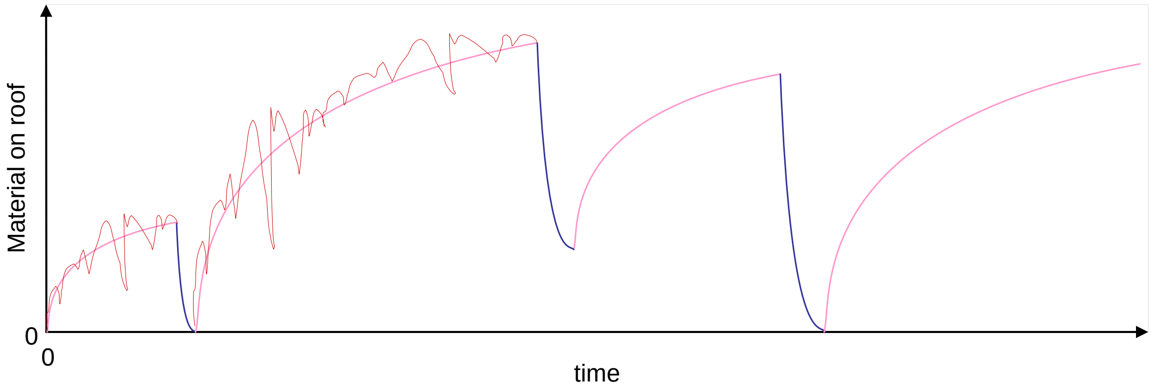
There are, in fact two questions that need to be answered for effective first flushing. They are:

- How large should a device be in order to discard enough of the rainfall to achieve an appropriate water quality?
- How quickly should the device reset?

[1] In fact for the *actual* accumulated contamination load to be derived from measurements of its concentration, it must be integrated over the total rainfall *volume*. The volume of rainfall is obtained by multiplying the rainfall depth by the roof area, however as the roof area remains constant it is sufficient to use the depth alone for comparing runoff contamination from the same roof.

The two questions are answered by observation of the phenomena of *accumulation* and *wash-off*. Figure 12.2 shows the contaminants on a roof over a period of time. The accumulation occurs over a dry period and shows a slow net build-up of material on the roof. Wind, passing traffic and other factors will deposit and remove material so the overall accumulation will fluctuate (red line) but rise over a period of time (pink line). Accumulation is further discussed in Section 12.1.2. The blue line shows the wash-off which occurs during the rainstorm and both delivers material to the tank and reduces the material on the roof. Wash-off is the best understood aspect of first-flush and usually the only aspect considered in the specification and design of first-flush systems. It is further discussed in Section 12.1.1. When the storm ends, accumulation will restart from the amount of material remaining on the roof at the end of the storm.

Figure 12.2: Accumulation and washoff



12.1.1 HOW MUCH TO FLUSH – CONTAMINANT WASH-OFF

There has been a fair amount of literature dedicated to question of the first flush volume. The most cited paper in the rainwater harvesting literature is that of Yaziz et.al. [1989] where a number of experiments based on fixed volumes (aliquots) are described. He suggested, as a rule-of-thumb *for the roof studied,* diverting 5 litres of foul flush. Other publications have recommended "between 1 and 2 gallons per 100 ft^2 of roofing", i.e. 0.4–0.8 mm of rain. [TWDB, 2005] and, "20-25 litres for an average size roof" [enHealth, 2004]. These rules-of-thumb have a large number of built in assumptions that may or may not be true. Often, particularly in rural areas of low-income countries, they substantially underestimate the optimum amount to flush. Little attention is paid to device reset and it is usually assumed that the device will be fully reset before each storm.

There has, however also been a great deal of interest and research into the first-flush phenomenon from engineers involved with stormwater runoff in urban areas as economies can be made in removing pollutants from stormwater by only processing the initial highly contaminated water, allowing the rest to flow without treatment. A number of formulae have been developed that are routinely included in such software as the US-EPA Stormwater management model (SWMM) [US-EPA, 2003]. The physics of deposition and removal of material on urban hardscape is similar to that of roofs, however there are several important differences.

- Roof catchments are more closely-coupled to their destinations so spatial effects (such as delays in material arriving at an outlet) will be much reduced
- Roofs have few different materials and the materials used tend to be smooth
- The slope of roofs is generally greater than that of streets
- Roofs tend to be corrugated in some way and so are dominated by channel flow rather than sheet flow

Sartor and Boyd [1972] developed an equation to describe the changes in runoff quality from streets as a function of rainfall intensity and time. The equation is based on the assumption that the material washed off a surface is proportional to the amount of material present on the surface and the rainfall intensity:

$$\frac{dL}{dt} = k_w i L \qquad \text{Equation 12.1}$$

Which integrates to:

$$L = L_0 e^{-k_w i \Delta t} \qquad \text{Equation 12.2}$$

Where; L is the contaminant load remaining at the end of a time-step; L_0 is the initial contaminant load available to wash-off prior to the time-step; k_w is the wash-off constant (mm^{-1}); i is the rainfall intensity (mm/hr) and Δt is the time-step (hr)

This equation fits Sartor and Boyd's data well and has since been confirmed by many other studies [Alley, 1981; Grottker, 1987; Pitt, 1987; Baffaut & Delleur, 1990; Millar, 1999; Chen & Adams, 2007].

The variable, Δt can also be restated as 'time since storm started' and L_0 as the load prior to the storm, in which case the product of i and t is simply the cumulative depth of rain that has fallen so Equation 12.2 can be stated in terms of accumulated rainfall (r).

$$L = L_0 e^{-k_w r} \qquad \text{Equation 12.3}$$

This formula provides a slightly simplistic view of first flush but does point to an exponential reduction as the storm progresses. In Sartor and Boyd's study, the value of k_w was found to be sensitive to location, pollutant, street texture, but insensitive to rainfall intensity and particle size. In stormwater modelling, it is usually set to a default value of 0.18 mm^{-1} [Alley, 1981], but the actual value to be used for k_w is the subject of much controversy and most recent authors recommend it be determined locally. In any case, the differences between roof and road catchments mean that the values of k_w for roof catchments are likely to be substantially different to that of road catchments.

In roofwater harvesting, the quality of runoff is the main issue rather than the water's ability to clean the surface. These two factors are, however directly linked as the contaminant being washed off the surface (and into the tank) is proportional to the remaining dirt load, Equation 12.3 can be simply restated, in terms of clarity of the wash-off water and the wash-off constant redefined in terms of changes in cleanliness of runoff rather than changes in remaining contamination.

It has been argued that rainfall *intensity* is the controlling factor rather than volume [Duncan, 1995], because larger particles require more kinetic energy to dislodge them either in the sheet flow or by the scouring action of raindrops. Yaziz [1989] found some connection between contaminant removal and storm intensity though the correlation was uncertain. Several studies have shown that the particle-size distribution of street dirt is coarser after a storm than before [Heaney et al., 1999; Vaze & Chiew, 2002] indicating that rainfall does selectively wash-off finer particles. As can be seen from Equation 12.1, the Sartor-Boyd equation does take rainfall intensity into account, assuming that more intense rain will remove proportionally more particles; however, it is not sensitive to intensity when taken over a particular rainfall amount as, while an intense rain will remove more pollutants (and likely larger or stickier pollutants) in a particular *time*, for a given *depth* of rainfall, an intense storm will be shorter so the overall pollutant reduction will be the same – though the particle size distribution will likely change. It is argued that this change in *type* of pollutant will, in fact also change the amount of pollutant moved and this can be described by reducing the initial load (L_0) in the Sartor-Boyd equation by multiplying it by an availability factor (*A*), which is related to rainfall intensity by the empirical equation [Novotny & Chesters, 1981]:

$$A = 0.057 + 0.04i^{1.1} \qquad \text{Equation 12.4}$$

The value of the availability factor, *A* starts out low and rises until, when it reaches 1, it is presumed that the storm's energy is enough to move all the washable material. *A* reaches this value at intensity i =18 mm/hr. The relation was derived from large, flat, course catchments that require a great deal of energy to remove particles from the course surface and transport them across the flat stretch in sheet flow. Conversely, most roofs require much less energy to dislodge particles from their smooth surface and then transport them by channel flow in corrugations down a slope[1]. It is therefore likely that equilibrium will be reached at a rainfall intensity much lower than 18mm/hr and, as tropical rainfall is also typified by high intensities, one might assume *A* has a value close to 1 for sloped roofs in the tropics and so rainfall intensity effects are likely to be negligible.

12.1.2 HOW LONG BETWEEN FLUSHES – CONTAMINANT ACCUMULATION

The speed of contamination accumulation during a dry period is a contentious issue. It is generally agreed that there is a dynamic balance struck between the processes of deposition and wind-driven removal. *Deposition* of debris/dust is considered a linear phenomenon as the likelihood of material being deposited is much the same one time-period to the next. *Removal*, is however, non-linear as it will depends on the level of material already accumulated and how firmly it is attached. It is usually removal and its resultant non-linearity, that confuses any modelling This confusion is exacerbated by the noisiness of the datasets that characterise all studies of material build-up on streets [Duncan, 1995] caused by the probabilistic nature of both deposition and removal. Most road surfaces also have means of material removal (such as the passage of traffic) not available to roofs. Several attempts have been made to model contamination build up on roads and these have some relevance to roofwater harvesting systems.

The most commonly applied build-up equation was developed by Shahhen [1975] and has had some success in modelling contaminant build up on roads [Alley & Smith, 1981; Grottker, 1987; Baffaut & Delleur, 1990; US-EPA, 2003; Chen & Adams, 2007]. It considers material deposition to be linear and removal to follow the same rules as first flush:

[1] An exception to this are the flat roofs found in dry areas such as the north of India and the Middle East.

$$\frac{dL}{dt} = \dot{L}_d - k_r L \qquad \text{Equation 12.5}$$

Which integrates to:

$$L = L_{\max} \;\; 1 - e^{-k_a t} \qquad \text{Equation 12.6}$$

Where L is the contaminate load; L_{max} is the maximum contaminate load that can be sustained by the surface, or more specifically the equilibrium load where the deposition and removal processes balance; $\dot{L}_d$ is the material deposition rate (contaminate/hr); and k_r is the exponential removal constant (hr^{-1}) – to avoid confusion with removal through washoff, this can be thought of as the "accumulation constant" (k_a) which has the same units.

Other authors that have proposed a linear build-up to a maximum load [US-EPA, 2003], a Michaelis-Menton relation ($L = L_{\max} \big/ \;\; k_d L + t$) [US-EPA, 2003], and a square law ($L = L_d - k_r{}^2 L + L_{end}$) [Pitt et al., 1999]. On streets, the time to reach a maximum load can be one week, however on roofs it should be considerably shorter as the materials are smoother and roofs are higher where wind tends to be stronger. It should also be noted that wind is also a primary force for *deposition* of material onto roofs, adding to the uncertainty. There is also some evidence that, as rainstorms are often associated with high winds, the wind during the rainstorm itself is a significant provider of deposited material [Evans et al., 2006].

12.2. DETERMINATION OF PARAMETERS – APPARATUS AND METHODOLOGY

To assess washoff and accumulation parameters to be used in Equation 12.2 and Equation 12.6, a large number of tests were carried in a number of tropical locations and with several roof types. The tests used a set of sequential discrete samples (aliquots) to measure the changes in runoff quality from the roof.

The apparatus used can be divided into two major components

- the catchment
- the sampler

The catchments were mainly specially constructed roofs consisting of several different roofing materials placed next to each other on a single structure (Figure 12.4a). Each section was about $1m^2$ in area and was separately guttered. The slope of the roof was typical of local housing made with that material. Roofs were made from a variety of materials;

- Corrugated galvanised iron sheet (GI),
- Corrugated asbestos sheet
- Clay tiles
- Tar sheet.

The catchments were located in a number of situations, namely:

- Near to a dirt road
- Away from the road in a compound
- Near to a busy tarmac highway

The sampler was based on the design used by Yaziz et. al. [1989]. It consisted of bottles connected together in series (Figure 12.3 & Figure 12.4c) so that the first bottle fills, followed by the second and so on. Each bottle was connected with a long pipe to constrict the zone of mixing and prevent subsequent water from mixing with that in an already full bottle. The whole apparatus was mounted at an angle to ensure each bottle fills in turn. As an exponential decay was expected, each bottle was generally larger than the one before. Several series of bottles were used as the experiments developed. The locations, roof types and bottle volumes are detailed in Table 12.1.

Figure 12.3: Sampler

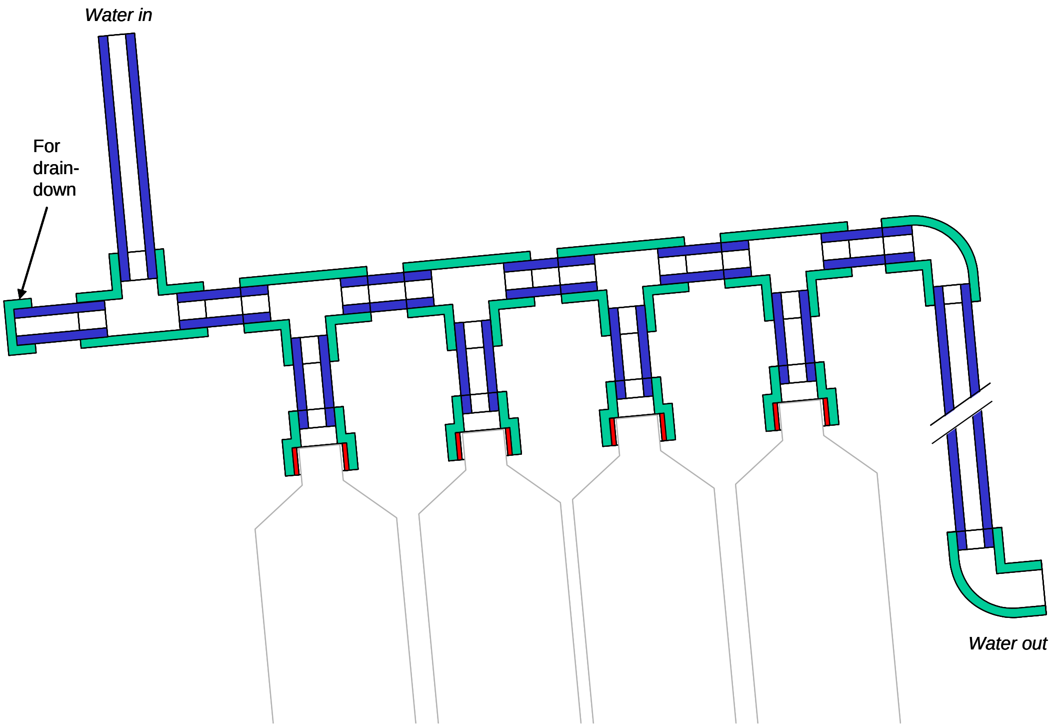


Figure 12.4: Equipment used in one of the experiments

a. Tile and GI roofs b. Location near to a dirt road c. Bottle array

Table 12.1: Characteristics of test locations

Location	Location	Roof materials	Roof area (m^2)	Bottle volume (litres *corresponding mm rain*)					
				1	2	3	4	5	6
Kandy	Away from road	Asbestos, GI, tar sheet	2.4	0.5 *0.21*	0.5 *0.21*	1.0 *0.42*	1.0 *0.42*	1.5 *0.63*	1.5 *0.63*
Colombo	Near busy tarmac highway	Asbestos, GI, tar sheet	2.4	1.0 *0.42*	1.0 *0.42*	1.0 *0.42*	1.5 *0.63*	1.5 *0.63*	1.5 *0.63*
Kampala 1	Near dirt road (see Figure 12.4b)	GI, Clay Tiles	1	0.5 *0.5*	0.5 *0.5*	1.0 *1.0*	1.0 *1.0*		
Kampala 2	Away from road	GI, Clay Tiles	1	0.5 *0.5*	0.5 *0.5*	1.0 *1.0*	1.0 *1.0*		

All horizontal pipe lengths were made as short as possible and their capacities recorded. A means of draining the pipes down was installed on the inlet end of the apparatus

As soon as possible after a rainfall event, the bottles were unscrewed from the sampler, briskly shaken to re-suspend any sediment and water from each bottle was tested for:

- Turbidity as a proxy for suspended material
- Conductivity as a proxy for dissolved matter (Colombo only)
- Thermo-tolerant coliforms as a proxy for risk of microbiological contamination (Kandy only)

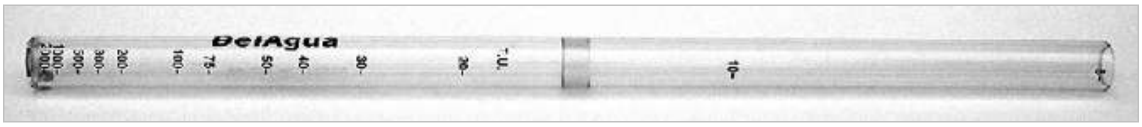
These values were recorded along with the date. After each test, the pipes were drained down and the bottles thoroughly rinsed to ensure contamination was not carried through from test-to-test. To avoid single rain events being recorded twice, the sampler was reassembled for the next set of measurements at least 4 hours after the end of the rainstorm.

Rainfall data from a nearby station was obtained to correlate with the runoff data.

12.2.1 LIMITATIONS OF MEASUREMENT

The turbidity measurement were taken with the turbidity tube illustrated in Figure 12.5. The tube has graduations at 5, 10, 20, 30, 40, 50, 75, 100, 200, 300, 500, 1 000 & 2 000 so results have a potential error of due to misinterpolating the non-linear scale. A linear interpolation between the readings will yield a reading that is above the actual value, however it is unknown how well this was carried out so, for the sake of calculation, interpolation errors were assumed to be unbiased and random and have been ignored. Furthermore, the tube has a maximum reading of 2000 NTU and a minimum reading of 5 NTU. As readings close to these limits were extremely uncertain, particularly towards the top of the scale, any readings outside the measurement range or within 10% of these values were discarded.

Figure 12.5: DelAgua turbidity tube

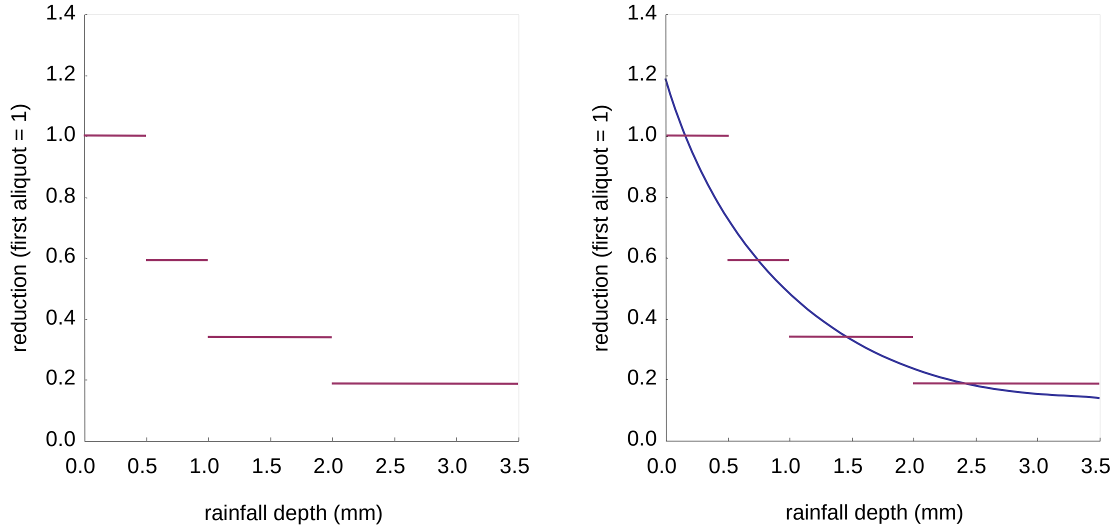


Colombo-based turbidity readings were taken using a Hanna C114 nephelometric turbidity meter and are assumed to be accurate.

12.3. DETERMINATION OF PARAMETERS – ANALYSIS OF WASHOFF RESULTS

Figure 12.6: Charts of first flush test data

a. alliquots b. exponential curve

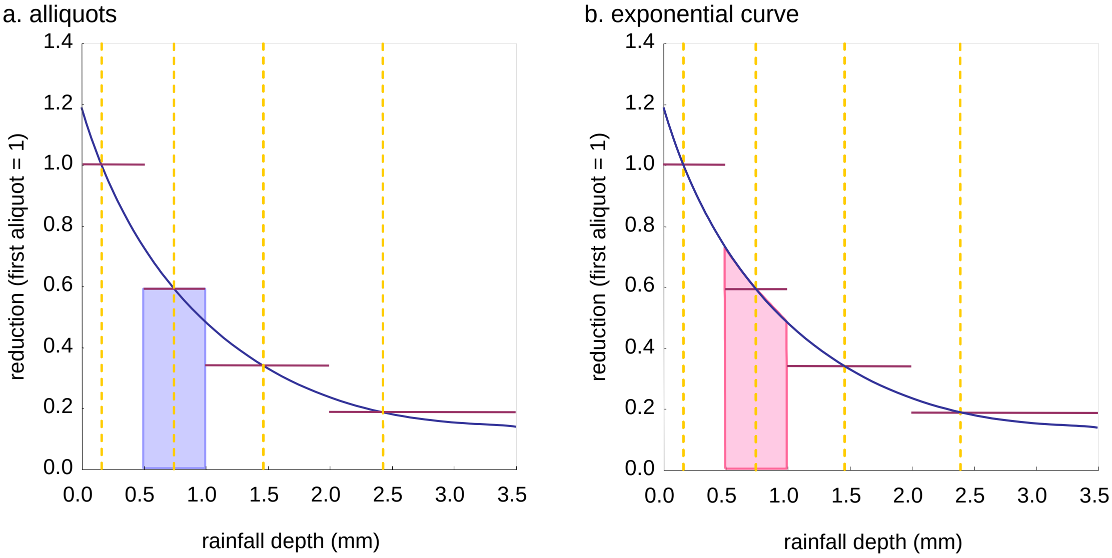


The results were normalised by dividing each measure by that of the first sample so the measurement of each aliquot was expressed as a fraction of the turbidity of the first aliquot. Each set of measurements can therefore be charted as shown in Figure 12.6a. Each line represents one aliquot sample. The height of the line represents the fraction of the first aliquot value and the width the rainfall depth caught in that aliquot. An exponential decay function can be fitted to the data as shown in Figure 12.6b. The curve starts higher than one, as each aliquot represents an averaging of the exponential change in runoff quality over the sample.

When measuring stormwater runoff, individual samples are taken at intervals throughout the storm, however when measuring roof runoff, from such a small catchment *all* the water must be captured in order to ensure enough volume for adequate measurement. The sampling of a continuously changing quantity by use of a series of a sequence of relatively large samples presents a problem for curve fitting as each sample represents a mixture of all the contamination delivered by the roof over the course of the aliquot. It is, therefore, insufficient to simply fit a curve to the central point of the sample as the non-linearity of the changes over the sample may be significant as is shown in Figure 12.7a.

A solution lies in the fact that mass must be conserved over the course of each aliquot and over the series. As the *total* contaminant contained in each aliquot is represented by the *area* under the lines as shown in Figure 12.7a it is possible to fit the curve by matching the area under the curve (Figure 12.7b) to the area under the horizontal aliquot line.

Figure 12.7: Areas of under curves



The area under the horizontal aliquot line is simply a rectangle and can easily be calculated, the area under the exponential curve can be found by integrating over the aliquot volume:

$$L = \left[-L_0 \frac{e^{-k_w r}}{k_w} \right]_a^b \qquad \text{Equation 12.7}$$

Where: L is the total load over the aliquot; L_0, k_w and r are as defined in Equation 12.3; and a and b represent respectively the beginning and end of the cumulative rainfall represented by the aliquot.

Having calculated the areas, these can be used to calculate a least squares fit for the exponential curve by iteratively substituting values for k_w and L_0. Microsoft Excel’s solver function which uses a “generalised reduced gradient” algorithm to observe the sensitivity of output values to input values and iterates until the output values converge to a desired outcome was used. In this case, the desired outcome was a minimum of the squares of the difference between the areas under the exponential curve and the areas under the aliquot lines. Initial attempts were sensitive to first estimates of the parameters, however constraining the solution so that the total area under the exponential graph in the region of sampling must be equal to the total area under the aliquot lines produced a stable solution.

In view of the limits to measurement described in Section 12.2.1, all measurements within 10% of the limits of measurement were discarded. If only two samples remained after this process, a range of curves would be possible, so the series was discarded.

A curve was fitted to each set of results and the values for L_0 and k_w noted along with the coefficient of determination. These parameters and the actual data are tabulated in Appendix E.

12.4. DETERMINATION OF PARAMETERS – WASHOFF RESULTS

12.4.1 TURBIDITY

There was considerable variation in washoff coefficient which had a roughly lognormal distribution as is shown in The values range over an order of magnitude and show a much greater variation between storms than between roofs and locations. The results were also insensitive to initial turbidity levels with similar values for k_w resulting from L_0 values ranging over three orders of magnitude. While it is unknown what causes the changes in k_w, candidate causes include:

- rainfall intensity
- wind blown dust during the storm or wind gust cleaning of the roof before the storm
- Atmospheric conditions such as pollution
- changes in type of solid matter (e.g. pollen vs. dust)
- preceding weather (very low intensity storms may not wash dust off but may wet clays enough to fix them to the roof)

Further research is needed to quantify or dismiss these factors. Overall, though the washoff coefficient is considerably higher than the default value for stormwater runoff of 0.18 mm^{-1}, supporting the hypothesis that roofs wash faster than horizontal paved surfaces.

The geometric means and ranges for the different roofs are shown in Table 12.2

.Figure 12.8: Exceedance plot of turbidity washoff coefficients

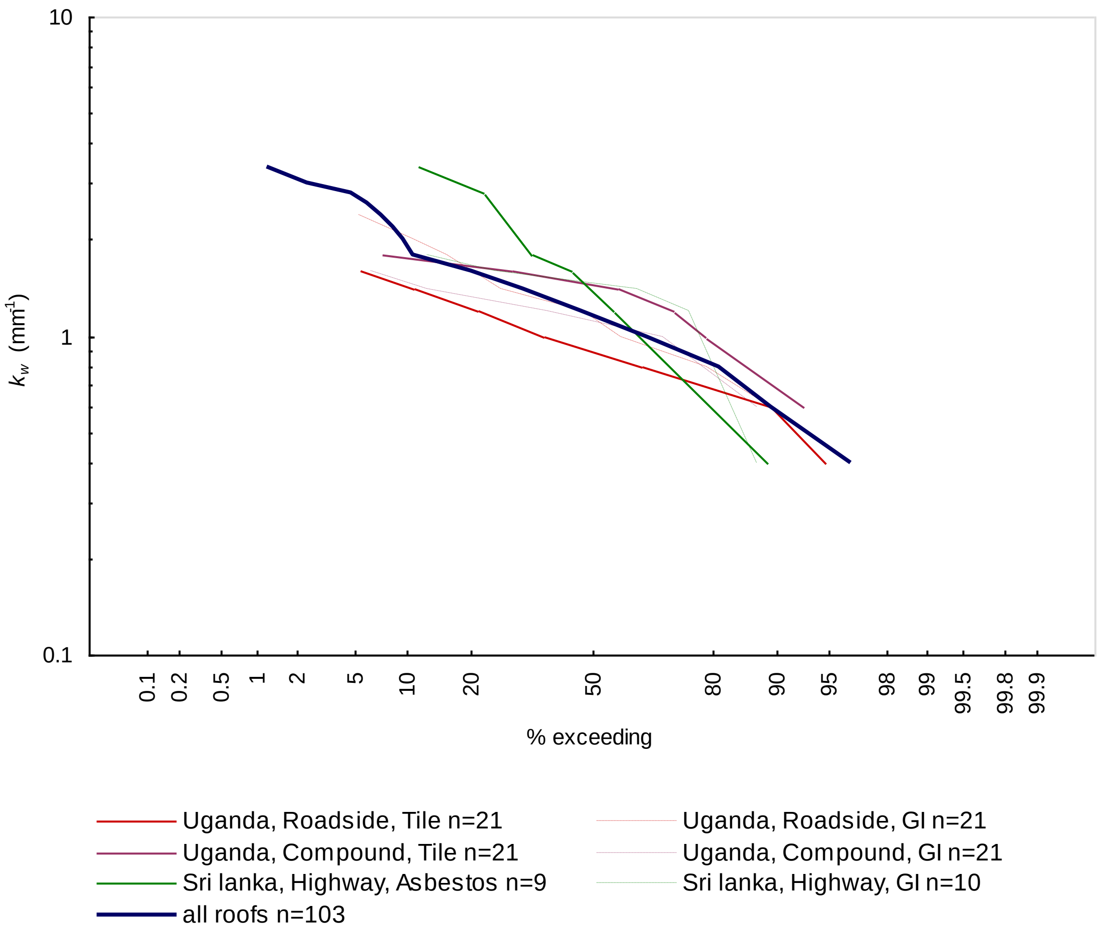


Table 12.2: Turbidity Washoff coefficient (k_w) for roofs

Location	Roof	Washoff coefficient (mm^{-1})	
		Geometric mean	Range
Colombo (Sri Lanka) – Near paved highway	Asbestos	1.52	0.31 - 4.40
Kandy (Sri Lanka) – 200m from road	Asbestos	1.71	0.97 - 3.27
Kabanyolo (Uganda) – Near dirt road	GI	1.44	0.58 - 2.82
Kabanyolo (Uganda) – 100m from road	GI	1.10	0.52 - 2.95
Colombo (Sri Lanka) – Near paved highway	GI	1.16	0.23 - 2.02
Kandy (Sri Lanka) – 200m from road	GI	1.35	0.79 - 2.41
Kandy (Sri Lanka) – 200m from road	Tar Sheet	1.56	1.19 - 1.98
Kabanyolo (Uganda) – Near dirt road	Tile	0.87	0.30 - 1.67
Kabanyolo (Uganda) – 150m from road	Tile	1.30	0.56 - 2.56
All roofs		1.19	

The values range over an order of magnitude and show a much greater variation between storms than between roofs and locations. The results were also insensitive to initial turbidity levels with similar values for k_w resulting from L_0 values ranging over three orders of magnitude. While it is unknown what causes the changes in k_w, candidate causes include:

- rainfall intensity
- wind blown dust during the storm or wind gust cleaning of the roof before the storm
- Atmospheric conditions such as pollution
- changes in type of solid matter (e.g. pollen vs. dust)
- preceding weather (very low intensity storms may not wash dust off but may wet clays enough to fix them to the roof)

Further research is needed to quantify or dismiss these factors. Overall, though the washoff coefficient is considerably higher than the default value for stormwater runoff of 0.18 mm^{-1}, supporting the hypothesis that roofs wash faster than horizontal paved surfaces.

12.4.2 DISSOLVED SOLIDS

Total dissolved solids (TDS) were measured in Colombo on three roof types. Again, the results follow a lognormal cumulative frequency distribution which is shown in Figure 12.9. The geometric means and ranges for the different roofs are shown in Table 12.3

Figure 12.9: Exceedance plot of TDS washoff coefficients

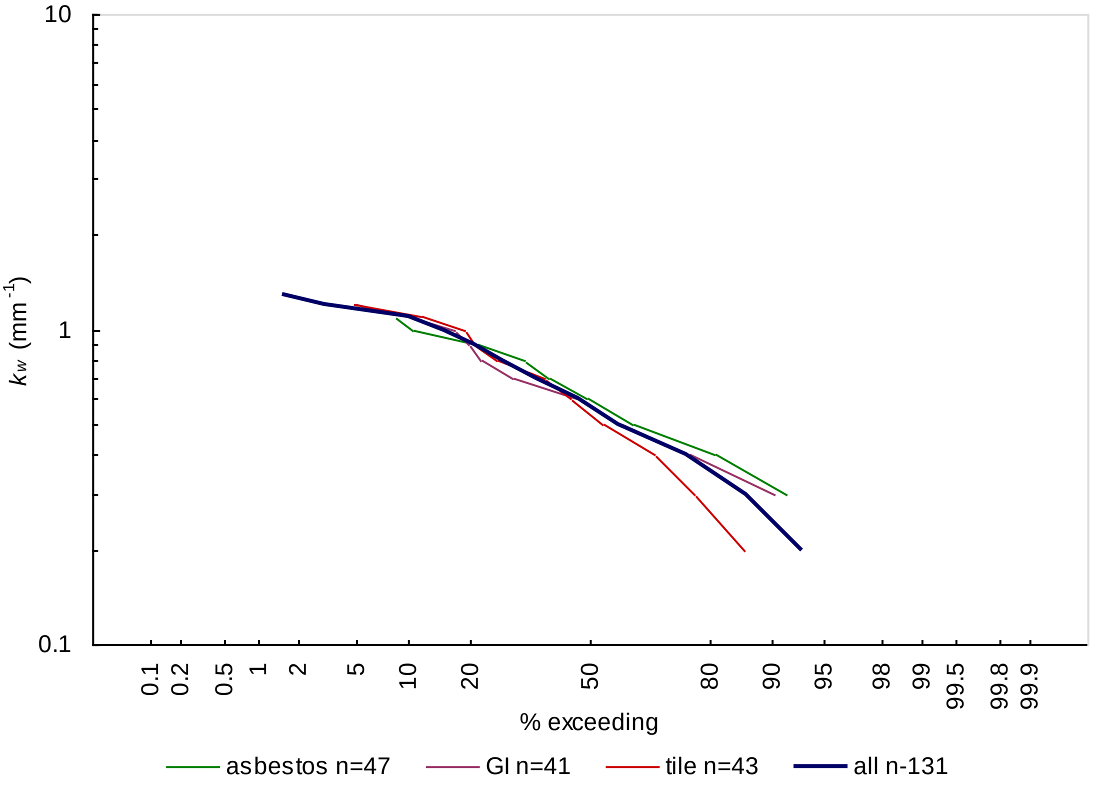


Table 12.3: TDS washoff coefficients for roofs

Roof	Washoff coefficient (mm^{-1})	
	Geometric mean	Range
Asbestos	0.60	0.08 - 1.20
GI	0.52	-0.22 - 1.26
Tile	0.55	-0.12 - 1.52

The coefficients show a slower washoff than for suspended matter. Again the coefficients show a wide range but are insensitive to roof type.

12.4.3 BACTERIA

Measurements for thermo-tolerant coliform bacteria were carried out at the Kandy site. Unfortunately due to insufficient rain, only three series could be analysed so these results can only be considered preliminary. The geometric mean of the washoff coefficients was 1.22 and the range; 0.39 – 4.67. These figures are similar to those found for turbidity, giving some weight to the hypothesis that bacteria are washed into the tank with suspended material.

12.5. DETERMINATION OF PARAMETERS: ACCUMULATION

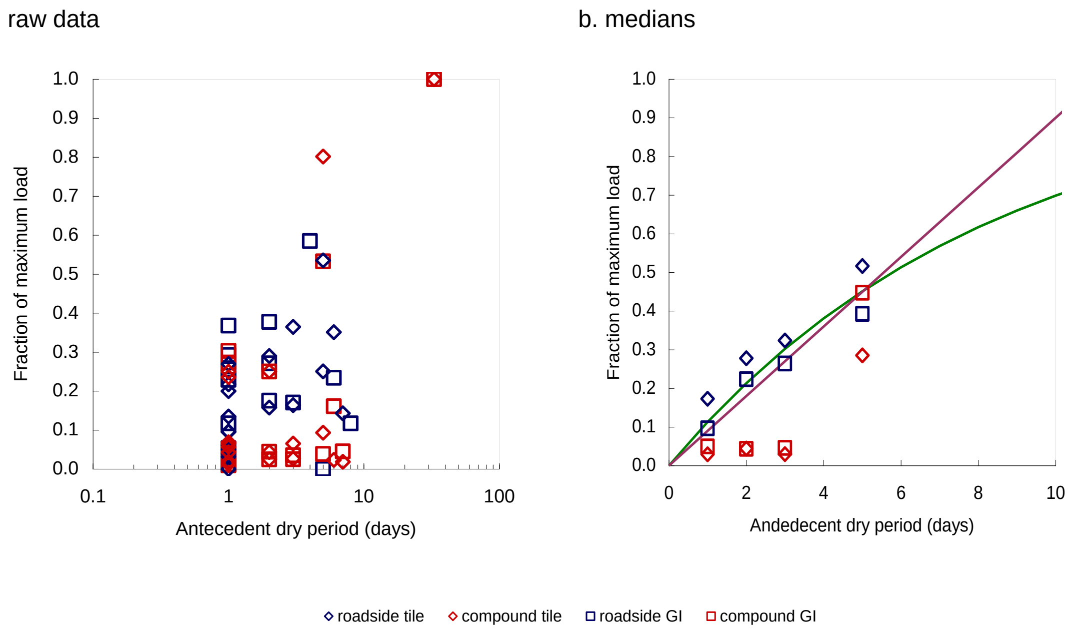
Rainfall data was collected from a nearby station to the location of the experiments, allowing the dry-days preceding the measurements to be found. As a storm does not necessarily leave the roof clean, it is not sufficient to simply count the number of days preceding a storm and determine the build-up from zero. A starting load, upon which new material will accumulate, must be set for each dry period. This starting load will be the amount of material left on the roof after it is washed by the preceding storm. Runoff quality was not measured for every day it rained so two kinds of washoff must considered:

- For measured storms, real data can be used so Equation 12.3 was applied for the total day's rainfall with the k_w and L_0 as obtained in the washoff analysis for that day

- For unmeasured storms, artificial values for L_0 and k_w must be determined so the value for L_0 was calculated using Equation 12.6 over the antecedent dry period with an initial estimate of the accumulation coefficient which was iterated for best-fit. The geometric mean value for k_w of 1.19 mm^{-1} was used for calculating the washoff

When these calculations are done and the initial load is charted as a fraction of the maximum load over the course of measurement, the very noisy data set illustrated in Figure 12.10a is revealed. There is a rising of the range of initial loads with antecedent dry period, however the loads found vary from very low to a rising upper limit.

Figure 12.10: Accumulation of suspended solids on roofs in Kabanyolo



Removing the noise by taking the median value for each day results in the shown in Figure 12.10b. a rise in initial load is clear and can be fitted to a curve of Equation 12.6 with an accumulation coefficient of 0.13 day^{-1} resulting in a 90% maximum accumulation in just under 18 days and 99% in 35 days. The highest L_0 associated with a measurement is found after a dry period of 33 days, so this may be an underestimate of the accumulation rate, it is however very close and is certainly indicative of the scale of the accumulation which seems much slower than expected. It is also possible to fit a line which reaches the maximum in just over 11 days, however the fit is not as good as the exponential function.

Applying such a slow accumulation to the resetting of a first-flush device is a technical challenge. The vast majority of first flush devices in the field use a weep hole to empty out the device resulting in reset times of several minutes. Even assuming a hole small enough to reset the device in several days was technically possible, it would certainly block thereby removing the automatic reset action. Manual reset is possible however this relies on user intervention, which is not reliable. A device that has the potential to reset in days rather than hours is under development as a series of undergraduate projects at the University of Warwick [Crabbe, 2004; Knight, 2005; Whiffen, 2006]. It relies on seepage through a clay disc to reset in a few days and is insensitive to even a large dirt accumulation in the device.

12.6. CONCLUSIONS AND RULES-OF-THUMB

The first-flush effect has been shown to exist on roof catchments and successive sampling reveals that the effect is well described by exponential washoff as described by Sartor and Boyd [1972]. Washoff coefficients are fairly widely distributed but are insensitive to design variables such as roof type and slope and to location. The coefficients are significantly higher than those recommended for stormwater runoff but do suggest larger optimum first flush volumes than have been previously recommended in the literature. The accumulation of material is also much slower than expected and very much slower than the reset times of currently available devices, resulting in a major technical challenge to develop a robust, reliable slow resetting device that can be made cheaply enough to be affordable to poor people in low-income countries.

The wide range of washoff coefficients makes providing a rule of thumb problematic, however the relatively low sensitivity to design variables supports the use of a single washoff coefficient in sizing first-flush systems. A fairly conservative approach to determining a single recommended washoff coefficient is to set it at a low value of 0.7 mm^{-1}. This value is exceeded in 85% of storms for suspended matter and (based on initial estimates) for bacteria and in 50% of storms for dissolved matter. As dissolved material tends to be at a lower level than suspended and is usually of a much lower health significance, the k_w value of 0.7 can be justified. Solving Equation 12.3 with this figure yields a simple to remember and communicate rule-of-thumb:

"For each mm of first flush, the contaminate load will halve"

Applying this rule to reduce very dirty (say 2000 NTU) water to the high WHO standard of 5 NTU [WHO, 1997], the first 8.5 mm of rain must be diverted. This is an extreme case and was only encountered at sites close to a dirt road and after a long dry period. The average initial turbidity near a dirt road was closer to 900 NTU (needing 7.5 mm of first flush to reach WHO standards), away from roads the average dropped dramatically to 150 NTU (needing 5 mm of first flush).

The large amounts of first-flush water indicated by this rule look alarming, however placing them in an mass balance model is reassuring. The impact of first-flush diversion on water yield is the subject of the next chapter.

These examples assume that the water entering the tank must be of WHO quality – Actually quality standards refer only to water at the tank outlet. Further processes such as mixing and sedimentation take place in the tank that substantially reduce the water's turbidity. In addition, the turbidity should be averaged over the total water entering the tank after the first-flush diversion. This average will be lower than that measured immediately following first-flushing.

13. MODELLING THE EFFECT OF FIRST-FLUSH REMOVAL ON WATER YIELD

One of the primary concerns among householders considering installing a first-flush system is the loss of water that will result. This is particularly true if the large amounts indicated in Chapter 12 are to be removed from the incoming stream as these appear to form a substantial fraction of many rainstorms.

Both filters and first-flush diverters change the inlet stream; reducing the contaminant load, but usually also reducing the water delivered to the tank. This paper discusses the characteristics of both these methods, with particular regard to two interacting measures of performance:

- Removal efficiency – the fraction of the contaminant load is removed from the incoming stream
- Volumetric efficiency – the fraction of the incoming stream that can be used by the system.

In both cases, any action to increase one of these measures usually results in a reduction in the other meaning that some balance must be struck between them.

The changes to the inlet stream also interact with the overall system. This results in significant differences between the changes produced by an intervention on the incoming stream and the resultant changes to the outflow from the entire system. To investigate the effect of a change in the incoming stream on the performance of a rainwater harvesting system, a mass-balance approach is taken where the modified input stream is used in a model of the behaviour of an entire rainwater harvesting system. The modified system behaviour is then compared with the behaviour without the appliance to gauge its effects.

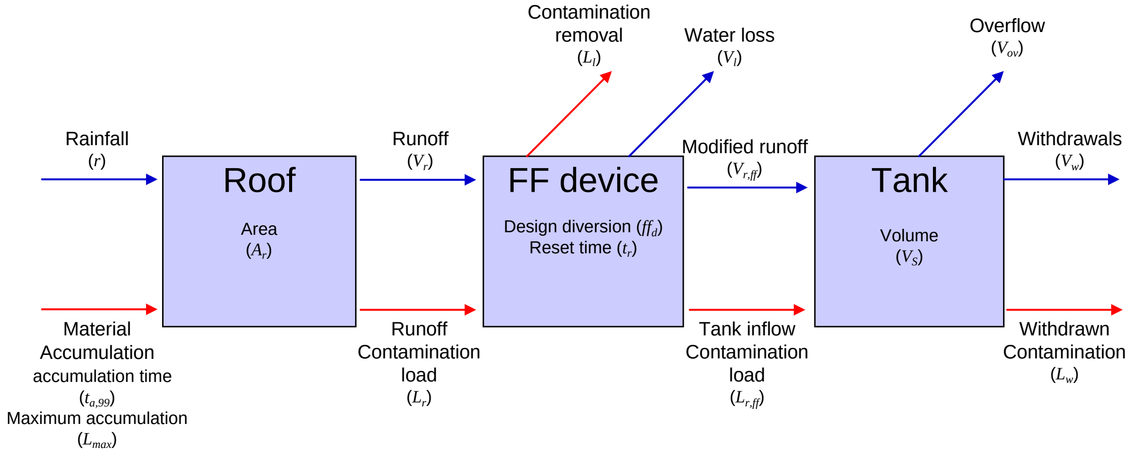
The outputs from the model are presented as a series of performance diagrams; which can be compared for a desired quality improvement or required volumetric output. Several such charts are presented for a range of system sizes, water drawing behaviour and for a number of climate types.

13.1. THE EFFECT OF FIRST-FLUSH ON SYSTEM PERFORMANCE

13.1.1 CONTAMINANT AND WATER FLOWS

The performance of a first-flush system depends on the flow of contaminants and water through the system and the behaviour of the first-flush device itself. Such a flow is shown in Figure 13.1. Chapter 12 was concerned mainly with measuring the flow of water and contaminates off the roof, this chapter is concerned with how these flows interact with first-flush device behaviour and with the storage tank.

Figure 13.1: Contaminant and water flow through a RWH system with first-flush



Material accumulates on the roof as discussed in Section 12.1.2 according to an exponential function with a maximum possible accumulation (L_{max}) and a time constant (k_a). The water and contaminant material are then washed off the roof and into the first-flush device with the contaminant concentration reducing exponentially with accumulated rainfall as described in Section 12.1.1.

The first-flush device diverts a certain amount of the rainfall depending on its volume and allows the remainder to enter the tank. As the reduction in contamination from the roof is described in terms of accumulated rainfall, it is convenient to discuss first-flush devices in the same terms; dividing both runoff and first-flush volume by roof area achieves this aim. The design diversion (ff_d) of the first-flush device represents the maximum that can be flushed, however once the rain has stopped, a well designed diverter will slowly reset over time and so when it next rains the device may not be completely reset. As a result, the actual first-flush diversion (ff) will increase with antecedent dry period until the full design diversion is reached after the complete reset time (t_r).

Once the first-flush device has diverted the appropriate amount of runoff, the remaining water is then allowed to enter the tank. Thus there will be reduced water flow ($V_{r,ff}$)and a reduced contaminant flow ($L_{r,ff}$) delivered to the tank.

Water is then withdrawn from the tank or allowed to overflow according to a mass balance as described in Section 2.4.1, where the incoming stream (V_r) is replaced by the reduced water flow from the first-flush device ($V_{r,ff}$). The contaminants will also undergo reduction processes within the tank such as die off, sedimentation, chemical reactions, and biofilm action. These processes can either be represented by exponential functions or something similar which are directly proportionate to initial values, so it has been assumed in the analysis that contamination load in the withdrawn water will be proportional to the load at the tank inlet.

For ease of communication with non-engineers, particularly users, the time constant, k_a can be restated in terms of time to reach a certain level, referred to as “accumulation time” In this case 99% of maximum has been chosen ($t_{a,99}$) but this is arbitrary and any level could be used. Similarly the washoff constant, k_w can be expressed in terms of a “washoff rate” which is the fraction of contaminant removed per mm of rainfall.

It is in the interaction of the washoff rate and design diversion, and the accumulation and reset times that the performance of the first-flush device on the incoming stream of water rests. The tank volume also plays a role as some water will simply overflow and tank diseconomies of scale and user demand behaviour will have an effect.

13.1.2 PERFORMANCE MEASURES

As with filters, first-flush systems have two competing performance measures which affect each other. The more water that is diverted by the first first-flush device, the cleaner the water delivered to the tank will be, however greater diversion will also mean less water will be delivered to the tank. Balancing these factors is key to rational first-flush device design and maximising water delivery for a given cleanliness is a route to an optimised system.

REMOVAL EFFICIENCY

The removal efficiency of a first-flush system is a measure of how well it removes contaminants from the incoming water stream. It can simply be defined as the ratio of contaminant removed by the first-flush system (L_l) to the total contaminant load washed off the roof (L_r):

$$\eta_r = \frac{\sum L_l}{\sum L_r} = \frac{\sum L_r - L_{r,ff}}{\sum L_r} \qquad \text{Equation 13.1}$$

The measure can either be applied over an individual storm or over a number of storms. In this thesis, the removal efficiency is applied to the entire time series to give the overall performance of a particular system.

VOLUMETRIC EFFICIENCY

The volumetric efficiency is a measure of how little water is “wasted” by the first-flush system. It can be measured in two places; the tank inlet and the tank outlet. Discussions with water professionals and users indicate that most people tend to consider only the loss to the incoming stream with comments such as “it will throw away X% of the water” common when discussing first-flush systems. In reality it is the loss at the tank *outlet* reflecting the reduction in available *withdrawals* that is the real loss to the user. The efficiency when measured at the tank outlet differs from that at the inlet and is usually higher.

Volumetric efficiency at the tank inlet ($\eta_{v,i}$) can be calculated by dividing the sum of runoff after first-flush diversion by the sum of the runoff without diversion, as the roof area is the same for both, the $\eta_{v,i}$ can simply be calculated using the rainfall (r) and first-flush diversion (ff):

$$\eta_{v,i} = \frac{\sum V_{r,ff}}{\sum V_r} = \frac{\sum r - ff}{\sum r} \qquad \text{Equation 13.2}$$

Volumetric efficiency at the tank outlet is calculated by using $V_{r,ff}$ in place of V_r in a mass balance and dividing the total withdrawals from the system with the first-flush diverter ($V_{w,ff}$) by the total withdrawals from a separate mass balance without first-flush diversion (V_w)

$$\eta_{v,w} = \frac{\sum V_{w,ff}}{\sum V_w} \qquad \text{Equation 13.3}$$

13.2. DEVICE RESET MODELS

Automatic first-flush systems rely on a fixed volume or mass of water to activate the device and are reset over time by the volume of water emptying out. During rainfall the roof runoff first fills the device and then when it is full, the water is switched towards the main storage tank. When the rainfall ceases, the container holding the needed volume slowly empties, more of its volume becomes available for diverting the first-flush of the next storm. As a result, first-flush systems reset gradually with the available diversion volume varying between zero and the full device volume depending on how much time has passed since the last rain event. As the device is designed so the volume is proportional to the design diversion (ff_d), the available FF *diversion* (ff) will also vary the same way. This is convenient as the accumulation of contaminants on a roof also takes place over a period of time so a sensible design strategy would appear to be to match the reset to the accumulation as closely as possible to avoid excess flushing or excess contamination entering the tank.

Emptying of first-flush devices is driven by water pressure and so rates of emptying are functions of the height of water within the device. The available FF diversion (ff) is proportional to the *available* volume (V_a) in the device which is the volume of the container available to be filled with water i.e the design volume (V_d) minus the actual volume of water in the device (V). First-flush devices are primarily of uniform cross section so water height is proportional to water volume and so:

$$\frac{ff}{ff_d} = \frac{V_a}{V_d} = \frac{h_a}{h_d} = \frac{V_d - V}{V_d} = \frac{h_d - h}{h_d} \qquad \text{Equation 13.4}$$

Where: h_a is the height that is available to be filled, h_d is the design height V_a is the volume that is available to be filled.

Equation 13.4 shows that the proportion of reset of a first-flush device is directly equivalent to the proportional changes in height in the device and so analysis of device emptying can be carried out in terms of height only.

13.2.1 TURBULENT EMPTYING

The method for emptying almost all automatically resetting first-flush devices is a small "weep" hole, which allows water to slowly flow out. For the majority of the emptying, the flow is turbulent so this type of emptying can be described by energy conversion of static head to velocity as described by Bernoulli's equation:

$$h = \frac{v^2}{2g} \qquad \text{Equation 13.5}$$

Combining this with continuity and integrating over time yields:

$$h = \begin{cases} \left(\sqrt{h_0} - t \frac{A_h}{A_c} \frac{\sqrt{2g}}{2} \right)^2 & t < \frac{A_c}{A_h} \sqrt{2h_0 g} \\ 0 & t \geq \frac{A_c}{A_h} \sqrt{2gh_0} \end{cases} \qquad \text{Equation 13.6}$$

Where: A_h is the area of the hole; A_c is the cross horizontal sectional area of the container; and h_0 is the height of water in the container at the beginning of emptying

As discussed above, for a resetting first-flush device, the height remaining in the device is not as important as the height that is available for diverting the next first-flush. This is the difference between the remaining height (h) and the maximum height that the design height (h_d) after which the device will allow water to be delivered to the tank i.e.:

$$h_a = h_d - h \qquad \text{Equation 13.7}$$

In the case of turbulent emptying, this height is given by combining Equation 13.6 and Equation 13.7:

$$h_a = \begin{cases} h_d - \left(\sqrt{h_0} - t \frac{A_h}{A_c} \frac{\sqrt{2g}}{2} \right) & t < \frac{A_c}{A_h} \sqrt{2h_0 g} \\ h_d & t \geq \frac{A_c}{A_h} \sqrt{2gh_0} \end{cases} \qquad \text{Equation 13.8}$$

It is possible to use Equation 13.6 to calculate the emptying time of a typical downpipe-based first-flush system such as that shown in Section 11.3:

E.g. if a 110 mm ID downpipe is used and the pipe is 2.5 m long, its volume will be about 24 litres, equivalent to just under 0.5 mm of first-flush from a 50 m^2 roof. If a 5 mm hole is used to reset the device, the reset time will be just over 5 minutes; if a 2mm hole is used, the reset time will be 33 minutes; for a one-day reset, a hole size of 0.3 mm is required. Making the very small sized holes needed for long reset times in weep-hole emptied devices is a technical challenge beyond all but the best machine shops in low-income countries. Moreover, such small holes are extremely likely to block in a very short time.

For modelling purposes the constant and design parameters within Equation 13.8 can simply be collected into a constant:

$$h_a = \begin{cases} h_d - \left(\sqrt{h_0} - k_r t\right)^2 & t < k_r \sqrt{h_0} \\ h_d & t \geq k_r \sqrt{h_0} \end{cases} \qquad \text{Equation 13.9}$$

Where: k_w is a reset constant; h_0 is the water depth at the end of the previous rain event; and t is the time elapsed since that event ended.

13.2.2 LAMINAR EMPTYING

If the emptying is slow enough, the flow will become laminar and viscosity will dominate. This is particularly true if the reset mechanism is based on flow through a porous medium. Laminar flow is described by Darcy's Law:

$$Q = -kA \frac{\Delta\left(\dfrac{p}{\rho g} + z\right)}{L} \qquad \text{Equation 13.10}$$

Where: k is the hydraulic conductivity (with dimensions of L/T); A is the cross-sectional area in the direction of flow; p is the pressure; ρ is the fluid density; g is gravitational acceleration; z is the change in elevation across the medium; L is the length the pressure drops over.

Within a first-flush system, a number of assumptions can be made to simplify Darcy's equation:

- The pressure gradient is caused by the static head of the water on the inside of the device and atmospheric pressure on the outside, so the pressure change (Δp) is proportional to the total head (h)

- The change in energy over the length of travel (Δz) can be simply accounted for by taking the datum for h as the bottom surface of the permeable medium

- g and ρ are constant

- The flow area remains constant so continuity can be used to find the change in height per unit of time

- The pressure drop length is constant for a particular design so can be incorporated into k

With these assumptions, Darcy's equation reduces to:

$$q = k_r h \qquad \text{Equation 13.11}$$

Where: q is the "Darcy flux" which is the overall velocity of the water across the disc; h is the height of water in the container; k_r is the product of hydraulic conductivity and pressure drop length of the porous medium and has dimensions of T^{-1}

Integrating Equation 13.11 over time yields:

$$h = h_0 e^{-k_r t} \qquad \text{Equation 13.12}$$

Where: h_0 is the height of water in the container at the beginning of emptying.

Again, this can be rewritten in terms of the available height:

$$h_a = h_d - h_0 e^{-k_r t} \qquad \text{Equation 13.13}$$

When the device is full (i.e. letting water flow into the tank), $h_0 = h_d$ and Equation 13.13 can be rewritten as $h_a = h_d\left(1 - e^{-k_r t}\right)$ which closely matches exponential accumulation behaviour (Equation 12.6).

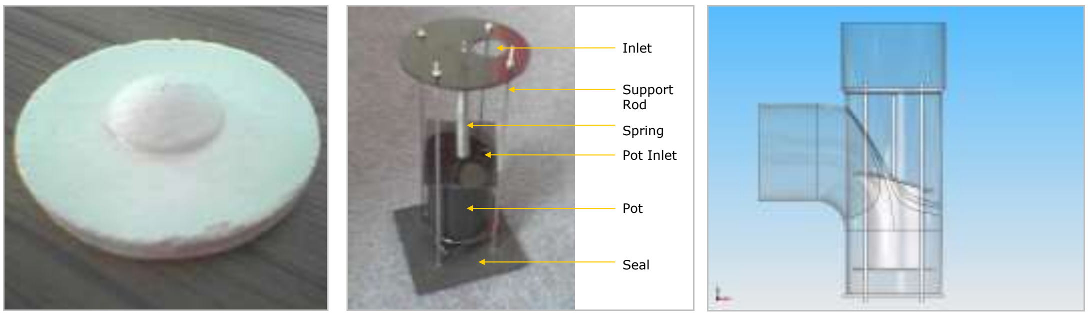
“Reset time” does not have a real meaning with laminar emptying as the exponential function is asymptotic, however if 99% empty is considered as “empty enough” a nominal reset time can be considered as the time taken to reach this level ($t_{r,99}$).

For the downpipe-based system described in Section 13.2.1, if a 1cm thick porous disc the same diameter as the downpipe is used to regulate the emptying. For a reset time of 1 day, the required hydraulic conductivity is 520E-9 m/s; for 3 days, 180E-9 m/s; for 7 days, 76E-9 m/s; and to match the accumulation profile found in Section 12.5, a hydraulic conductivity of 15E-9 m/s is required. These values are very small in comparison to those of porous materials such as clay used in some sterilizing water filters currently being manufactured in low-income countries (1.1 E-6 m/s) [Lantagne, 2001]. However with proper control these conductivities can be obtained. Reducing the area of permeable material and/or increasing its thickness will also reduce flow and increase reset time.

13.2.3 DEVICE TO SLOWLY EMPTY FIRST-FLUSH SYSTEMS USING POROUS CLAY

The use of clay disc (Figure 13.2a) as a permeable medium for emptying first-flush systems has been investigated in the form of several undergraduate projects[1] [Crabbe, 2004; Knight, 2005; Whiffen, 2006]. Reset times are sensitive to clay composition but can be controlled by changing material thickness and introducing appropriate amounts of sawdust into the clay before firing. Controlling the moisture content and level of compaction during manufacture has also been found to be critical. If correct manufacturing and quality control procedures are followed, the reset times are predictable and controllable with results in the order of several hours to several days.

Figure 13.2: First-flush diverter based on clay disc (from Knight [2005])
a. Clay disc b. valve assembly c. assembly mounted in tee

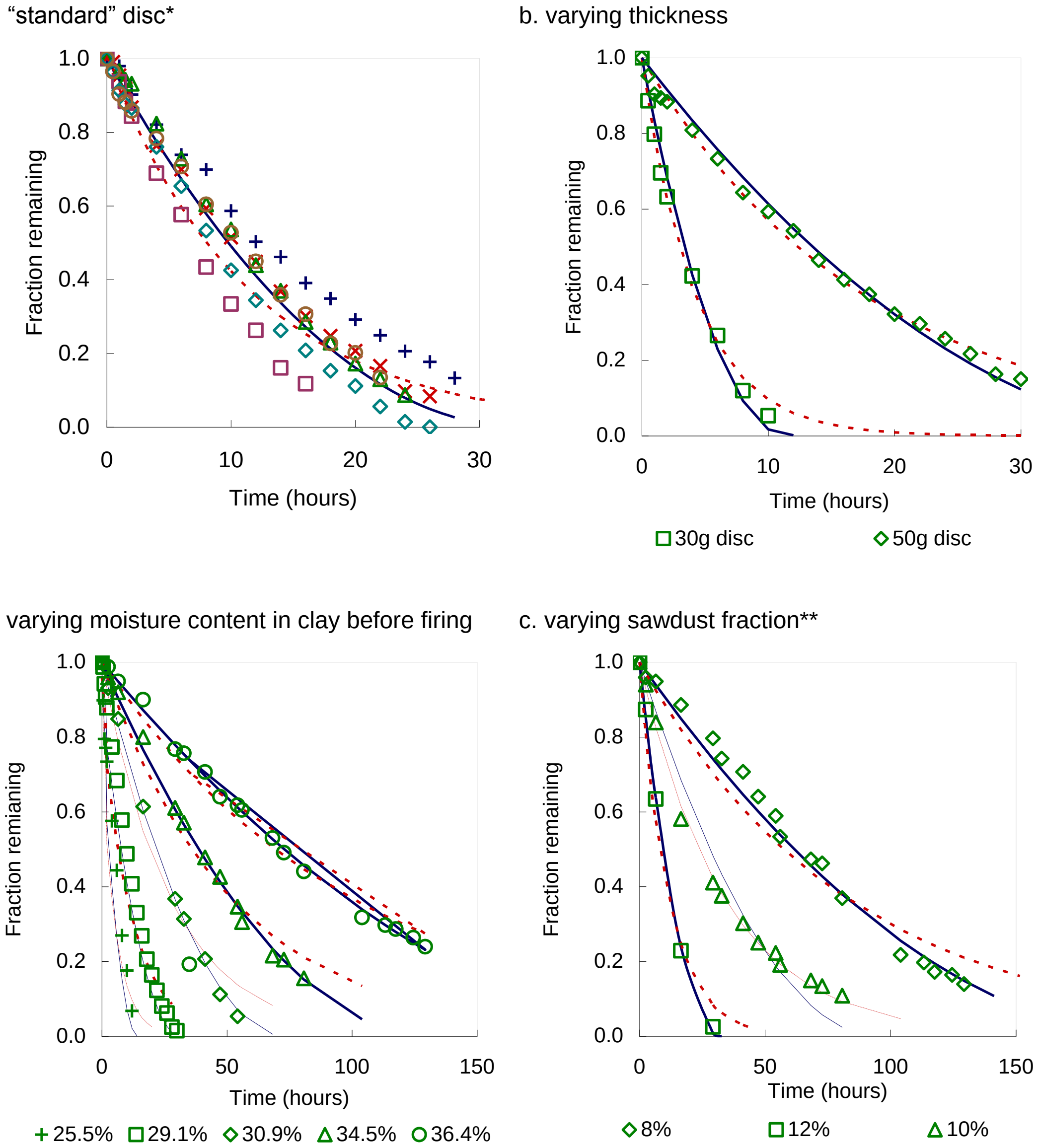


The actual mechanism is built into a standard plumbing tee and is depicted in Figure 13.2b and Figure 13.2c. It uses a small but fixed fraction of the runoff flow to fill a container which is

[1] Projects were carried out under my supervision

suspended from a spring or rubber band. As the container fills, its increased weight stretches the spring and the container descends onto a seal mounted to the bottom of the tee. When the container is less than full it hangs by the spring and the seal is not made; water is allowed to flow down and out of the device. The diverted water may be collected for non-potable purposes. Once the container is full, the seal is made, and water is redirected horizontally to the main tank. This is a similar arrangement to that used in designs by SafeRain [Church, 2001]and FloTrue [FloTrue, 2006], however drainage of the container is by slow seepage through the clay disc rather than through a weep-hole so the device operates by diverting a volume of water rather than reacting to flow-rate.

Figure 13.3: Water remaining after seepage through a clay disc (corrected for evaporation)



Based on data from Knight [2005]

* A “standard disc” is composed of 40g of kaolin clay with 29.1% water content and 10% sawdust added, compressed into a disc Ø60mm with a force of 5000N

** for the sawdust fraction tests, the clay moisture was adjusted to compensate for moisture in the sawdust.

In the course of the projects several tests were made of water flow through the clay discs. The results of some of these tests are shown in Figure 13.3. Two regression lines representing respectively, laminar and turbulent emptying models have been fitted to the data and, surprisingly, turbulent emptying is a better fit for the data in most cases. The turbulent (solid) and laminar (dotted) lines are, however very close and within the range of noise of the measurements. The times to achieve 99% emptying varied from 10 hours to over 10 days for the test containers demonstrating that slow reset times in the order of several days are possible using seepage through clay.

13.3. MODELLING ACCUMULATION AND WASHOFF

13.3.1 ACCUMULATION

While Equation 12.6 appears to adequately describe accumulation of material on a roof, each storm does not necessarily thoroughly wash the roof surface and there will likely be some material remaining on the roof after each storm – so simply applying the accumulation function to each dry period from zero is not an adequate solution. The problem has been traditionally dealt with in stormwater management by considering the time necessary to accumulate the remaining material and “backdating” the accumulation time by that amount [Alley & Smith, 1981]. So Equation 12.6 becomes:

$$L_a = L_{a,\max}\left(1 - e^{-k_a (t+t_0)}\right) \qquad \text{Equation 13.14}$$

Where: t_0 is the time that would be necessary to accumulate the material left on the roof at the end of the preceding storm ($L_{a,0}$). This can be calculated by rearranging Equation 12.6 with t_0 and $L_{a,0}$:

$$t_0 = -\frac{1}{k_a}\ln\left(1 - \frac{L_{a,0}}{L_{a,\max}}\right) \qquad \text{Equation 13.15}$$

A computationally simpler approach is to consider the accumulation in the same way as laminar FF device emptying. The maximum accumulated load ($L_{a,max}$) is analogous to the design height of the FF device (h_d), and the initial dirt load ($L_{a,0}$) is analogous to the available height $(h_d - h_0)$:

$$L_a = L_{a,\max} - \left(L_{a,\max} - L_{a,0}\right) e^{-k_a t} \qquad \text{Equation 13.16}$$

Either of these methods yields an equation describing material accumulation from any starting point. The starting load ($L_{a,0}$) for each accumulation event is the final load at the end of the preceding storm.

13.3.2 WASHOFF

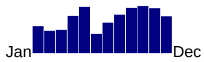
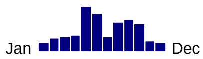
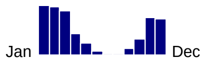
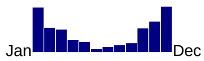
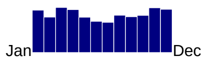
Washoff is modelled using Equation 12.3, taking $L_{w,0}$ as the accumulated load (L_a) calculated by Equation 13.16 at the end of the preceding dry period.

13.4. THE MASS-BALANCE MODEL

13.4.1 DATA USED AND ITS LIMITATIONS

Fifteen minutely data was obtained from the US National Climatic Data Center (NCDC product DS3260) representing a number of climate types and rainfall patterns as shown below inTable 13.1. The data has been chosen to reflect single wet season and bimodal rainfall distributions in both high and low rainfall areas. A typical temperate climate with medium rainfall without marked seasonality has also been included for comparison.

Table 13.1: Data sources for mass-balance models with 15-minutely data

State	Town	Köppen climate type	Mean annual rainfall (mm)	Rainfall Pattern
Puerto Rico	Corozal	Am	1 900	
Texas	Big Lake	BSh	480	
California	Blue Canyon	Dsb	1 700	
Hawaii	Kekaha	As	550	
Rhode Island	Newport	Cfa	1 200	

While the data gave good temporal precision, the rainfall itself was mostly reported in 1/10" (2.54mm) increments and so was fairly coarse. The exception to this was the data for Blue Canyon which was reported in 1/100" (0.24mm). In practice this did not affect results a great deal however it should be noted that storms of less than 2.54mm would be carried on to the next increment so a 15 minute reading of 2.54mm could actually indicate a much longer period of low rainfall intensity or several accumulated small rain events antecedent to the reading.

To save on document size, the dataset did not report zero values. This resulted in two different kinds of time-step:

- When rainfall was occurring there was a time-step every 15 minutes
- When rainfall was not occurring the time-step was calculated from the end of the preceding rain-event to the beginning of the next one and was therefore highly variable

13.4.2 MODELLING PROCEDURE

The mass balance model was similar to that described in Section 2.4.1, however, as the data was presented with two different time-steps, these were also used in the model. Modelling washoff was straightforward as it only occurred when there was rain (and therefore data and a fixed 15 minute time-step), however processes that occurred in the dry periods such as material accumulation and withdrawals from the tank had to be summed over the varying time-step.

The model handles contaminants in two forms:

- Contaminant concentrations, e.g. Turbidty (NTU) which is a proxy for total suspended solids, (dimension = M L^{-3}), Conductivity, a proxy for total dissolved solids (M L^{-3}), and bacterial contamination (Count L^{-3}). The accumulation and washoff equations describe the changes to these concentrations in terms of rainfall depth.
- Total contaminants, e.g, total mass of suspended solids (M), total mass of dissolved solids (M), total bacterial contamination (Count). These total loads can be determined by integrating the accumulation and washoff equations over time.

No attempt is made to differentiate the *type* of contaminant as all contaminants have shown to have similar behaviour in a first-flush system, though the rates of change may be different. The results of the mass balance will therefore be equally valid for dissolved and suspended material and also bacterial contamination.

For each time-step (raining or otherwise):

- The available first-flush diversion (*ff*) was modelled by calculating the device reset using the default of laminar-flow (Equation 13.13) and using the available height (h_a) in Equation 13.4. Several runs were made using a turbulent model (Equation 13.8) but results were very similar with laminar emptying consistently showing a percentage point or two better performance. “Reset time” ($t_{r,99}$) is 1 day (k_r = 3.2E-3 mins^{-1}).

The initial height (h_0) for the emptying was taken to be the height in the device at the end of the previous rainfall event.

- The total water withdrawn from the tank was calculated as the sum of demanded tank withdrawals subject to the constraint of total water available (V_s) in the tank:

$$V_w = \begin{cases} n_w V_{w,i} & V_S > n_w V_{w,i} \\ V_S & V_S \le n_w V_{w,i} \end{cases} \qquad \text{Equation 13.17}$$

Where: V_w is the total withdrawal volume; nw is the number of withdrawals over the time-step; $V_{w,i}$ is the volume of each individual withdrawal, given by:

$$V_{w,i} = \frac{V_d}{n_{w,d}} \qquad \text{Equation 13.18}$$

Where V_d is the daily water demand; n_{wd} is the number of withdrawals in a day.

The actual number of withdrawals depends on the behavioural model used. Different models were tried including 24 equal collections, 1 daily collection, 12 collections during a 12 hour period (6am – 6pm) and 3 collections during a 12 hour period. As rainfall only falls for a tiny proportion of the time, daily withdrawal behaviour made almost no difference to the output so withdrawals were simply modelled as just once per day.

Water demand was taken to be constant i.e. no demand management strategies were applied.

For each (variable) non-raining time-step:

- Material accumulation was modelled using Equation 13.16 resulting in an initial contaminant load ($L_{w,0}$) for washoff calculations at the start of the next raining time-step. The default accumulation time ($t_{a,99}$) was taken as 1 day (k_a = 3.2E-3 mins^{-1}).

For each (15 minute) raining time-step:

- The contaminant load in the water at the end of the time-step was calculated using Equation12.3 with the total rainfall (in mm) over the time-step. The default washoff rate was taken as that recommended at the end of chapter 12 namely that 1mm of rainfall halves contaminant load (k_w = 0.7 mm^{-1})

- The total contaminant washed from the roof in the time-step was calculated using by solving equation 12.7 for the rain falling in the time-step and multiplying by the roof area. In practice, this roof area calculation was not necessary as comparison is made between identical systems with and without FF, so the roof area is cancelled out.

- If the time-step ends before the first-flush device has activated, the remaining diversion capacity at the end of the time-step is calculated. This amount will be used as the starting point for calculating reset for the next time-step.

$$ff_{end} = \begin{cases} ff_0 - r & 0 \leq ff_0 - r \\ 0 & 0 > ff_0 - r \end{cases} \qquad \text{Equation 13.19}$$

  Where ff_0 is the available diversion at the beginning of the time-step; ff_{end} is the available diversion at the end of the time-step; r is the rainfall.

- The contaminant load at the end of the first-flush and total contaminants removed were calculated using the same method steps 1 and 2, but with the available first-flush diversion (*ff*) rather than the rainfall (*r*) (Note: if the available FF diversion was larger than the rainfall, the rainfall was used)

- If the rainfall over the time-step was greater than the available FF diversion, water was delivered to the tank. This reduced inflow was added to the store as described in Section 2.4.1

- A separate water balance was made based on directing the entire flow into the tank

Removal efficiency and volumetric efficiencies of the inlet water stream and tank outlet were calculated based on Equation 13.1, Equation 13.2 and Equation 13.3 calculated over the entire simulation period of ten years.

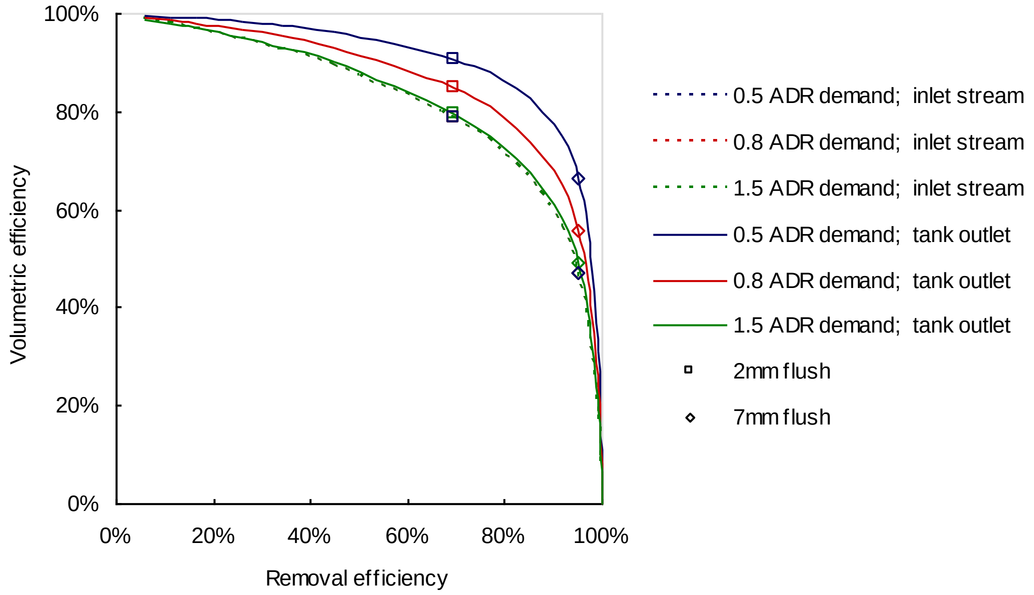
To create the charts in Section 13.5, a range of first-flush diversions were applied to the simulated system and removal and volumetric efficiencies noted. The volumetric efficiency was then platted against the removal efficiency. For reference, a 3mm and 7mm FF diversion were also plotted on the charts.

13.5. RESULTS AND DISCUSSION

13.5.1 VARYING DEMAND

The effect of setting water demand to 0.5 ×Average daily runoff (ADR) and 1.5 ×ADR, are shown in Figure 13.4 along performance for the default demand of 0.8 ×ADR. First-flush amounts of 2 mm and 7 mm are shown as squares and diamonds, respectively. Unsurprisingly, there is no effect on the inlet stream, however examination of the lines representing the efficiency at the tank outlet shows a strong sensitivity to water demand. Removal efficiency is unaffected, but the volumetric efficiency shows an improvement as demand on the system is reduced. Figure 13.5 shows the effect of changing demand on demand satisfaction on a system with different first-flush diversions.

Figure 13.4: Removal and volumetric efficiency of first-flush systems with varying demand based on Corozal data.

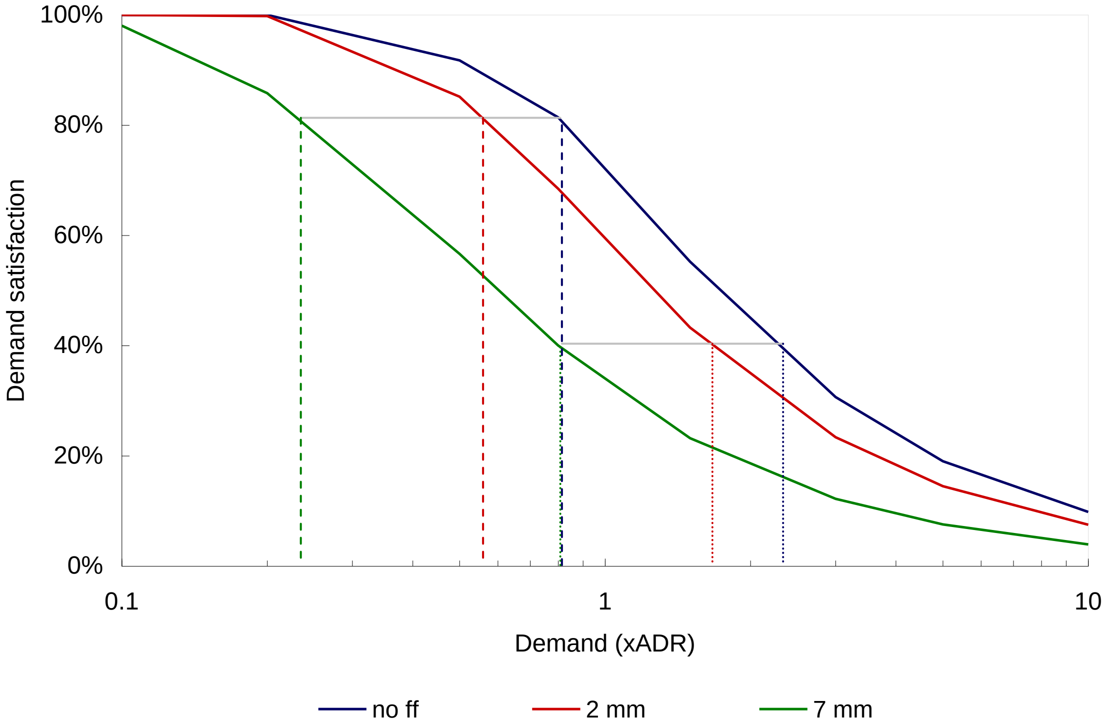


Tank size = 10 ×ADR, k_w = 0.7 mm^{-1} (50% contaminant reduction / mm rainfall), k_a = 0.0032 min^{-1} (99% reset in 1 day)

The effect of first-flush is actually to reduce runoff entering the tank, therefore when demand is expressed in terms of runoff, the demand with a first-flush is equivalent to a larger demand if all runoff were allowed to enter the tank. This can be seen in Figure 13.5 as the curves for first-flush are similar to that for no first-flush but are offset to the left (i.e. toward a smaller demand);

e.g. a demand of 0.8 ×ADR with a first-flush of 7mm, for example gives the same performance as if the demand were 1.8 ×ADR with a 3mm first-flush or 2.5 ×ADR with no first-flush. Similarly, for a system with a first-flush was required with an equivalent performance to a non-first-flush systems with a demand of 0.8 ×ADR, the demand on a systems with a 2mm first-flush would have to be 0.55 ×ADR and a system with a 7mm first-flush, 0.25 ×ADR. The simple (and fairly obvious) conclusion from this is that first-flush devices are most effective when there is water to spare – when rainfall is high, demand is low and/or roofs are large.

Figure 13.5: Effect of changing demand on water delivery with varying levels of first-flush based on Corozal data

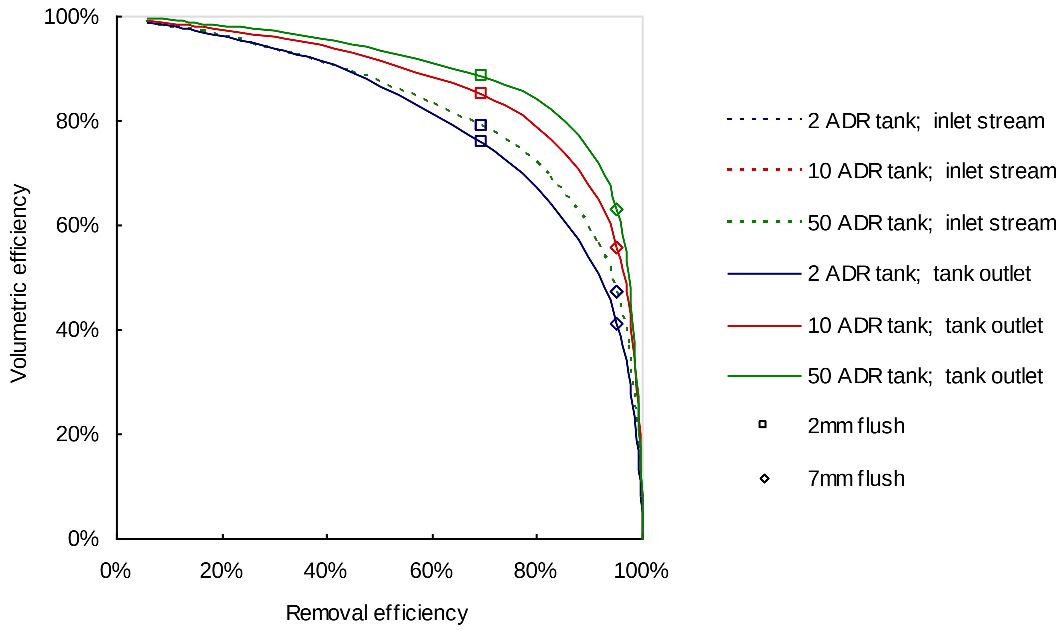


Tank size = 10 ×ADR, k_w = 0.7 mm^{-1} (50% contaminant reduction / mm rainfall), k_a = 0.0032 min^{-1} (99% reset in 1 day)

13.5.2 VARYING TANK SIZE

Varying the tank size provides a slightly surprising result. The initial supposition was that overflow water would be a driving influence on the volumetric efficiency of a first-flush system, i.e. if the tail end of a storm were to overflow the tank, removal of initial water would simply be equivalent. The simulation results show that the opposite is the case and in some cases, a small tank can result in a worse performance than the inlet stream.

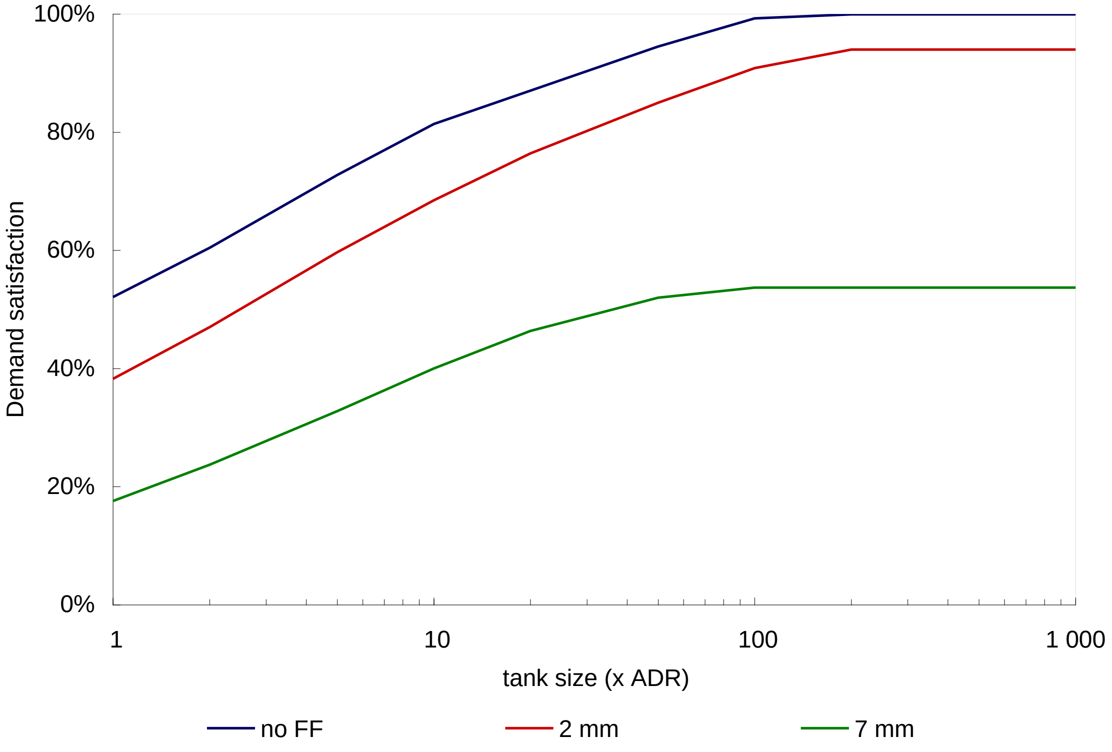
Figure 13.6: Removal and volumetric efficiency of first-flush systems with varying tank size based on Corozal data.



Demand = 0.8 ×ADR, k_w = 0.7 mm^{-1} (50% contaminant reduction / mm rainfall), k_a = 0.0032 min^{-1} (99% reset in 1 day)

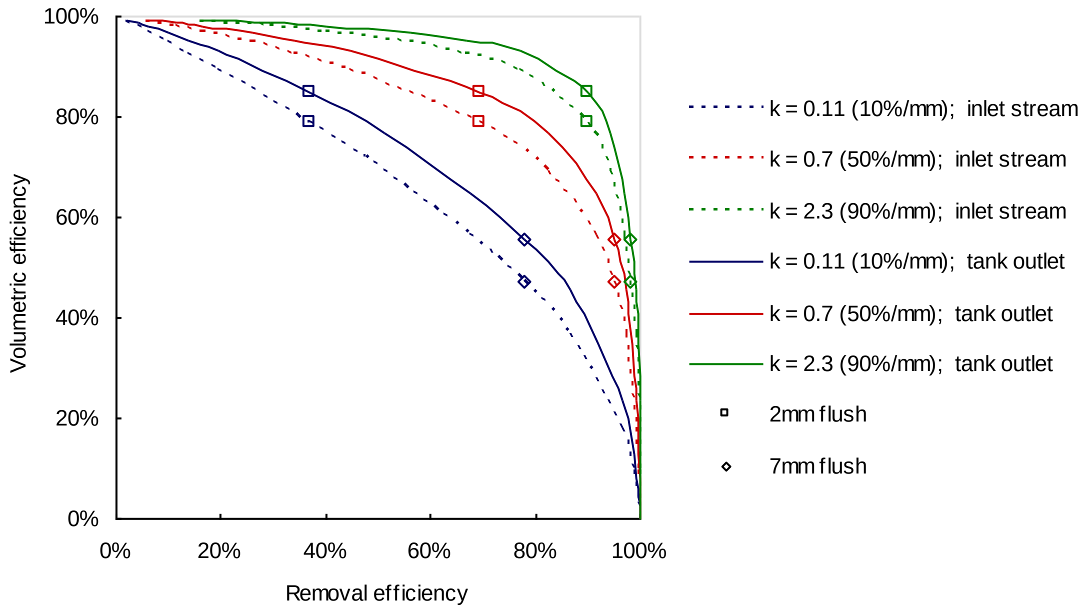
Like demand, changing tank size has no effect of the inlet characteristics or on the removal efficiency – all change takes place with the volumetric efficiency at the tank outlet. Figure 13.7 shows how tank size effects demand satisfaction. The curves for demand satisfaction for first-flush and non-first-flush systems are very similar but with first-flush systems offset downwards reflecting a lower demand satisfaction for a given tank size and a slightly varying rate of improvement in demand satisfaction with increased tank size. More important than the individual effects, though is their ratios. Consider a system with a 1 day ADR store. Without a first-flush it has a demand satisfaction of 52% and a system incorporating a 2mm first-flush has a satisfaction of 38%. The difference in water delivery of these systems is 14% of the demand and the ratio of FF to no-FF is 73%. When the tank is 10 days ADR, the no-FF system has a satisfaction of 81% and the 2mm FF system a satisfaction of 68% – a similar difference of 13%. However as both of the satisfactions have risen, the ratio of the two is now 84%, causing a greater volumetric efficiency for larger tanks. The increase continues until the loss of runoff to the first-flush system causes the system to roof limit or 100% demand satisfaction is reached. With particularly large tanks and small demands, it is therefore possible to have a first-flush system with 100% volumetric efficiency

Figure 13.7: Effect of tank size on demand satisfaction with varying levels of first-flush based on Corozal data



13.5.3 WASH-OFF RATES

Figure 13.8: Removal and volumetric efficiency of first-flush systems with varying wash-off rates, based on Corozal data.

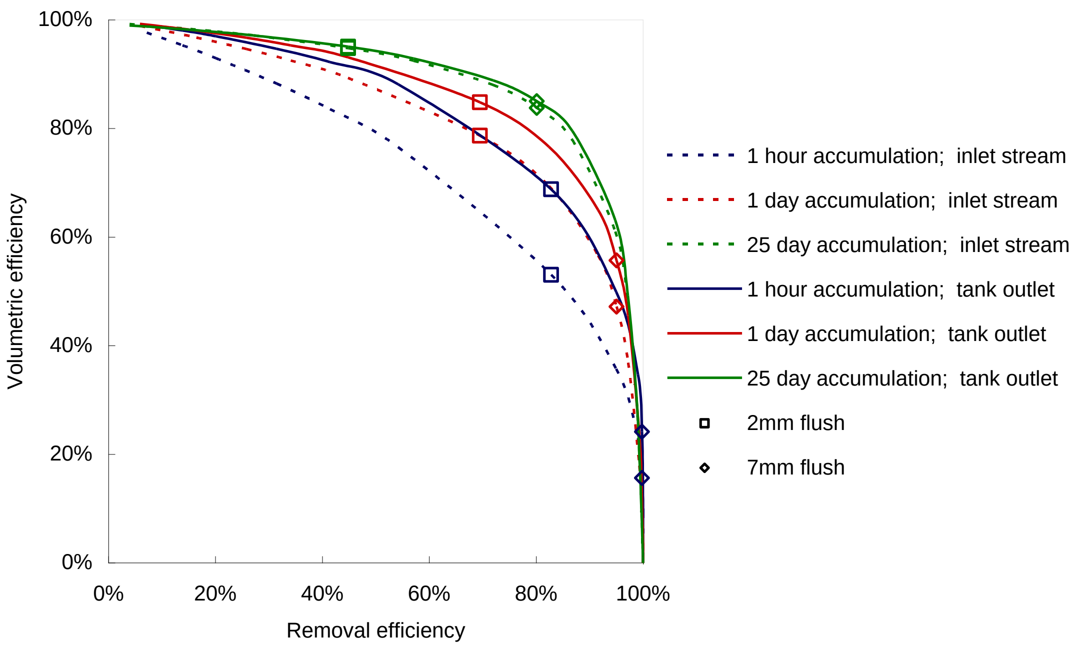


Tank = 10 x ADR, Demand = 0.8 ×ADR, k_a = 0.0032 min^{-1} (99% reset in 1 day)

Figure 13.8 shows the effect of wash-off rate. The wash-off rate does not affect the volumetric efficiency as the same amount of water will be removed for a given first-flush size, it does, however strongly affect how well the first-flush system removes contaminates. The default wash-off constant for the simulation is based on the conservative rule developed in chapter 12, i.e. halving the contaminate load for each millimetre of rainfall (k_w = 0.7 mm^{-1}). Actual washoff rates vary widely with a median value of 1.2 mm^{-1} so overall first-flush performance should be slightly better than shown in the graphs in this chapter.

13.5.4 ACCUMULATION

Figure 13.9: Removal and volumetric efficiency of first-flush systems with varying contaminant accumulation rates and identical device reset rates based on Corozal data.



Tank = 10 x ADR, Demand = 0.8 ×ADR, k_w = 0.7 mm^{-1} (50% contaminant reduction / mm rainfall)

Using the hypothesis that device reset should be matched to the material accumulation, varying these together affects both volumetric and removal efficiency. It also affects both the incoming stream and the tank outlet. As material accumulation times become greater, more must be diverted for equivalent material removal. The greater first-flush cause a greater equivalent demand as described in 13.5.1 resulting in a convergence between inlet and outlet curves. This convergence reduces the differences between the efficiency curves of different material accumulation rates when measured at the tank outlet. The convergence also means that a change in accumulation rate between 1 hour and 25 days only causes a change in efficiency of a few percent indicating that material accumulation in itself may not be a critical parameter. If this is true, it would be a benefit as, has been discussed in Section 12.5, this parameter has proved difficult to measure even with many field tests due to very noisy data.

Figure 13.10 shows the effect of using a device with a reset that is different from the accumulation rate. Generally, the penalty for not matching accumulation and reset rates is considerably smaller than expected and, in some cases mismatched rates where the reset is shorter than accumulation produce *better* results than “perfectly” matched systems, particularly at very high removal efficiencies. This effect is apparent at both inlet and outlet and never happens when the device reset is slower than the material accumulation.

Figure 13.10: Removal and volumetric efficiency of first-flush systems with varying contaminant accumulation rates, and non-matching reset rates, based on Corozal data.

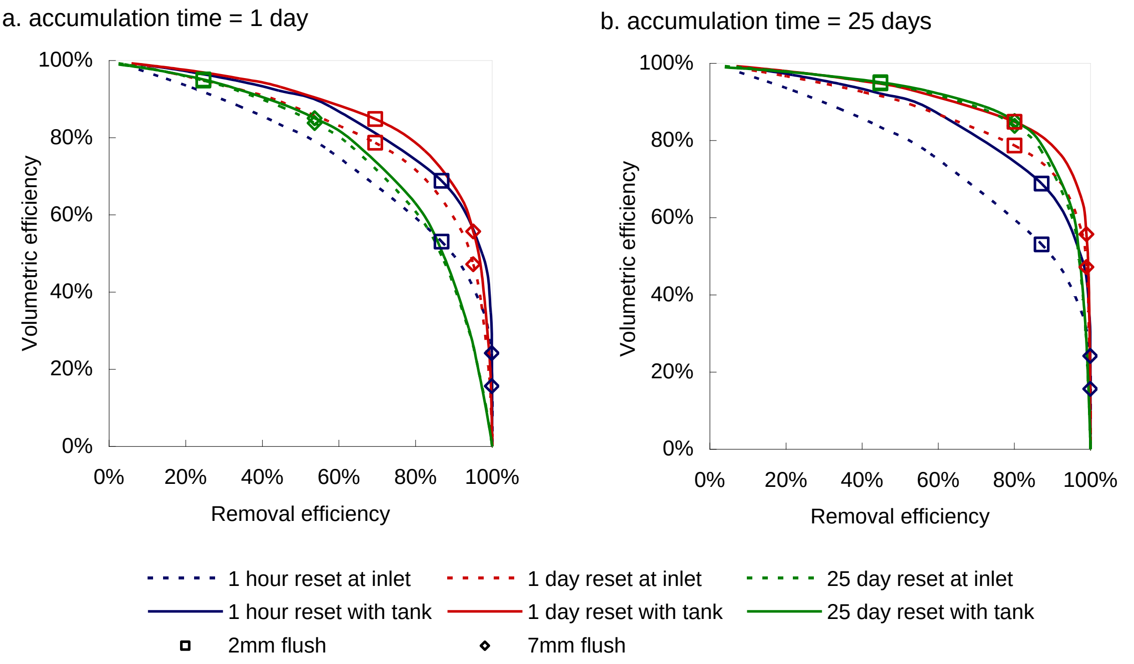
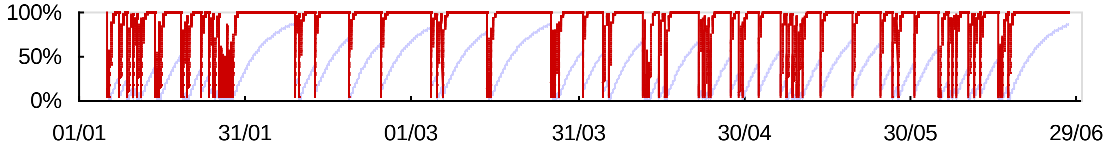


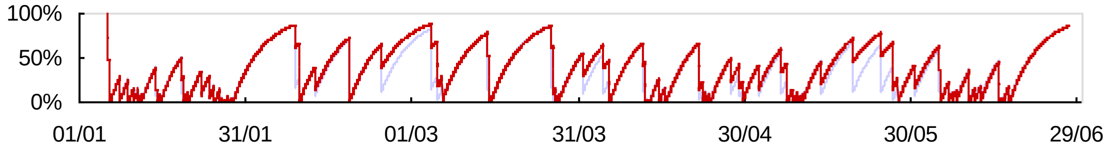
Figure 13.11 shows a time series of the dynamic behaviour of a several first-flush devices and that of the accumulated contaminants the device is intended to remove. The devices have been sized so that they all provide a removal efficiency of 90%, so the faster resetting device is smallest and the slowest reset, the largest. Devices with three reset times are shown; a fast reset time, where the reset is only 1 day, a matched reset where the reset is matched to the accumulation of 25 days and a slow reset where the reset is 250 days.

Figure 13.11: Device reset and material accumulation over 6 months based an accumulation time ($t_{a,99}$) of 25 days and devices with a removal efficiency of 90% using Corozal data

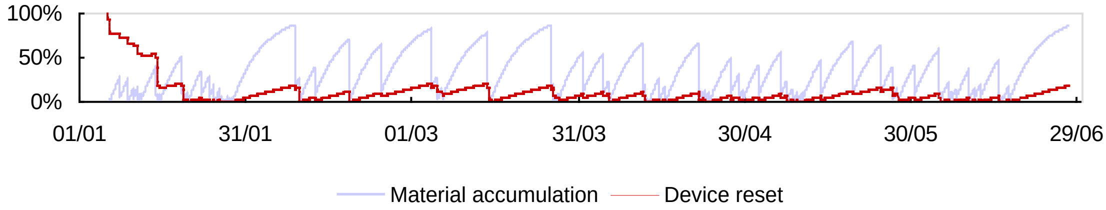
a. 1 day reset (ff_d = 2.4 mm, vol. efficiency = 67%)



b. 25 day reset (ff_d = 9.6mm vol. efficiency = 70%)



c. 250 day reset (ff_d = 64mm vol. efficiency = 62.5%)



When storms are spaced widely apart, behaviour is very similar between all devices, but several factors should be considered:

- Fast and matched devices are both nearly empty and can divert the full amount but the fast resetting device is smaller than the matched one so will divert a smaller amount – allowing dirtier water into the tank, in this case diverting 2.4mm rather than 9.6mm. increasing volumetric efficiency but reducing material removal

- The slow resetting device cannot completely reset and so can only divert a fraction of its total capacity. This fraction is unlikely to be equivalent to the capacity of a correct device. In this case the full capacity is 64mm and the device only ever resets to 20% of capacity so will flush 12.8mm, compared to the matched device's 9.6mm, reducing volumetric efficiency for a very slight gain in material removal.

When storms are close together the devices behave in very different ways:

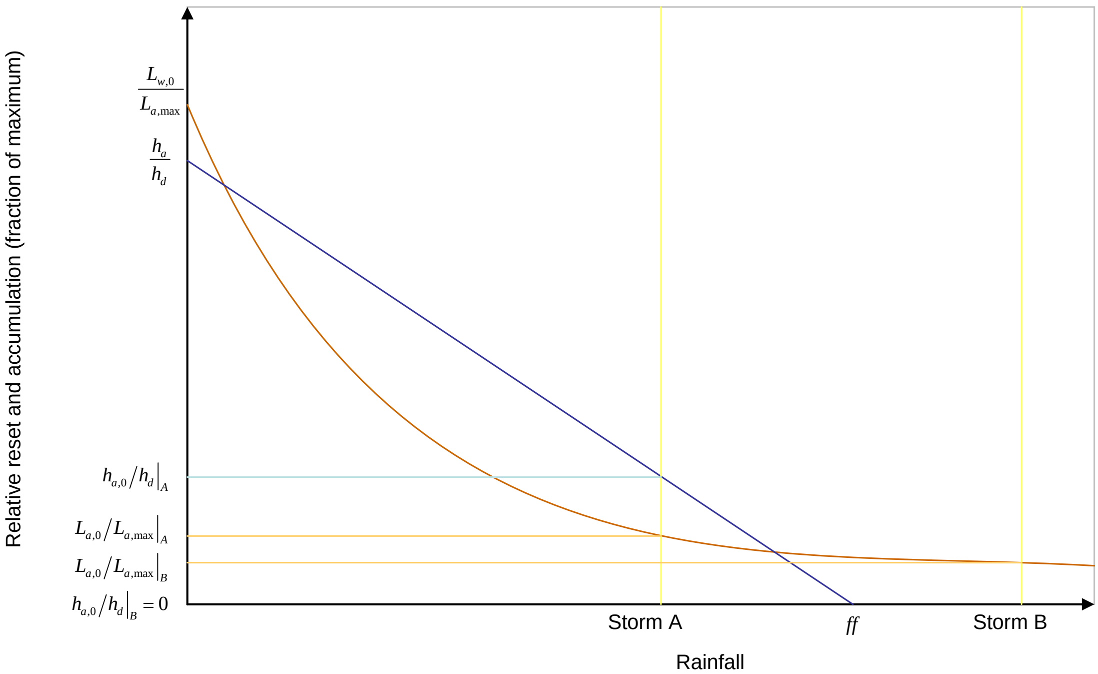
- The fast device diverts water from more storms than the other two devices resulting in higher material removal and lower volumetric efficiency – the reverse of long-spaced storms. A good example of this is around 25/1 when a number of storms come in quick succession (seen as washoff events on the material accumulation curve). The 1 day reset time system washes between 60% and 100% of its 2.4mm capacity for each of these storms where the 25 day reset time system only washes between 5-20% of its 9.6mm capacity.

- The slow-resetting system largely ignores these storms and diverts very little.

A storage tank adds to the complexity as it will selectively accept or overflow rain depending on how full it is; rain that falls after a long dry spell is most likely to be collected in the tank while rainstorms that fall in rapid succession may simply be diverted to overflow.

The changes in efficiency of different sized first-flush devices with different rainfall spacing goes some way to explaining the small difference between devices, it does not, however explain how it is possible for a device with a reset that does not match the accumulation to outperform a perfectly matched one. The likely answer lies in the initial water level before a device resets (h_0) – which is dependent on how much the device has filled during the preceding storm, and initial material level before accumulation ($L_{a,0}$) – which is dependent on material washoff behaviour during the preceding storm.

Figure 13.12 Material washoff and first-flush device filling over the course of a storm



The initial levels for both accumulation and device reset are determined by the washoff of contaminates during the storm and the filling of the first-flush device as shown in Figure 13.12. As the design diversion (ff_d) of a first-flush device is proportional to the design height and should be set to flush the maximum accumulation ($L_{a,max}$) the contaminant material level (L) is most conveniently expressed as a fraction of the maximum accumulation (the *contamination fraction* = $L/L_{a,max}$), and the available device height (h_a) as a fraction of the design height (h_d) (the *diversion fraction* = h_a/h_d),. The contamination fraction (shown as the red line in the graph) reduces exponentially as described by Equation 12.3 as material is washed off a roof. The diversion fraction (shown as the blue line), reduces linearly with accumulated rainfall as the device fills with the water from the roof. These curves can only cross at a maximum of two rainfalls – for all other rainfalls there will be some discrepancy between the contamination and diversion fractions and therefore the initial load for accumulation and the initial level for device reset. This discrepancy can clearly be seen on several occasions in Figure 13.11b most prominently from 20/2 to 7/3 and 10/5 to 30/5, although smaller discrepancies can be seen throughout. Where the points of intersection are located depends on the washoff rate, the material accumulation before the storm and the available FF diversion before the storm.

The first-flush device will, most likely have a capacity smaller than that need to completely wash the roof so there will be many storms where the rainfall will be larger than the available FF diversion (*ff*), and will end with the device having no remaining capacity ($h_{a,0}$ = 0) and the roof with unwashed material ($L_{a,0}$ > 0), so $L_{a,0}/L_{max} > h_0/h_d$. This situation is shown as "Storm B" in Figure 13.12. As these are the occasions when water is delivered to the tank they will be frequent.

For storms where the rainfall is less then the available FF diversion, the first-flush device only partially fills and therefore passes no water to the tank as shown in Figure 13.12 as "Storm A". The device will reset from some arbitrary point between zero and the design diversion. For a great deal of this range $L_{a,0}/L_{max} < h_0/h_d$, but the reverse may be true for very small storms if the diversion fraction is lower than the contamination fraction at the beginning of the storm. When reset and accumulation rates are matched and the previous storm ended with a full FF device, the reset will start from zero and so the diversion fraction is certain to be lower than the contamination fraction for the subsequent dry period. $L_{a,0}/L_{max}$ will also be higher than h_0/h_d toward the end of the first-flush diversion.

When the discrepancy between device reset and accumulation is plotted as shown in Figure 13.13, the relative effects of divergences in initial conditions are shown.

Figure 13.13: Relative discrepancy of contamination fraction compared to diversion fraction

Periods where the diversion fraction is less than the contamination fraction dominate as would be expected by the frequency of occasions when the FF device is allowing water to pass. These discrepancies are, however associated mainly with frequent rainstorms and therefore are in effect for only a short time. Periods where the device reset is greater than the accumulation are less frequent but are associated with less frequent rain. The long time period of these divergences make them very visible in Figure 13.11b, however, as these discrepancies occur when little or no water is being delivered to the tank they are, in fact relatively unimportant. A truly optimum solution would balance under and over resetting so that the overall average reset is in-line with the overall average accumulation. The presence of a tank means that the balance should be based on contribution to tank water accredation rather than just to water flow.

13.6. EMPIRICAL SIZING RULES

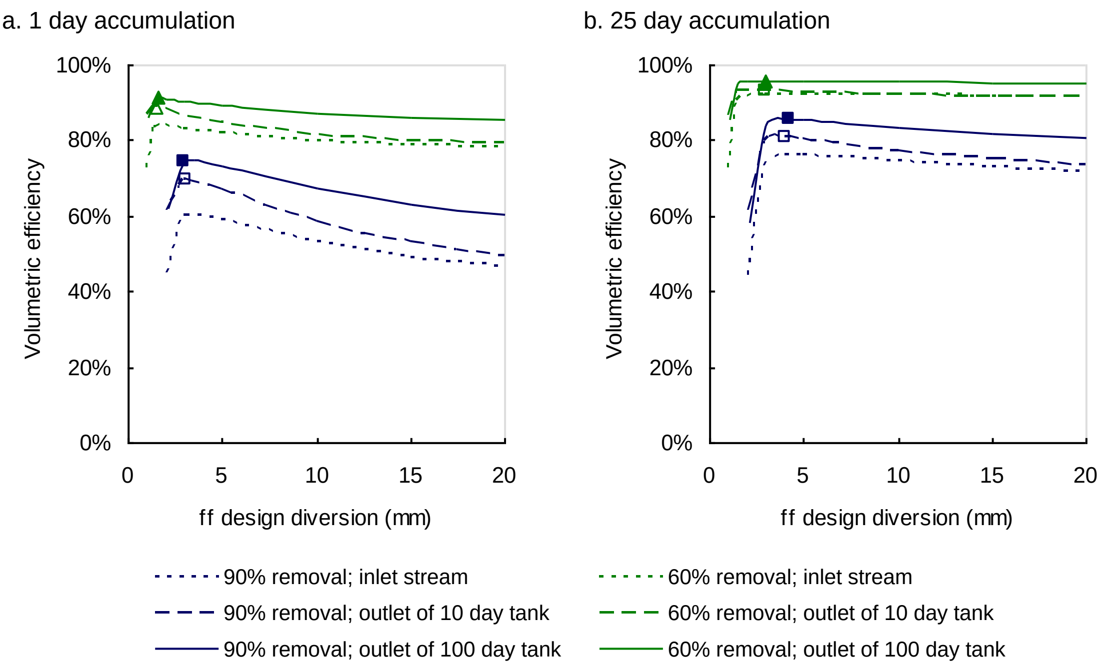
13.6.1 FIRST-FLUSH DESIGN DIVERSION

By holding the target removal efficiency constant and varying first-flush design diversion, the optimum design diversion for that removal efficiency becomes clear. This is shown in The optimum first flush diversion changes considerably with target removal efficiency and accumulation time with higher values for greater material removal and slower accumulation; and slightly with the presence or absence of a tank and very slightly with tank size. The graphs also rise very quickly to the optimum but fall very slowly; so the penalty for under-sizing a first-flush system is high whereas over-sizing the system has a relatively small penalty.

Figure 13.14 which plots the effect on volumetric efficiency resulting from a number of mass balance simulations where reset time is varied until a target removal efficiency is achieved for a certain FF diversion. The optimum FF diversion, based on maximising volumetric efficiency for the chosen removal efficiency was also iteratively calculated using Excel’s solver function.

The optimum first flush diversion changes considerably with target removal efficiency and accumulation time with higher values for greater material removal and slower accumulation; and slightly with the presence or absence of a tank and very slightly with tank size. The graphs also rise very quickly to the optimum but fall very slowly; so the penalty for under-sizing a first-flush system is high whereas over-sizing the system has a relatively small penalty.

Figure 13.14: Volumetric efficiency of different first-flush devices with constant removal efficiency

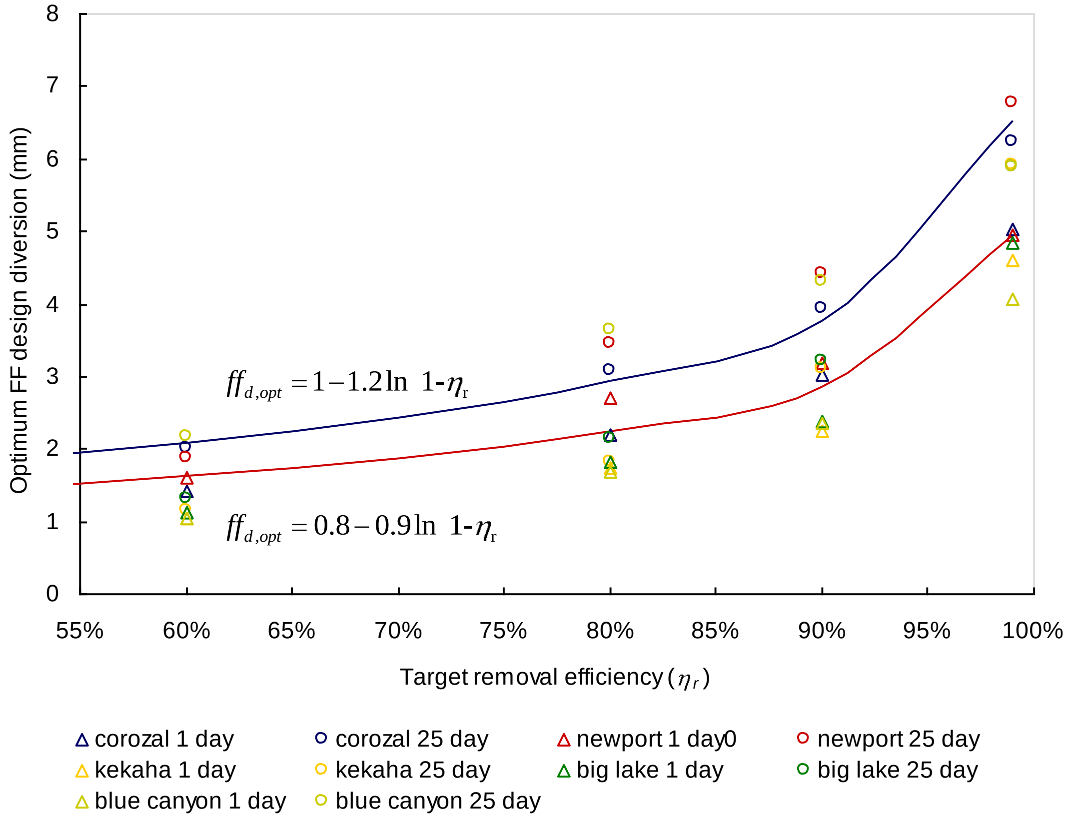


Only two target removal efficiencies are shown in The optimum first flush diversion changes considerably with target removal efficiency and accumulation time with higher values for greater material removal and slower accumulation; and slightly with the presence or absence of a tank and very slightly with tank size. The graphs also rise very quickly to the optimum but fall very slowly; so the penalty for under-sizing a first-flush system is high whereas over-sizing the system has a relatively small penalty.

Figure 13.14, but simulations were carried out for 6 different target removal efficiencies (20%, 40%, 60%, 80%, 90% & 99%). When the optimal FF design diversions ($ff_{d,opt}$) are plotted against target removal efficiency, they fall onto a curve. Logically this curve should pass through the origin as when no water is flushed, no material will be removed. It should also be asymptotic to 100% as flushing all the water will be needed to completely clean the roof. An exponential function of the form $ff_{d,opt} = -k \ln 1\text{-}\eta_{\mathrm{r}}$ is a good candidate for this and the data fits roughly onto such a function when accumulation time is short, however the longer accumulation times do not appear to fit the simple function. A better fit at higher removal efficiencies can be found if the assumption of the curve passing through the origin is abandoned. As it is very unlikely that anyone would want to design a first-flush system with very low material removal this may be justified to gain a better fit in the area of interest.

Figure 13.15 shows the optimum first-flush diversion from several sites with 1 and 25 day reset times. The chart has been cropped at a target removal efficiency of 55%.

Figure 13.15: Calculation of optimum first-flush design diversion for a target collection efficiencies for accumulation times of 1 and 25 days in 5 locations



Two functions of the form $ff_{d,opt} = \beta - k\ln\ 1\text{-}\eta_r$ (where β and k are constants with the units of mm) have been plotted over the simulation results and show a reasonable fit through the data. The lines correspond to:

$$ff_{d,opt} = 0.8 - 0.9\ln\ 1\text{-}\eta_r \qquad \text{Equation 13.20}$$

For a 1 day accumulation time, and:

$$ff_{d,opt} = 1 - 1.2\ln\ 1\text{-}\eta_r \qquad \text{Equation 13.21}$$

for a 25 day accumulation time.

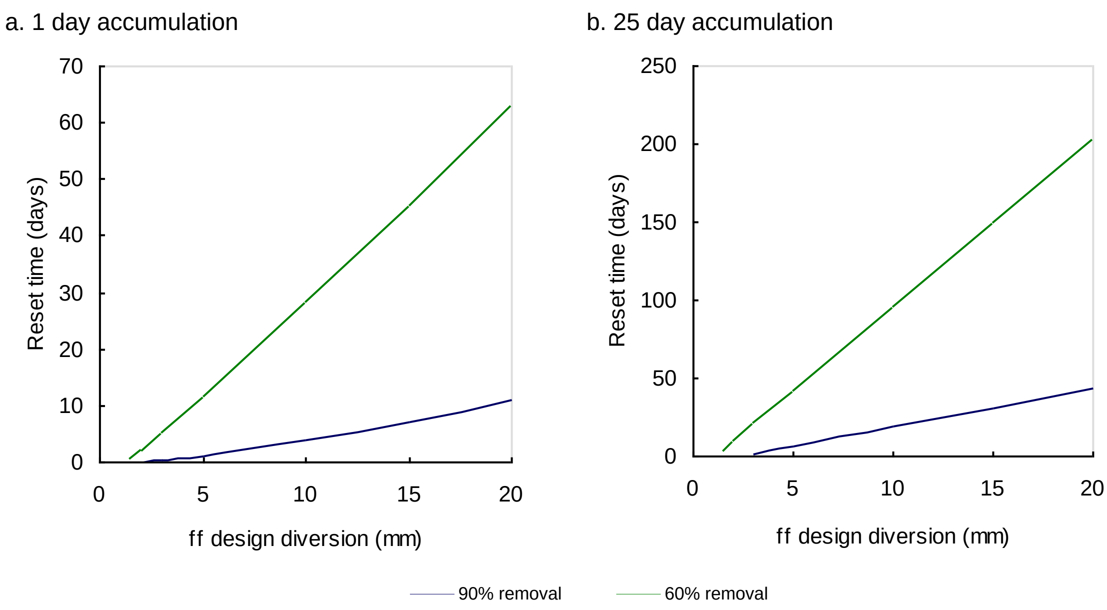
Equation 13.20 and Equation 13.21 can be used to give a reasonable approximation of the optimum first-flush diversion to for any chosen removal efficiency across a wide range of rainfall patterns, dependent only upon the accumulation time. The fit is imperfect and the data is fairly scattered, however it should be noted that when over-sizing is adopted the penalty is small. The two lines are also close enough together that if the accumulation time is evaluated incorrectly, the likely error will only be a fraction of a millimetre – again erring on the large side should ensure a small penalty in terms of volumetric efficiency.

13.6.2 RESET TIME

Having calculated the first-flush design diversion, the corresponding reset time needs to be determined. When the reset times corresponding to the first-flush design diversions shown in The optimum first flush diversion changes considerably with target removal efficiency and accumulation time with higher values for greater material removal and slower accumulation; and slightly with the presence or absence of a tank and very slightly with tank size. The graphs also rise very quickly to the optimum but fall very slowly; so the penalty for under-sizing a first-flush system is high whereas over-sizing the system has a relatively small penalty.

Figure 13.14 are plotted against their corresponding design diversion, the result is a series of near straight lines as shown in Figure 13.16.

Figure 13.16: Reset times and first-flush amounts based on Corozal data



Like in The optimum first flush diversion changes considerably with target removal efficiency and accumulation time with higher values for greater material removal and slower accumulation; and slightly with the presence or absence of a tank and very slightly with tank size. The graphs also rise very quickly to the optimum but fall very slowly; so the penalty for under-sizing a first-flush system is high whereas over-sizing the system has a relatively small penalty.

Figure 13.14, only two lines, corresponding to two removal efficiencies are shown, but similar lines were found for all 6 efficiencies simulated. The equation describing these lines varies with target removal efficiency and the accumulation time but in a way that can be used to derive a general equation for reset time for any design diversion.

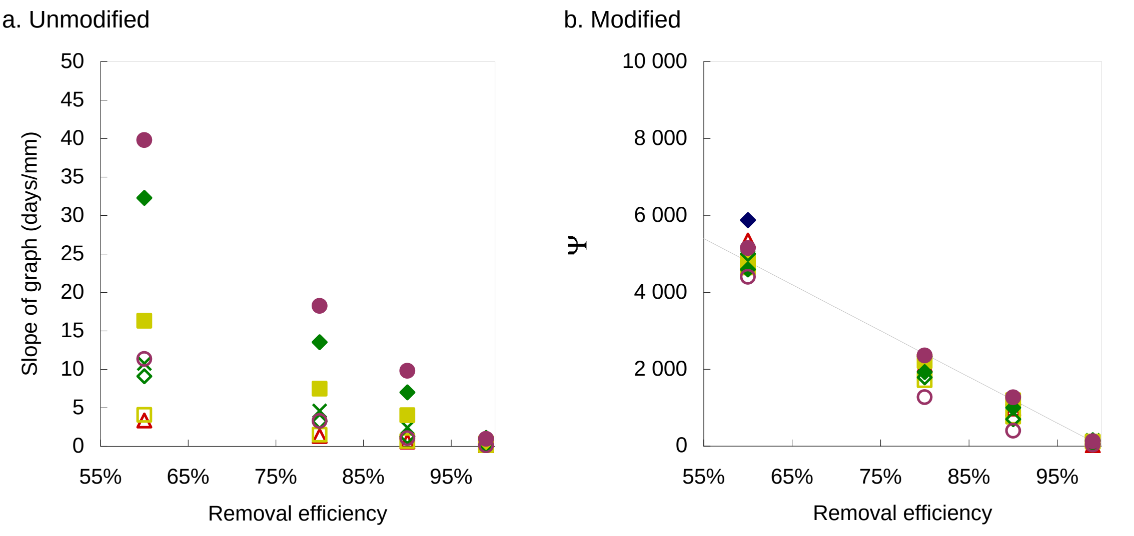
Starting with the slope, when the gradients of the reset-time/ff-diversion lines are plotted against removal efficiency for the five locations simulated, they fall onto a series of curves which can be made to collapse onto a single line when Equation 13.22 is applied:

$$\Psi = \frac{t_r}{ff_d} \times \frac{r_a}{t_a^{0.14}} \qquad \text{Equation 13.22}$$

Where: Ψ is the modified gradient, t_r is the reset time (days); ff_d is the first-flush design diversion (mm); r_a is the annual rainfall (mm) and t_a is the accumulation time (days).

The gradient and modified gradients for the simulated data are plotted against removal efficiency in Figure 13.17. Like Figure 13.15, the plots are truncated at 55% removal efficiency to focus attention on the performance band likely to be required.

Figure 13.17: Slopes of graphs of first-flush design diversions vs. reset times in 5 locations



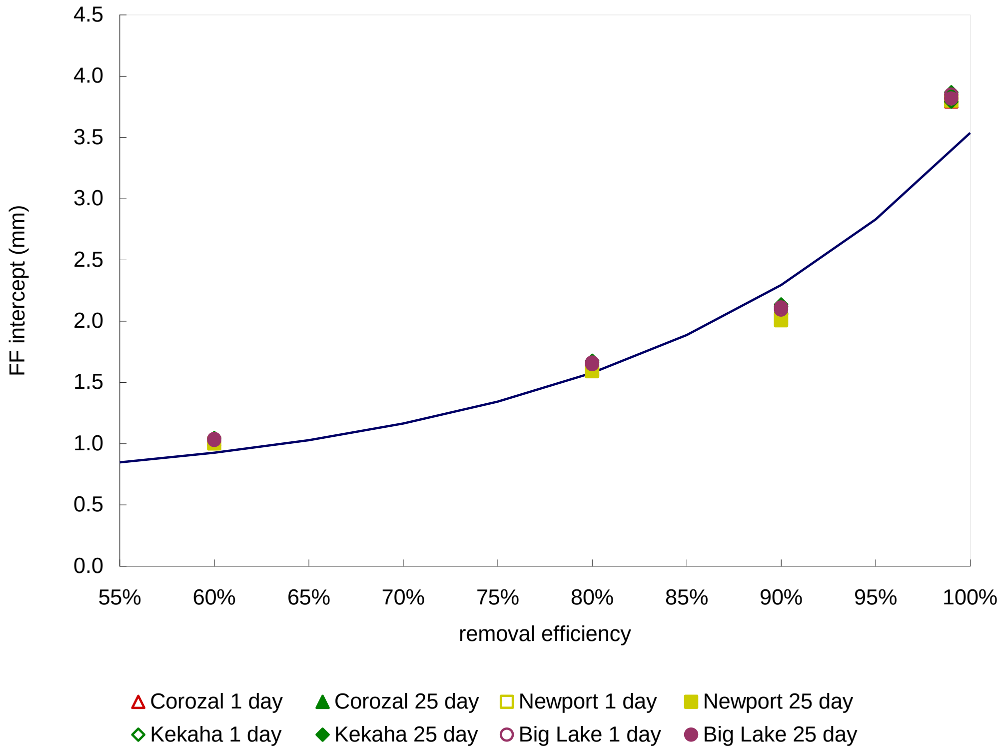
The curve in Figure 13.17b is nominally linear when removal efficiency is larger than 60%. The line corresponds to the equation:

$$\frac{t_r}{ff_d} = -12000\eta_r + 12000\,\frac{t_a^{\,0.14}}{r_a} \qquad \text{Equation 13.23}$$

A better fit could have been made with a curve, however considering the uncertainties in parameters in the simulations, noise in the FF design diversion selection and likely uncertainty in measurement of accumulation the simplified linear fit was seen as sufficient.

Turning attention to the intercept, Investigation of Figure 13.16 shows the offset along the design diversion axis[1] is stable regardless of accumulation time but varies depending on removal efficiency. This is true for all removal efficiencies simulated and for all locations. However, as the intercept on the reset-time axis is required for a formula of the form $t_r = m\,ff_d + C$, this will depend upon the slope determined by Equation 13.23. When plotted against removal efficiency as shown in Figure13.18, the offset along the design diversion axis shows a remarkable consistency.

Figure 13.18: Relationship between offset along the design diversion axis and removal efficiency



[1] This offset actually corresponds to an accumulation time of 15 minutes which is the smallest time step available with the data. However, considering the scale of reset time is in "days" this can be considered close to zero.

An exponential curve can be fitted to the results:

$$ff_{d,0} = 0.6 + 0.012e^{5.5\eta_r} \qquad \text{Equation 13.24}$$

Where $ff_{d,0}$ is the intercept on the design diversion axis and the constants 0.6 and 0.012 have the dimensions of mm.

The offset on the design diversion axis can simply be multiplied by the negative of the gradient to obtain the intercept on the reset time axis:

$$t_{r,0} = -\left(-1200\eta_r + 1200 \frac{t_a^{0.14}}{r_a} \right) 0.6 + 0.012e^{5.5\eta_r} \qquad \text{Equation 13.25}$$

Where $t_{r,0}$ is the intercept on the reset time axis.

This intercept is negative and can have no real physical significance in itself other than allowing a complete equation to be developed.

Finally, Equation 13.23 and Equation 13.25 can be combined to form an equation that gives a good approximation for the reset time required to produce a particular removal efficiency for any first-flush design diversion (in mm), given the accumulation time (in days) and the annual rainfall (in mm) for any location:

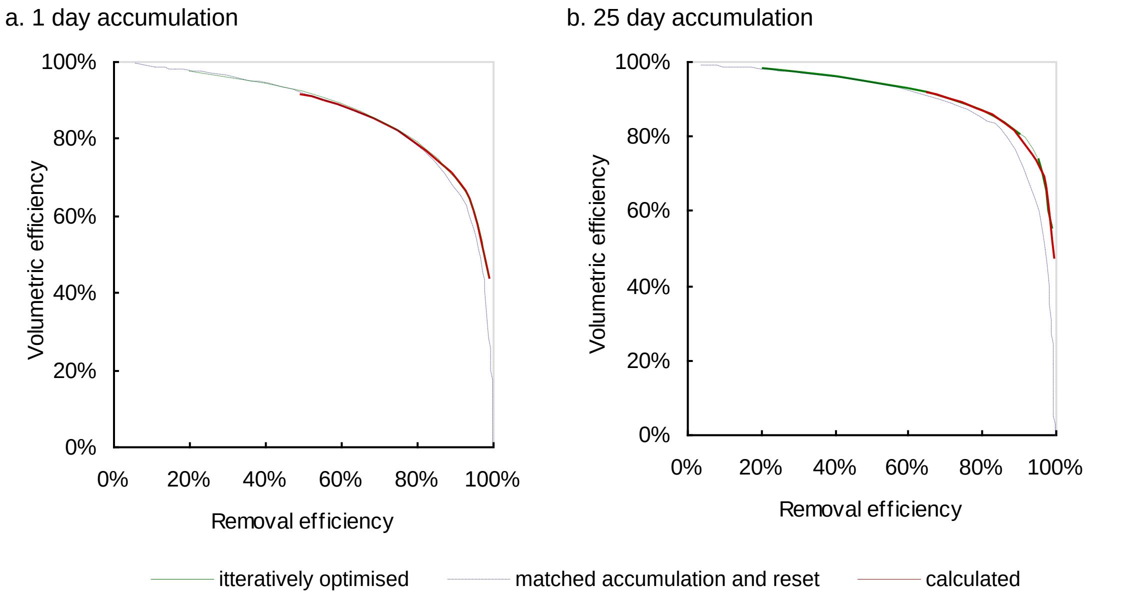
$$t_r = \left(-12000\eta_r + 12000 \frac{t_a^{0.14}}{r_a} \right) ff_d - 0.6 + 0.012e^{5.5\eta_r} \qquad \text{Equation 13.26}$$

As the intercept is negative, it is possible to find a negative reset time if a small enough design diversion is selected. If Equation 13.26 results in a negative reset time, the selected first-flush design diversion is simply too small to have a useful effect and will either result in a system with a very poor volumetric efficiency or may not even achieve the target removal efficiency.

13.6.3 VERIFICATION

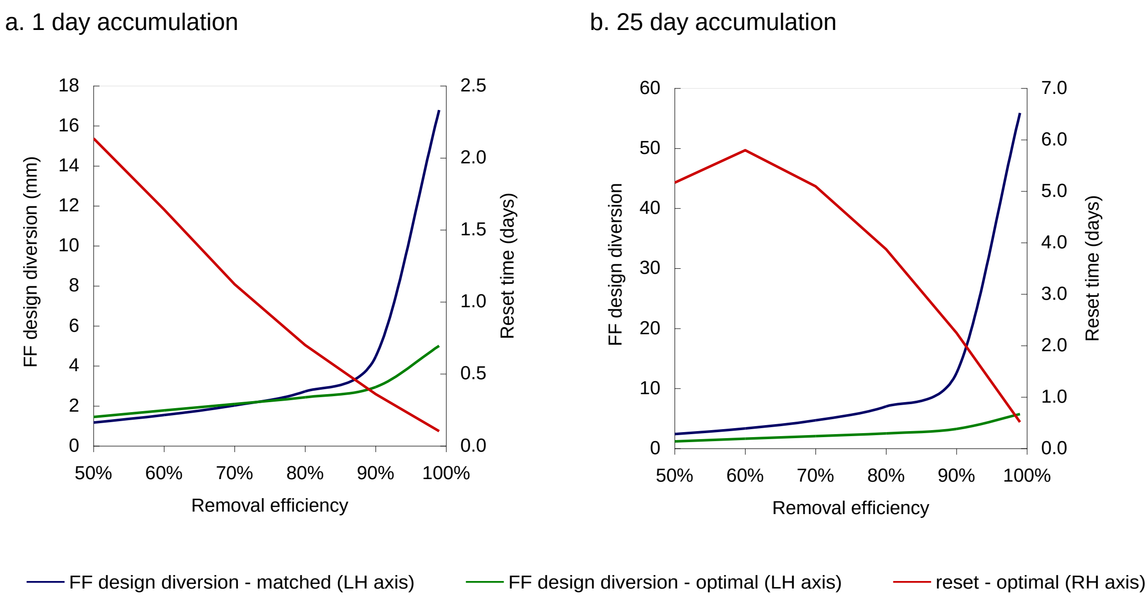
Applying Equation 13.20 and Equation 13.21 for optimum fist-flush diversion and Equation 13.26 for reset time to Corozal data results in the graphs shown in Figure 13.19.

Figure 13.19: Removal and volumetric efficiencies of optimum, calculated and "matched" first-flush devices based on Corozal data



The equations do not perfectly predict the optima derived through iterative modelling but they are very close and consistently demonstrate a better performance than by matching accumulation and reset rates. This is particularly true where accumulation is slow which is likely considering the field results described in Section 12.5. Similar results were found with all five locations and the resulting charts are in Appendix F.

Figure 13.20: Removal and volumetric efficiencies of optimum, calculated and matched first-flush devices



Note: reset time for matched device is not shown as it is, by definition, the same as accumulation

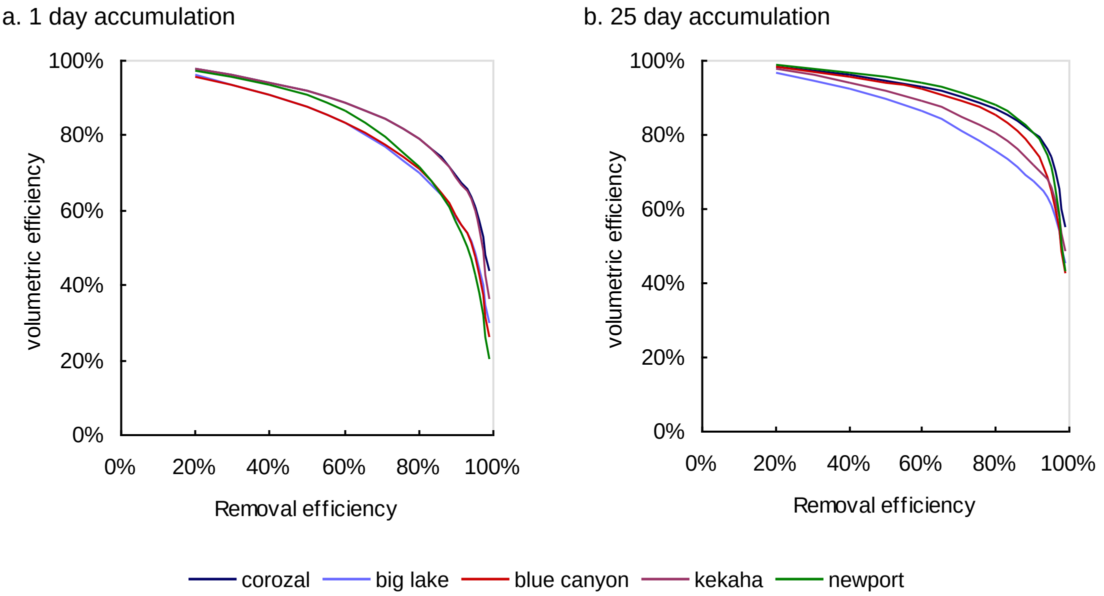
A further benefit of optimising using Equation 13.20 or Equation 13.21, and Equation 13.26 is that, for the most part, first-flush devices can be made smaller and reset times faster for the same performance. Figure 13.20 shows the first-flush design diversions and reset times needed for first-flush systems with matched accumulation and reset and those optimised by calculation. The optimised devices are usually smaller and, for the most part have shorter reset times. This is again particularly true at higher removal efficiencies and longer accumulation times.

13.6.4 VOLUMETRIC AND REMOVAL EFFICIENCIES OF OPTIMUM DESIGNS

The volumetric vs. removal efficiency of first-flush systems with optimal FF amounts and reset times is shown in Figure 13.21. These figures should be seen as fairly conservative as they reflect small 10 day tanks and a removal constant of 0.7 (as opposed to the median removal constant for turbidity of 1.12).

Climate is not as great a limitation as might be expected, with most climates performance within a 10% band except at very high volumetric efficiencies and fast accumulation. Overall, rainfall quantity is a greater delimiter than rainfall pattern, particularly at higher accumulation rates. The reason for this is that in low-rainfall areas, rain also falls less often, allowing first-flush devices to completely empty and therefore divert large amounts of the scarce rain.

Figure 13.21: Volumetric and removal efficiencies of optimised designs with 1 and 25 day accumulation in 5 locations

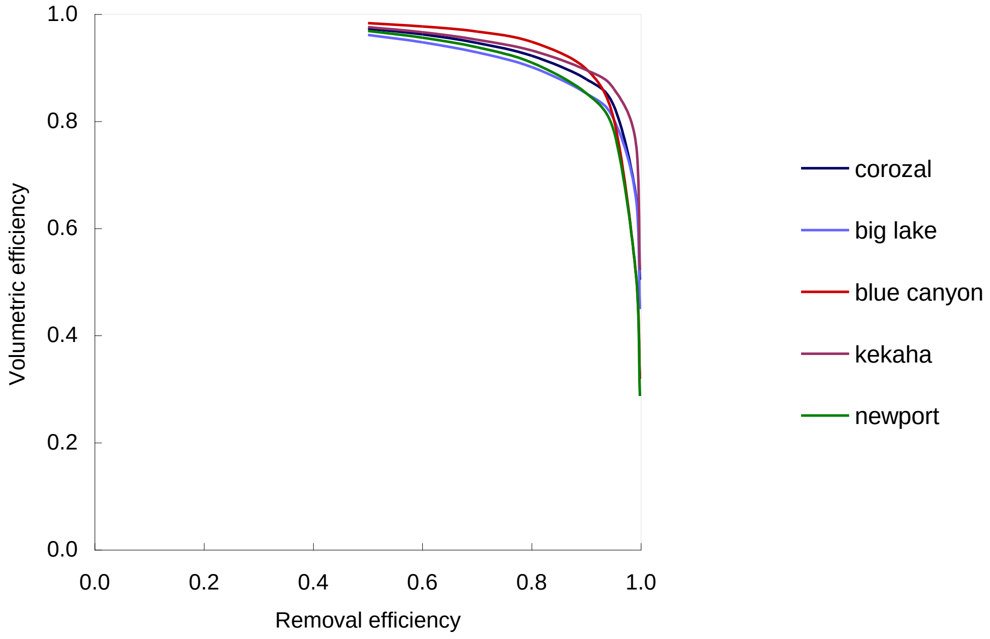


The differences are somewhat ameliorated by the fact that large tanks are necessary in areas with low rainfall for reasonable volumetric performance. As the performance of FF systems is improved with larger tank size, these tanks will also aid the volumetric efficiency of the first-flush system. When tanks are sized to provide a certain performance, the differences in location are much reduced. This is clearly seen in Table 13.2 and Figure 13.22 which show the performance of first-flush systems under a fairly typical situation where the tank has been designed to provide 80% of the household water demand.

Table 13.2: Tabulation of volumetric efficiencies of first-flush systems with a typical scenario

	Removal efficiency	
	50%	90%
	Volumetric efficiency	
Corozal	98%	90%
Big lake	97%	88%
Blue canyon	96%	85%
Kekaha	97%	85%
Newport	98%	90%

Figure 13.22: Performance of first flush systems with a typical scenario



Scenario: 25 day material accumulation, 0.8 x ADR demand and tank sized to provide 80% of demand

13.7. CONCLUSIONS

A mass balance approach demonstrates a number of aspects of first-flush systems. The effectiveness of first-flush systems are sensitive to wash-off rates, water demand, and tank size but less sensitive to accumulation rate. The accumulation rate is, however an important design parameter as changes in accumulation rate affect the required FF amount and reset time.

Matching the reset time to the accumulation rate, while intuitive, does not guarantee the best balance between removal and volumetric efficiencies and results in over-large devices with technically challenging slow reset rates.

Optimal first-flush diversions were found to be insensitive to rainfall amount and pattern, and to tank size so a simple empirical equations describing optimal first-flush amount was developed based on a number of simulations involving five locations with different climates: The equations are:

$$ff_{d,opt} = 1 - 1.2\ln\ 1\text{-}\eta_r \quad \text{for normal (25 day) accumulation}$$

$$ff_{d,opt} = 0.8 - 0.9\ln\ 1\text{-}\eta_r \quad \text{for fast (1 day) accumulation}$$

Where *ffd* is the design first-flush diversion (mm); and η_r is the target removal efficiency.

Optimal reset time was sensitive to overall rainfall amount but not to rainfall pattern, so a single equation suffices to describe the reset time corresponding to a particular design diversion and target removal efficiency:

$$t_r = \left(-12000\eta_r + 12000\ \frac{t_a^{\ 0.14}}{r_a} \right)\ ff_d -\ 0.6 + 0.012e^{\ 5.5\eta_r}$$

Where:, t_r is the reset time (days); r_a is the annual rainfall (mm) and t_a is the accumulation time (days).

In the field, the long calculations may beyond the appropriate personnel so a simpler means may be needed. The sensitivity of reset-time to annual rainfall precludes simple tabulation of results but it is possible to use a chart such as that shown in

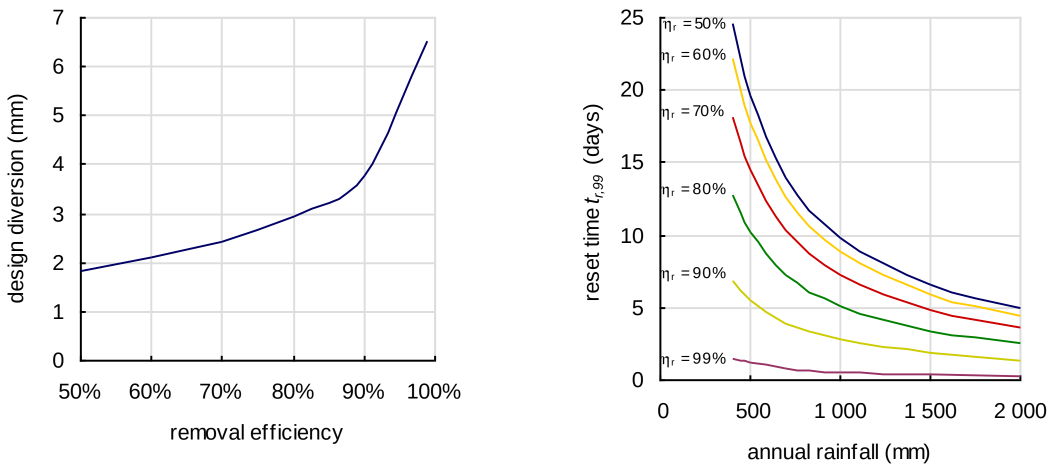
To use the charts or equations, a user decides on a desired removal efficiency. From this efficiency the required ff design diversion is calculated or looked up on the chart. Having obtained the design diversion, the size of the device to divert the full amount of the ff diversion (in litres) can be calculated by multiplying the diversion by the roof area (in m^2). The reset time is then calculated either using the equation or by looking along the line corresponding to the desired removal efficiency until the annual rainfall is reached and then reading off the reset time.

Figure 13.23.

To use the charts or equations, a user decides on a desired removal efficiency. From this efficiency the required ff design diversion is calculated or looked up on the chart. Having obtained the design diversion, the size of the device to divert the full amount of the ff diversion (in litres) can be calculated by multiplying the diversion by the roof area (in m^2). The reset time is then calculated either using the equation or by looking along the line corresponding to the desired removal efficiency until the annual rainfall is reached and then reading off the reset time.

Figure 13.23: Graphical representation of optimum first flush diversion and reset time based on desired removal efficiency (η_r) and annual rainfall

a. Optimum first-flush diversion — b. Reset time



The equations and charts could also be used to determine the potential removal efficiency of a known design diversion by making the diversion the subject of Equation 13.21 and then applying the reset time corresponding to the removal efficiency. The graphs can be used in a similar way by looking up the removal efficiency corresponding to the known diversion on Figure 13.23a and applying this efficiency in Figure 13.23b to determine the reset. This method will result in the best compromise between volumetric and removal efficiency, however other combinations are possible – resolving these will require the use of a mass balance model.

This method produces devices that have a better performance than systems where reset is matched to accumulation and results in smaller devices with faster reset times which are, in consequence easier and cheaper to make.

Optimised first-flush devices have reset times between a few hours and several days. These scales of reset time are technically possible only through the use of porous materials. Weep-holes will either be too big and empty much to quickly or will be liable to blockage.

Under normal working conditions optimally-designed first-flush devices can be expected to have volumetric and removal efficiencies above 85% and under favourable conditions higher than 90%.

14. CONCLUSIONS

14.1. Primary conclusions

14.1.1 TECHNOLOGY

Quality reduction can result in cost reductions on current best practice of up to 50% and up to 70% if only cash costs are considered. Minimum criteria are met and some desirable criteria can also be accommodated – though with some cost penalty. Careful design can reduce longevity problems or reduce their impact. Several designs were tested using this principle demonstrating both initial cost reduction and cost reduction over a period of 20 years when routine maintenance is included. For some designs the 20-year cost reduction was achieved without discounting the future, whereas some designs a discount rate had to be introduced to show a long-term cost reduction.

The discount rate required for a design to be more cost-effective overall than existing designs was termed the "crossover rate" and its existence goes some way to explaining why outside agencies often make different decisions to the people they serve. Agencies use NPV analysis and rational economics with discount rates under 10% whereas householders use subjective judgement with much higher implicit discount rates and may also be reliant on micro-credit which has high interest rates.

Underground tanks were found to have potential reductions of more than 50% in most cases and over 70% in a significant fraction of cases when compared to above-ground designs of the same volume and diameter. However the savings were not, as expected, due to the effect of soil support which in "medium" soil could only reduce the load in a cementicious tank by about

10% but by the ability of soil to provide distributed support which allowed underground designs to

- use more material efficient shapes
- use shapes which reduce bending moments
- be built without an expensive, solid foundation
- remove the need for some classes of material such as mesh in ferrocement designs and allow other classes of material such as thin plastics

14.1.2 HEALTH

Microbiological dangers from roofwater were found not to be due to exposure to human faecal material but animal sourced bacteria, opportunistic environmental bacteria or accidental contamination. As a result of the complex pathways and unusual pathogens, it is unlikely that faecal indicator bacteria will give results of public health significance – particularly if used in monitoring rainwater systems for *overall* water quality. Indicator bacteria can, however show *changes* in contamination within the roofwater harvesting system.

The path for contamination to enter the system was investigated and found to be complex and largely hostile route for entry of any foreign material. Reduction processes exist along the path followed by the rainwater so that chemical and microbiological contamination are considerably reduced at the tank outlet as compared to roof runoff. These processes can be assisted by design.

Inlet treatment can greatly reduce the contamination in a tank. Course filtering is effective, however fine filtering will suffer from trade-offs of volumetric and removal efficiency which may make effective filtering expensive. First-flush is more promising as it is insensitive to particle size and also effective against dissolved matter. Under normal working conditions optimally-designed first-flush devices can be expected to have volumetric and removal efficiencies above 85% and under favourable conditions higher than 90%.

The first-flush effect from roof catchments is well described by the exponential washoff equation used in stormwater management; $L = L_0 e^{-k_w r}$ where L is the contaminate load after rainfall r has fallen, L_0 is the contaminate load at the start of the rainfall and k_w is a coefficient. For roofs, the decay coefficient k_w is fairly widely distributed but insensitive to design variables such as roof type and slope and to location. A k_w value of 0.7 mm^{-1} is exceeded in 85% of storms for suspended matter and (based on initial estimates) for bacteria and disolved matter in 50% of storms. Using this washoff coefficient leads to a simple to remember and communicate rule-of-thumb:

"For each mm of first flush, the contaminate load will halve"

Applying this rule along with the observed accumulation rate to mass balance models of several locations with varying rainfall patterns showed the performance of first-flush systems are sensitive to water demand, and tank size. Varying the washoff and accumulation showed that the performance is affected by washoff rate and less so by accumulation.

Optimal first-flush diversions were found to be insensitive to rainfall depth and pattern, and to tank size so a simple empirical equations describing optimal first-flush amount could be developed:

$$ff_{d,opt} = 1 - 1.2\ln\ 1\text{-}\eta_{\mathrm{r}} \qquad \text{for normal (25 day) accumulation}$$

$$ff_{d,opt} = 0.8 - 0.9\ln\ 1\text{-}\eta_{\mathrm{r}} \qquad \text{for fast (1 day) accumulation}$$

Where $ff_{d,opt}$ is the optimal design first-flush diversion (mm); and η_r is the target removal efficiency.

Optimal reset time was sensitive to overall rainfall depth but not to rainfall pattern, so a single equation suffices to describe the reset time corresponding to a particular design diversion and target removal efficiency:

$$t_r = \left(-12000\eta_r + 12000\ \frac{t_a^{\ 0.14}}{r_a} \right) ff_d - \ 0.6 + 0.012e^{\ 5.5\eta_r}$$

Where:, t_r is the reset time (days); r_a is the annual rainfall (mm) and t_a is the accumulation time (days).

This method produces devices that have a better performance than systems where reset is matched to accumulation and results in smaller devices with faster reset times which are, in consequence easier and cheaper to make. The diversions suggested by this optimisation are, however much larger first flush diversions that have been previously recommended in the literature. The reset times are also much slower than those described in the literature and vary between a few hours and several days. These scales of reset time are technically possible only through the use of porous materials. Weep-holes will either be too big and empty much to quickly or will be liable to blockage

14.2. OTHER OUTCOMES

In the course of this research a number of other conclusions have come to light and several tools have been developed.

14.2.1 TECHNOLOGY

Based on bills of materials from 60 tanks, tanks had economies of scale in manufacture equivalent to Volume$^{0.65}$ based on this work, “normal” and “low-cost” ranges have been derived. Relative labour content was found to design specific but little changed by size within a design. The economies of scale in tank building were not sufficient to overcome the diseconomies of scale in tank performance. The “Equivalent Unit Cost” was proposed as a more accurate way of describing tank cost than simple unit cost which does not take economies of scale into account.

Investigation of a number of designs and methods employed in tank building shows that a number of strategies can be employed to reduce tank cost: Potential strategies for cost reduction are:

- Material reduction through improved formwork and shape optimisation
- Greater use of materials which can be gathered rather than bought, through functional separation
- Mass production of tanks and components
- Use of existing containers

Cost reduction achieved by reduction in system quality will impact initially only upon appearance, then on longevity and finally on water quality. Domestic systems for which water quality is paramount should stop short of this point and a number of minimum criteria have been developed:

- The tank should not have excessive loss through seepage or evaporation

- It should have sufficient structural strength to withstand water pressure and normal wear & tear

- 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 commensurate with its intended use. Drinking water in particular requires that:

  - the tank be covered to prevent entry of light, and sealed against intrusion by vermin including mosquitoes

  - the tank be ventilated to prevent anaerobic decomposition of any washed-in matter

  - the tank not give the water an unacceptable taste

Reduced quality designs are also likely to have components that need either frequent replacement or routine maintenance. Judicious design should make replacement easy, maintenance a task a householder can perform and replaceable parts a small proportion of the overall cost. These maintenance tasks should be spelled out when a design is being selected by householders so they can make informed decisions.

A number of low-cost tank design ideas were tested. The most promising were:

- Thin-shell cement lining as found in the underground cement tank with an organic roof and in the partially below-ground tank with a ferrocement dome

- 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

Special circumstances can also be accommodated at little cost. The costs of the designs are shown along with some benchmarks in Table 7.18. Labour fractions and household labour fractions were also enhanced.

Table 14.1: Equivalent unit costs of rain-tanks (based on Ugandan material and labour costs)

Tank type	Tank cost	Tank capacity (m^3)	Simple Cost per m^3	Equivalent unit cost
Moulded plastic	£470	25	£19	£94
Drum tank	£72	0.5	£140	£88
Open frame ferrocement	£220	10	£20	£60
Pumpkin tank	£100	5	£20	£42
Plate tank	£94	10	£9	£26
Thai jar	£28	2	£14	£23
Dome tank	£50	5	£10	£21
Mud tank	£28	2	£14	£19
Tube tank	£18	1	£18	£17
Tarpaulin tank	£39	5	£8	£16
Thatch tank	£38	5	£8	£16

The use of reduced quality designs in formal water programmes may cause resentment. If householders are exposed to trade-offs, this can form the basis of a dialogue that can result in them choosing a technology that matches their needs and abilities. A “roofwater harvesting ladder” approach is described where “climbing” can take place in two dimensions (quality and quantity).

An analytical solution was found determine the material saving of a cylindrical tank with a perfect foundation and good agreement has been found between this and finite element analysis, however it is likely that in the field vertical tensile forces will be present in any underground tank due to skin resistance between the tank wall and the surrounding soil making the analysis more complex.

14.2.2 HEALTH

A number of criteria were developed for good inlet treatment design for use in a low-cost system in a low-income country:

- The device should be easy to clean or largely self-cleaning and the need for user intervention minimised or eliminated

- It should not block easily (if at all) and blockages should be obvious and easy to rectify

- It should not provide an entrance for additional contamination
- The cost should not be out of proportion with the rest of the system; Uganda will only spend about 5% of the cost of the system on inlet treatment, users in Sri Lanka will spend closer to 10%.

A computationally simpler equation than that found in the literature was developed to describe material accumulation over time from a non-zero starting point.

14.3. RECOMMENDATIONS FOR FURTHER WORK

14.3.1 TECHNOLOGY

LINERS FOR MUD TANKS

One of the most promising tanks from a cost perspective was the mud tank. The small prototypes tried worked well, were well received and have been replicated. One large tank was constructed and held water but a reliable lining was problematic. A scalable lining is necessary for this design to be made larger. More specifically, some means of joining plastic sheeting in the field with 100% reliability is necessary as even a small leak may cause the tank material to liquidise and the entire tank will then fail – possibly catastrophically.

LONG-TERM FIELD-TESTING OF REDUCED-QUALITY DESIGNS

The maintenance routines described in Section 8.5 are based on a short period of time and are in-part estimated. Longer-term testing needs to be done to determine if these estimates hold over a longer period of time. Initial field testing has resulted in a number of design changes which have thus far been untested in the field.

14.3.2 HEALTH

MOSQUITO BREEDING

Mosquito breeding in tanks is controversial with one large study providing conflicting results and laboratory work indicating that while mosquitoes do breed in tanks adults are unlikely to emerge. A preliminary study indicates that pupa numbers (and therefore maturity and emergence) are related to tank design and condition. A larger study is necessary to confirm these results. The pupal survey appears a good methodology to follow and it would be useful to investigate correlation with nutrient levels. These results should be linked to a standardised set of criteria for best-practice tank design and condition.

INLET/OUTLET ARRANGEMENTS

At present there is some controversy surrounding the best place to put the outlet of a rainwater tank, and the role of bacterial motility and heavy metal sedimentation in this placement. There is a need to measure actual bacteria and health-significant chemical pollutants at a large number of heights within the water column, at frequent intervals, with varying inlet and outlet arrangements and simulated rainfall events.

FIRST-FLUSH – ACCUMULATION

Washoff is fairly well described by the research in this thesis, however only an early indication of accumulation rate was possible. While it was found that sizing of first-flush diverters was only mildly sensitive to accumulation, more order-of-magnitude data would be useful to confirm the results in this thesis. It would also be a good idea to have this correlated with location so rules-of-thumb can be drawn.

FIRST-FLUSH – MAXIMUM ACCUMULATION

The size of a first-flush system depends on the desired removal performance. This is easy to describe in terms of percentage removal, however the actual initial contamination is unknown. More specifically, if accumulation is to be calculated, the maximum possible accumulation must be ascertained. In the field experiments done as part of this thesis, turbidities in excess of the range of the apparatus used were found in some cases. Like the accumulation study described above, a series of experiments in a number of locations is needed to provide rule-of-thumb for maximum accumulation.

FIRST-FLUSH – EFFECTS OF RAINFALL INTENSITY

While rainfall intensity is largely irrelevant to the design of fixed-volume first-flush diverters, this is not the only possible design. High-resolution measurement and datalogging should be able to separate rainfall intensity effects. Time-stamped datalogging can also go some way towards describing accumulation. It is possible to trigger a datalogger using a fixed volume of runoff, measure turbidity and conductivity with electronic instruments and time-stamp the data. From this the runoff intensity can be derived, dry periods calculated and exact indications of dissolved and suspended material collected. As this involves expensive equipment which may be vulnerable to theft or vandalism, it is unlikely that this work can be carried out at unsupervised field stations, it should however be possible in a few well-chosen locations over a period of time.

OVERALL SYSTEM PROCESSES

The level of necessary cleaning to be affected by a first-flush system depends on both the incoming dirt load and the needed dirt load at the tank inlet. This will depend upon dynamic behaviour in the tank. Die-off is reasonably well understood, settlement can be analytically described using the particle size distributions found in this research, and reasonable estimates made for reduction processes for heavy metals. These processes need to be combined with the mass balance model described in Chapter 13 to find the level of removal efficiency necessary to get the tank outlet water to WHO standards for a given exceedance.

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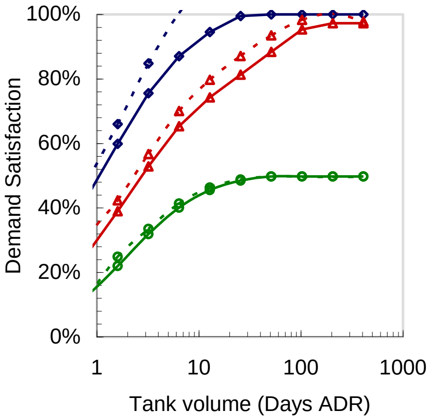
APPENDICES

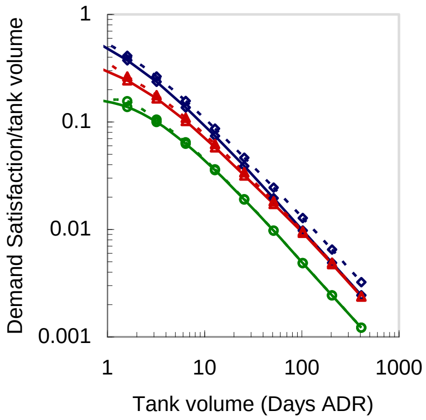
APPENDIX A
TANK PERFORMANCE CHARTS

Figure 24: Demand satisfaction

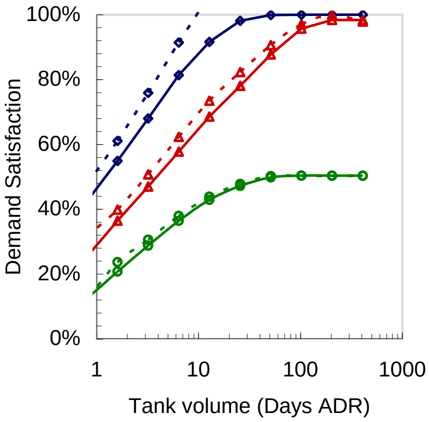
Quantities – Mbarara



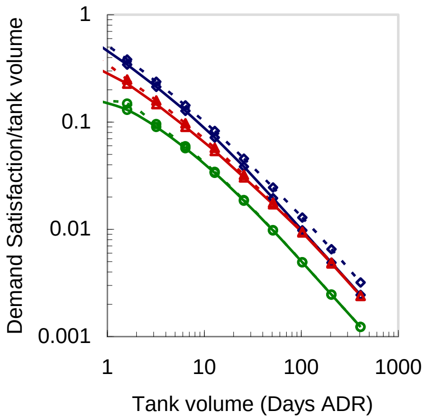
Unit quantities – Mbarara



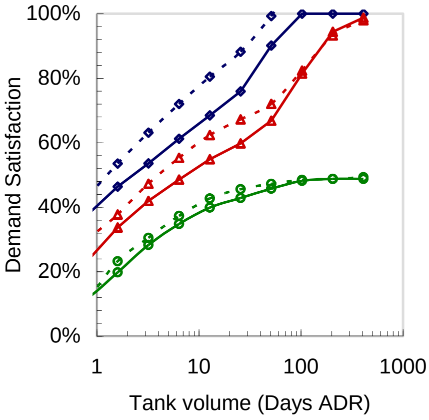
Quantities – Kandy



Unit quantities – Kandy



Quantities – Addis Ababa



Unit quantities – Addis Ababa

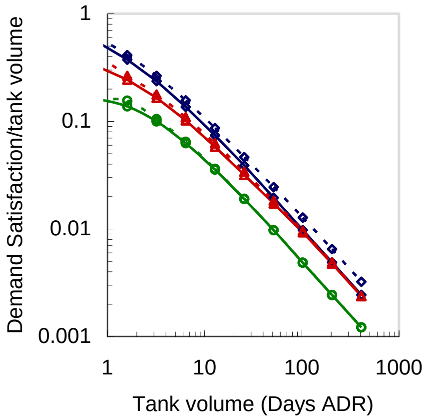
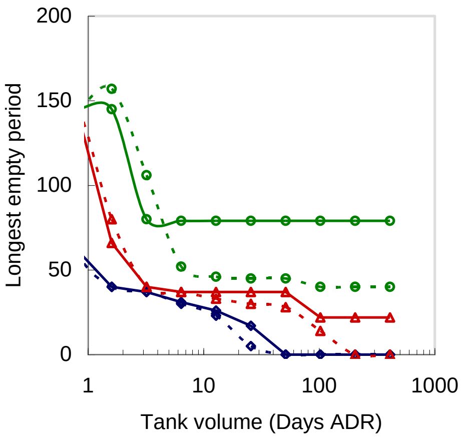
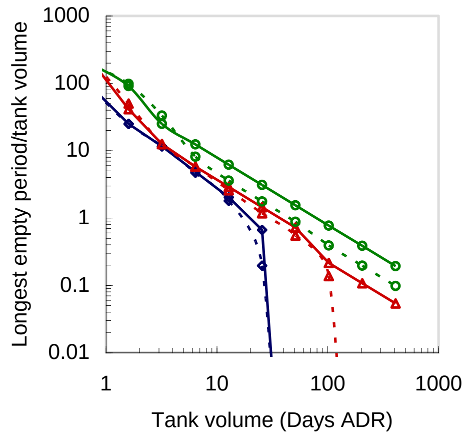


Figure 25: Longest empty time

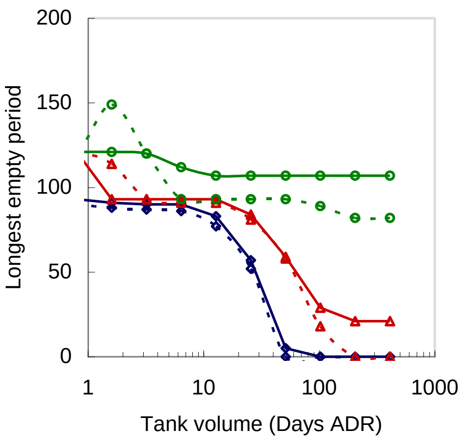
Quantities – Mbarara



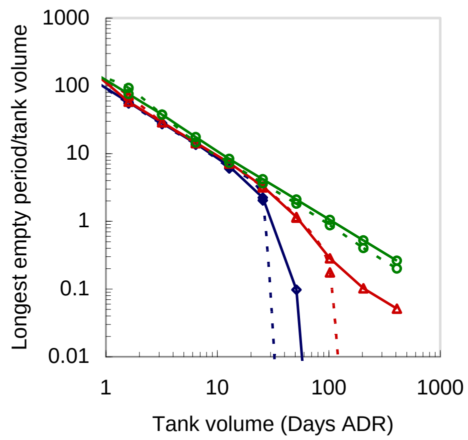
Unit quantities – Mbarara



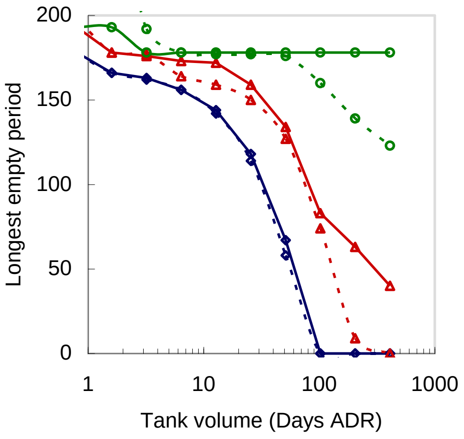
Quantities – Kandy



Unit quantities – Kandy



Quantities – Addis Ababa



Unit quantities – Addis Ababa

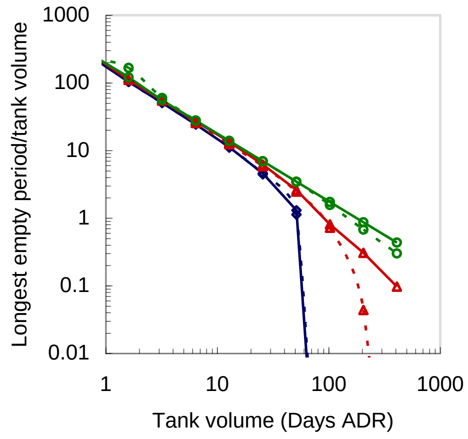
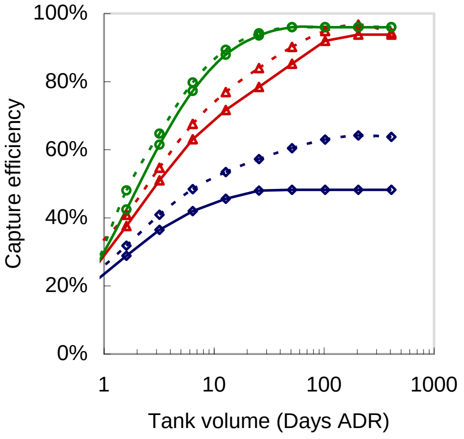
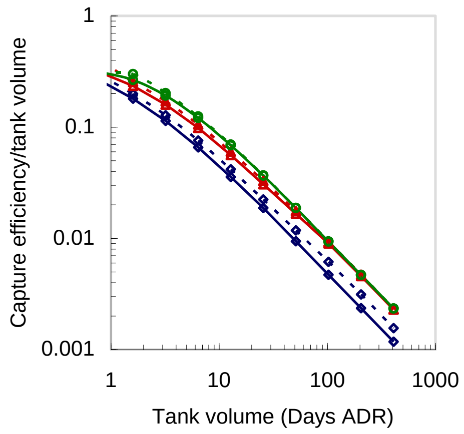


Figure 26: Capture efficiency

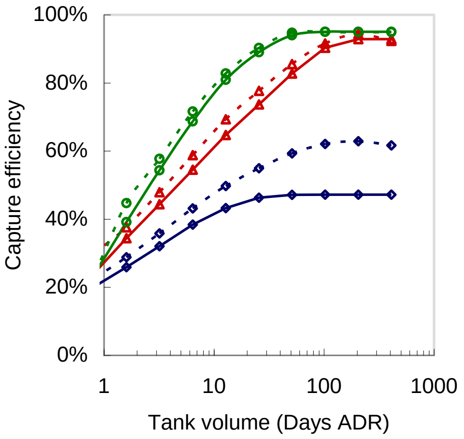
Quantities – Mbarara



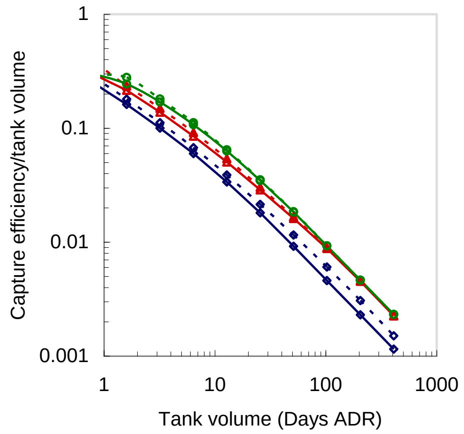
Unit quantities – Mbarara



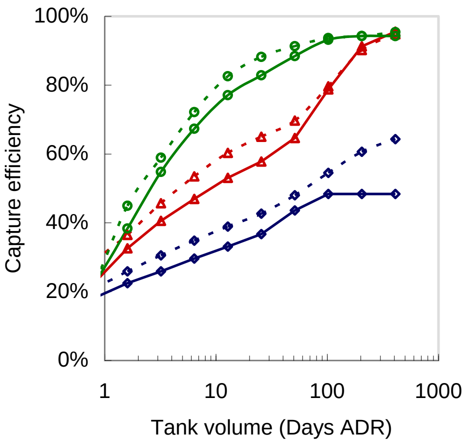
Quantities – Kandy



Unit quantities – Kandy



Quantities – Addis Ababa



Unit quantities – Addis Ababa

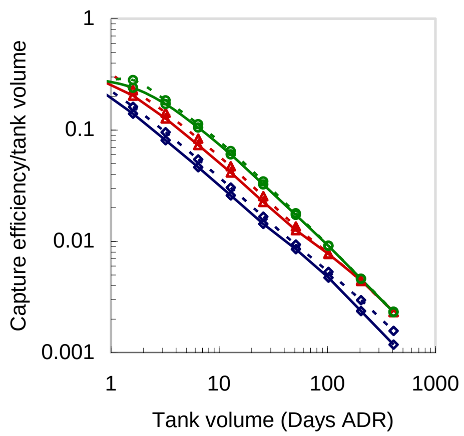
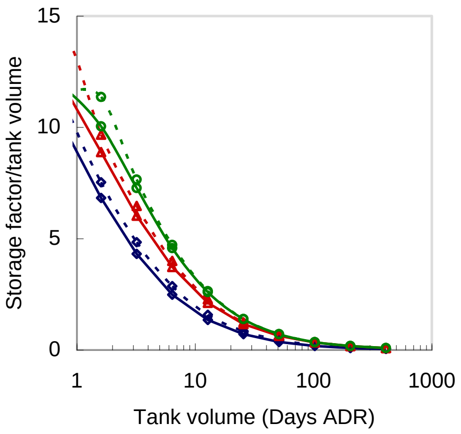
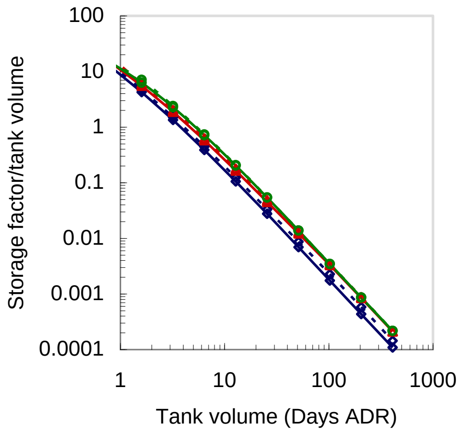


Figure 27: Storage factor

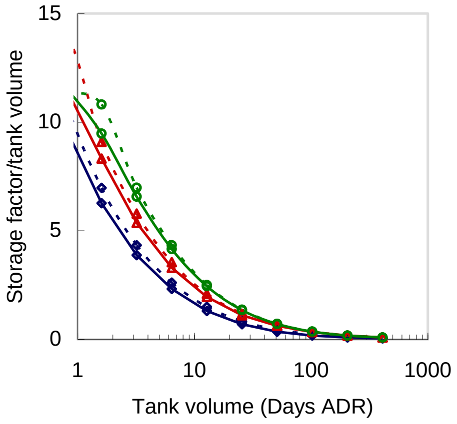
Quantities – Mbarara



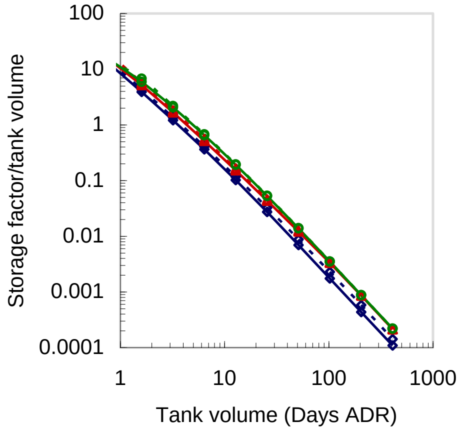
Unit quantities – Mbarara



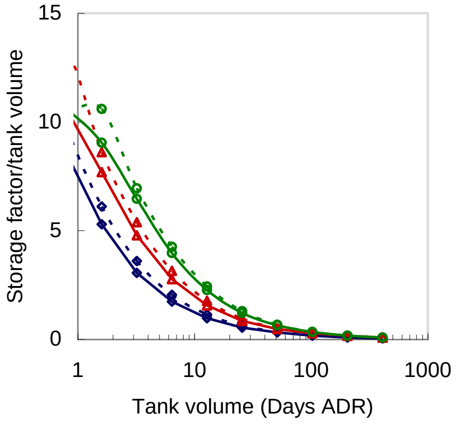
Quantities – Kandy



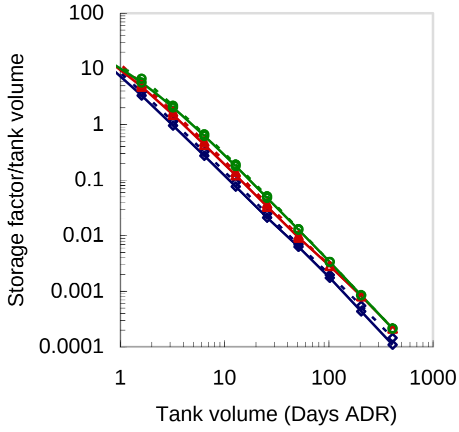
Unit quantities – Kandy



Quantities – Addis Ababa



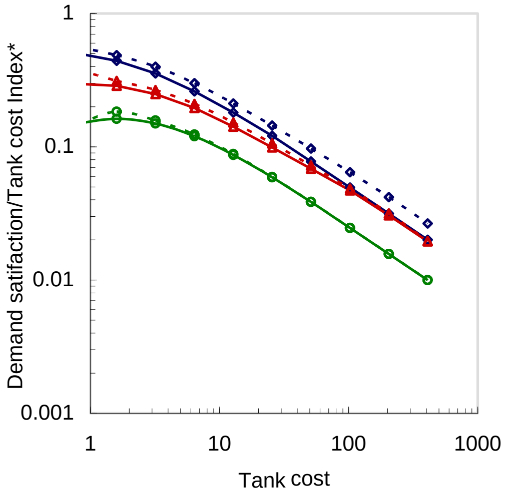
Unit quantities – Addis Ababa



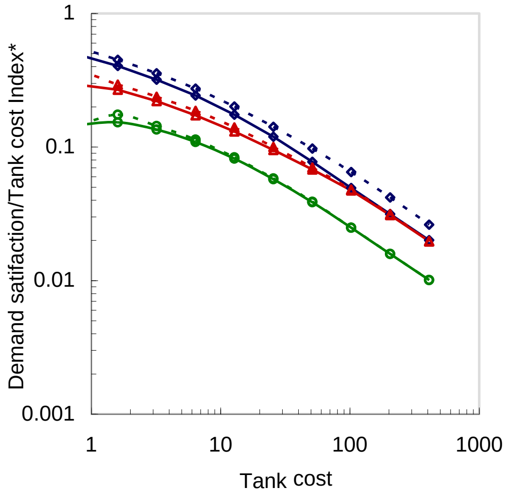
APPENDIX B
TANK PERFORMANCE CHARTS
COST ECONOMIES OF SCALE INCLUDED

Figure 28: Demand satisfaction

a. Mbarara



b. Kandy



c. Addis Ababa

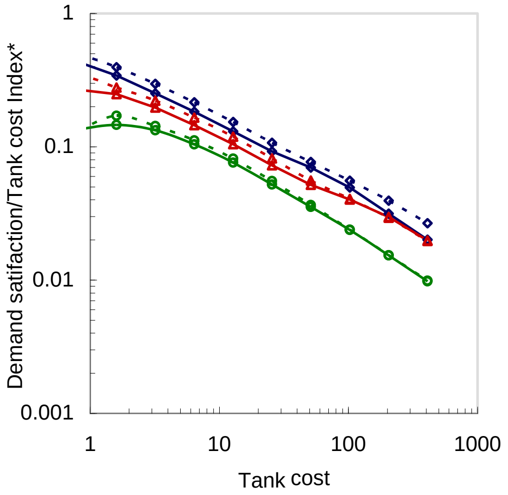
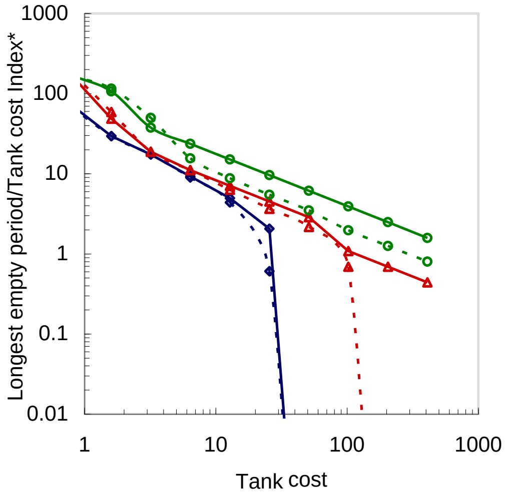
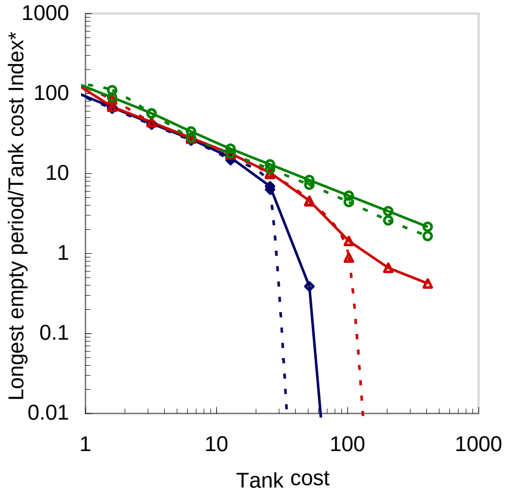


Figure 29: Longest empty period

a. Mbarara



b. Kandy



c. Addis Ababa

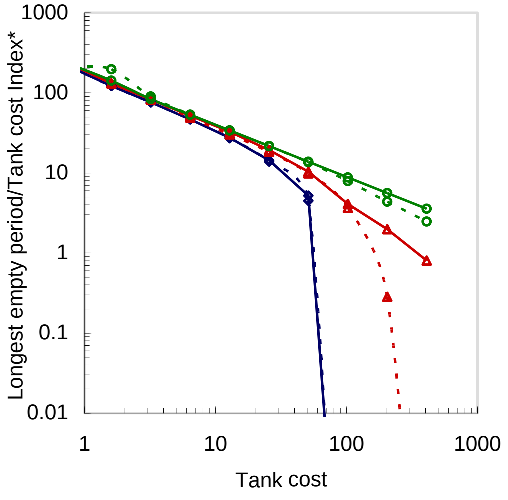
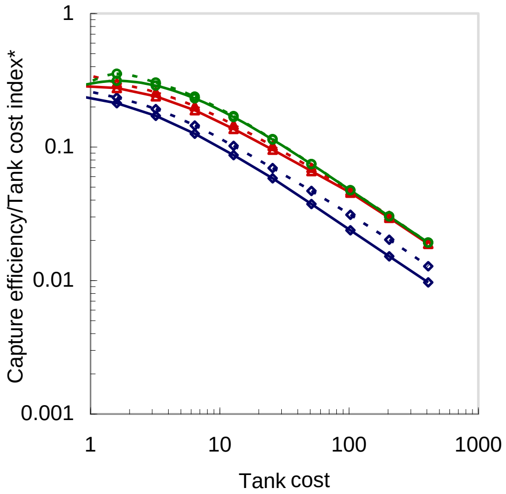
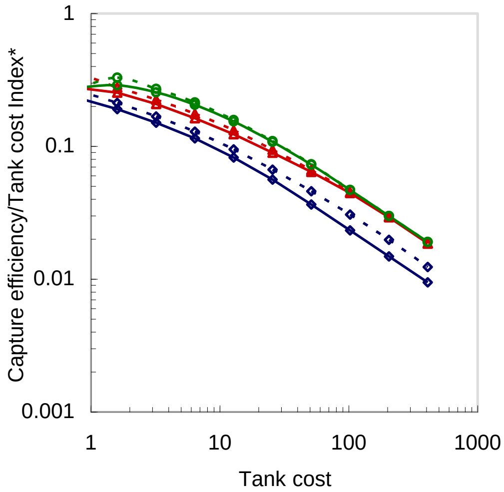


Figure 30: Capture efficiency

a. Mbarara



b. Kandy



c. Addis Ababa

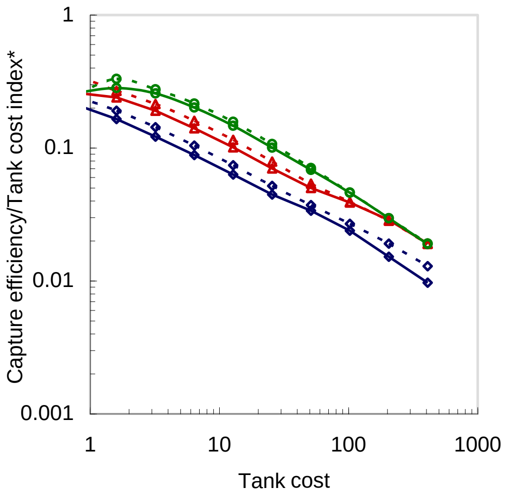
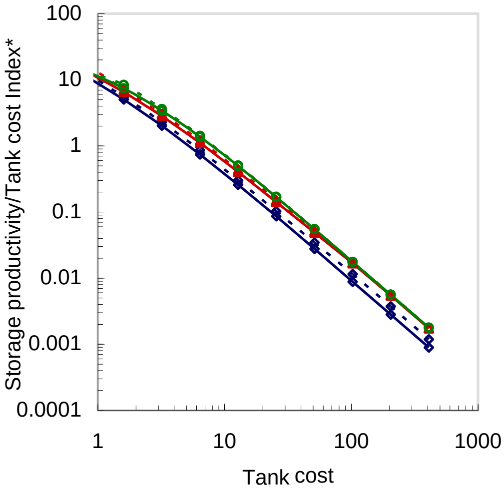
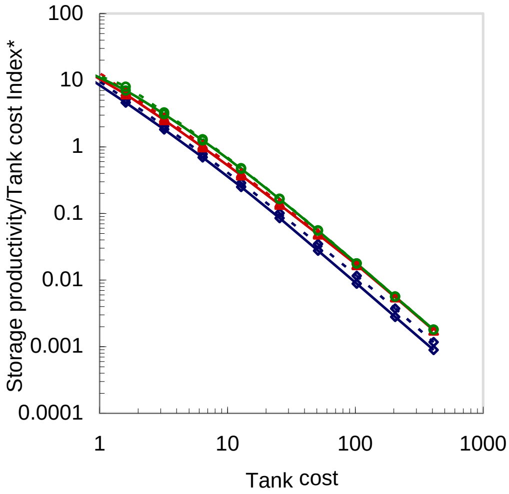


Figure 31: Capture efficiency

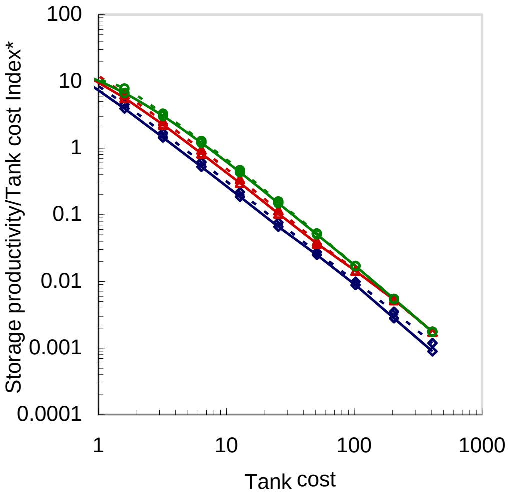
a. Mbarara



b. Kandy



c. Addis Ababa



* Tank cost index is based on normalising to the cost of a 1 day ADR tank

APPENDIX C
TANK DESIGNS

MOULDED PLASTIC

Mass produced commercial quality tank

LOCATIONS USED

Worldwide

DESCRIPTION

Plastic tanks, usually made from HDPE or GRP form the fastest growing segment of storage provision. They are already popular in developed countries where they compete directly with older technologies such as steel or concrete on a price basis. In developing countries, these tanks are generally more expensive by a factor of 3-5 which has slowed their adoption, however this is changing, In Sri Lanka the price penalty of a plastic tank is down to about 1.5-2 and in South Africa they are generally considered cheaper

Even in countries where there is a price premium for plastic tanks, they are often employed by water supply organisations, as they are quick to install and are known to work reliably (usually backed by a manufacturers guarantee). Consumers also like the tanks and see them as the most up-to-date method of storing water. They do, however put very little into the community in the way of employment generation and do not lend themselves to any kind of community input other than cash.

Plastic tanks in Uganda
[Picture D Ddamulira]

Plastic tank in Sri Lanka
[Picture T. Ariyananda]

CLOSED FORM FERROCEMENT TANK

Formwork used to reduce cost

LOCATION

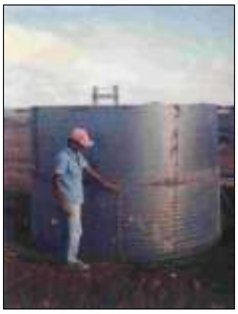
Africa, Asia and South America

DESCRIPTION

The closed form ferrocement tank has been used since the mid 1970s in many countries in Africa, Asia and the Americas. Early versions used a corrugated iron lid that has since proved too unreliable. These days, most ferrocement tanks have a domed cover also of ferrocement.

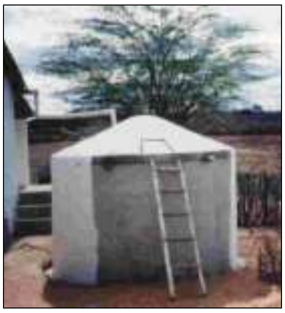
The tank is made by using a solid formwork made of either corrugated or flat galvanised steel sheet which is made in sections that bolt together forming a cylinder. Mesh is wrapped around this form and galvanised wire wound in a spiral around the tank with smaller spacing at the bottom and larger spacing at the top. The mesh is then plastered over with mortar, which is left to cure overnight. The form is then dismantled and the inside plastered with mortar. Most of these tanks are then lined with cement slurry that seals the tank rendering it waterproof; others use a waterproofing agent in the main mortar coating.

Screwing together the steel sheet form



Steel sheet form, wrapped by wire mesh and galvanized wire

Application of the first layer of mortar on top of the wiring



Finished tank being painted

Note: pictures taken from Gnadlinger [1999a]

SOURCES

[Gnadlinger, 1999a; Szilassy, 1999; Watt, 1993]

OPEN FRAME FERROCEMENT TANK

Flexible sized tank with low tooling costs

LOCATION

Throughout Asia and Africa

DESCRIPTION

Developed slightly later than the closed form ferrocement tank, the open frame tank is more expensive overall. but much more flexible in size. It has now become the most popular form of ferrocement tank and is made in Africa and Asia. A cylinder made from BRC mesh or a network or reinforcing bars replaces the role of the solid form, which means that there are no additional mould costs and any size can be made. The bars also form part of the overall structure resulting in a stronger overall tank (although the square mesh will also tend to concentrate stresses somewhat).

The downside to this design is that the form is included in the tank structure and therefore not reusable and the formwork is also more flexible resulting in a thicker and less controlled wall section. This has resulted in a tank that generally costs more than the closed-form structure.

Moving the mould into place

Partially rendered tank

Note: pictures taken IRDC [1986]

SOURCES

[Gera, 1999; Gould & Nissen-Petersen, 1999; IRDC, 1986; Nissen-Petersen & Lee, 1990]

PUMPKIN TANK

Ferrocement shape optimisation

LOCATION

Sri Lanka

DESCRIPTION

Initially developed for boat building, ferrocement is a mouldable material and lends itself to the manufacture of a large number of different shapes. This has been exploited to make tanks approach the ideal spherical shape – maximising volume for surface area and reducing bending forces. Notable is the Sri Lankan “pumpkin” tank.

The pumpkin tank was developed as part of the Community Water Supply and Sanitation Programme in Sri Lanka in 1995 and since than several thousand have been built as part of water supply schemes in various parts of Sri Lanka. The design uses an open formwork that is shaped to approximate a sphere by rounding the top and bottom. Another advantage of this scheme is that there is no need for a large separate cover.

One of the formwork legs

Partially rendered tank

Finished tank
[Pictures D. Rees[

SOURCES

[Heijnen & Mansur, 1998; LRWHF, 2000]

PLATE CISTERN – BRAZIL VERSION

Modular construction for a large tank

LOCATION

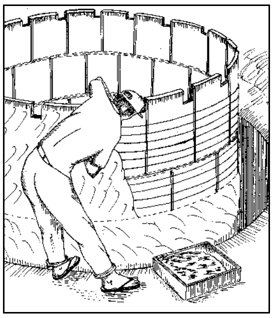
Brazil

DESCRIPTION

The plate tank was developed in Northeaster Brazil where it is now the most popular form of tank construction. It is made partially below ground with about 2/3 below ground and 1/3 above. The construction is of plated of mortar or sometimes concrete 3-4cm thick and about 50cm square made in a steel mould. The plates are placed together and fixed by winding wire around the construction. A layer of mortar is plastered inside and out to finish the tank. The roof is also made from pre-cast parts which are placed and plastered over to produce the final tank.


Making the plates


Fixing the plates


Plastering the outside


Installing the roof

Note: pictures taken from Gnadlinger [1999a] and Haury [2002]

SOURCES

[Gnadlinger, 1999a; Haury, 2002]

FERROCEMENT PRECAST SEGMENTAL SHELL TANK

Large mass produced tank

LOCATION

India

DESCRIPTION

In response to the need for a quick to install, mass produced tank, the Structural Engineering Research Centre (SERC) of Ghaziabad near Delhi developed a tank that can be made in sections, transported to a site and assembled. The sections are made on a rounded support resulting in a smooth cylindrical shape and reduced stress concentration at the join. The mould is either made on-site from sand or from other materials such as steel in a workshop. A panel of BRC mesh about 1m x 1.5m is bent over the base and mesh laid on top. Mortar is then pressed through the mesh against the curved surface of the base leaving a 10cm border around the outside and the panels are allowed to cure. The final assembly is achieved by binding the panels together with binding wire and plastering over the join.


Mesh


Laying mortar on mesh


Removing panels


Assembling panels

SOURCES

[Sharma, 2005]

INTERLOCKING BLOCK TANK

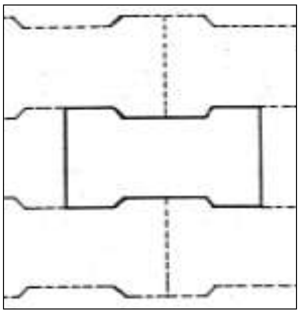
Above-ground material substitution with new jointing methods

LOCATION

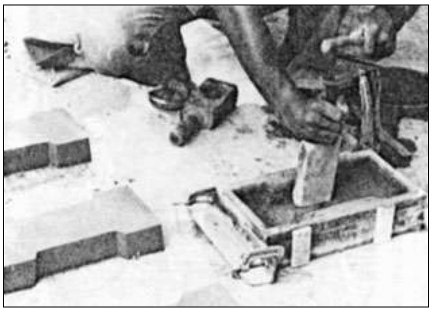
Thailand

DESCRIPTION

Several attempts were made to reduce costs in larger tanks in Thailand including bamboo cement and various blocks. .While not widely replicated due to its complexity it is instructive. The blocks were designed to interlock, efficiently transferring the load between blocks and reducing mortar used for joining. The thickness of the blocks and the high level of compaction used in their manufacture results in a tank that can be made without reinforcement, although a ring of reinforcement was incorporated for safety. Problems came with aligning the blocks and ensuring a consistent layer of binding between the blocks. If these problems can be overcome, the technique could conceivably be used with stabilised soil blocks or shaped burned bricks dramatically reducing cement use.



Block configuration



Making blocks

Finished tank

Note: pictures taken from IRDC [1986]

SOURCES

[IRDC, 1986].

BRICK-LIME CISTERN

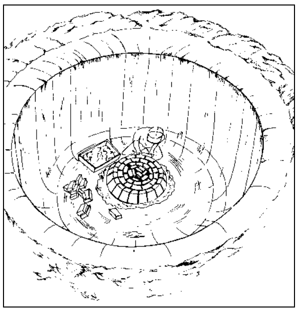
Underground functional separation

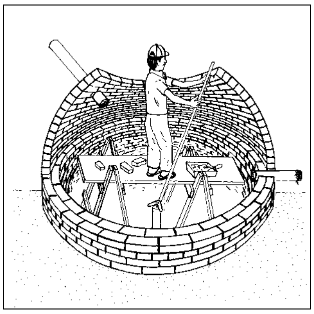
LOCATION

Brazil

DESCRIPTION

When the ground can be relied on to take some of the load, tanks can be made much weaker and materials that would otherwise be unusable can be employed. Such an underground design is the brick-lime cistern developed in the northeast of Brazil and has been built for several years. The design is totally underground with only a dome protruding. Locally-made burned bricks are laid directly against the sides of an excavation and mortared together with lime resulting in a slightly flexible structure that transfers a great deal of its load to the surrounding earth. The inside surface is sealed with a lime-cement mix and waterproofing is achieved by a cement slurry coating applied with a brush.


Beginning building the cistern


Making the brick dome


finished tank

SOURCES

[Gnadlinger, 1995; Gnadlinger, 1999a; Gnadlinger, 1999b]

THAI JAR

Workshop-based production using solid formwork and an optimised shape

The 1980s was the Water and Sanitation Decade but by the end of a decade of extended effort and targeted development, most countries were little better off than at the beginning. Not Thailand, however. By the end of the decade, Thailand could boast almost 100% water-supply coverage. No small part in this was played by the development of rainwater harvesting technologies and particularly the “Thai jar”.

The jar began as a community-made item using a mould made from sacking filled with sand or sawdust. The jars soon reduced in price to about US$20 largely through commercial manufacture. The price today is less than US$15 and the jars are almost universally found in rural homes in northern Thailand and are also to be found in neighbouring countries such as Cambodia where they sell for less than US$10. This price makes the jars affordable by all but the poorest and has caused DRWH to become widespread without further input from any institution.

The secret of the low cost is not necessarily “mass manufacture” in the traditional sense as they are often made by part-time farmers in small batch quantities, but in the optimised shape, the quality and availability of tooling and in the quality control available by making them in a workshop space rather than on-site. Each jar is made on a mould made of cement bricks, which are coated with mud as a mould release. The mould sets themselves are also made locally, so a factory may have several. The steel formers for making the moulds, however, are made centrally ensuring tight quality control of the size and shape. The high quality solid mould allows a very uniform and thin coating of mortar to be applied resulting in a highly optimised product.

Attempts have been made to transfer the jar to other countries notably in Africa, however while the basic design of a small jar has been maintained, the workshop practice has not – with most jars being made with using filled sacks as formwork. This has resulted in a product that, while cheap compared to larger tanks, is much more expensive (and less well finished) than jars made in Thailand. More recently there has been a move toward using workshops and wooden moulds which has yielded a more economical product.

Jar factory with made jars

Mould pieces

Transporting jars by cart in Uganda

SOURCES

[Ariyabandu, 2001; Bradford & Gould, 1992; IDRC, 1986; Wirojanagud, 1990; Luong, 2002; DTU, 2006; Vinh, 2003]

TARPAULIN TANK

Ground support used by low-cost mass produced and gatherable materials

LOCATION

Uganda

DESCRIPTION

The tarpaulin tank is an excellent example of what can be achieved if a strict eye is kept on the costs while maintaining only the bare essentials of function.

The civil war in Rwanda brought large numbers of refugees into southern Uganda. Many of these refugees settled in the mountains near the town of Mbarara in places such as the Orikinga valley where the water table well below the surface and can be contaminated with unacceptable levels of iron and manganese.

The Refugees had little capital to buy equipment but the UNHCR had supplied several tarpaulins to be used as shelter. On finding these tarpaulins waterproof, a number of families lined holes with them and used them to collect rainwater. This was successful; however the lined pits were liable to ingress of foreign matter and were open to the sky, allowing algae to develop resulting in a reduction in water quality over time.

ACORD Uganda worked with the households to develop an improved design that would allow for increased water quality but retain the low-cost nature of the tank. The improved design featured an enclosure made from wattle and daub with a galvanised steel roof. The enclosure meant that light and foreign matter were kept out of the tank improving water quality; the top edge of the tarpaulin could be raised about 10cm to keep ground runoff out of the tank; an overflow arrangement could be introduced and access to the tank was by dipping a half-jerrycan through a wooden door.

The tarpaulin tank is however, not a durable solution in all cases. The problems are primarily location related – if the design works in one place in a location, it should work everywhere, if it does not, it will not be suitable for the location.

Frame

Daubing the walls

The tarpaulin

Completed tank

Note: Pictures from Rees [2000]

SOURCES

[Rees, 2000]

PVC LINED CONCORD CLOTH BAG WITH BAMBOO FRAME

Above-ground quality reduction

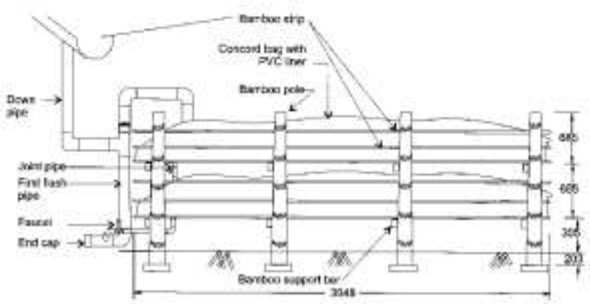
LOCATION

Bangladesh

DESCRIPTION

The PVC Lined concord cloth bag is a recent innovation by International Development Enterprises (IDE). Like the tarpaulin tank the majority of the materials are gatherable and others are made locally. Only the PVC liner and plumbing need to be imported. The above-ground design allows the water to be drawn without effort and also allows the tank to be monitored for damage.

The tank consists of two cloth bags waterproofed with a PVC liner. Each bag is laid horizontally in a bamboo frame and plumbed to a tap. The total capacity of the tank is about 3000 litres and the total cost is about $30.



General Assembly Diagram


Competed tank

SOURCES

[IDE-Bangladesh, 2004]

APPENDIX D
SELECTED RESULTS OF STUDIES OF WATER QUALITY

BIOLOGICAL CONTAMINATION

INDICATORS

Table 2: Thermo- tolerant coliforms (Faecal coliforms (FC) and *E-Coli*)[1] (WHO recommendation: <1 CFU/100ml but risk zones are also indicated – 1-10 = low risk; 10-100 = intermediate risk; 100-1000 = high risk; >1000 = very high risk [WHO, 2004])

<table>
<tr><th>Location</th><th>Indicator</th><th>Notes</th><th>Source</th></tr>
<tr><td>Micronesia</td><td>FC</td><td>One-time survey of 203 RWH systems.
43% samples positive
19% > 10 cfu/100ml
9% > 100 cfu/100ml
Median 0.4 cfu/100ml</td><td>[Dillaha & Zolan, 1985]</td></tr>
<tr><td>Hawaii</td><td>E. Coli</td><td>One-time sampling of 9 cisterns about 6” below surface and from HH tap.
Tank: 8/9 positive (range 0 – 4 700; mean 880 CFU/100ml)
HH; 3/9 positive (range 0 – 7; mean 1 CFU/100ml)</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>Hawaii</td><td>FC</td><td>One-time sampling of 9 cisterns about 6” below surface and from HH tap.
Tank: 8/9 positive (range 0 – 4 800; mean 923 CFU/100ml)
HH; 4/9 positive (range 0 – 48; mean 6 CFU/100ml)</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>Hawaii</td><td>FC</td><td>One-time sampling of 15 cisterns about 6” below surface and from HH tap.
Tank; range 0 – 520; mean 103 CFU/100ml)
HH; range 0 – 420; mean 58 CFU/100ml)</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>Thailand</td><td>E. Coli</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers<table><tr><th></th><th>% not meeting std (0 cell/100 ml)</th></tr><tr><td>Roof and gutter</td><td>10%</td></tr><tr><td>Tank and jar</td><td>12%</td></tr><tr><td>In-house</td><td>33%</td></tr></table>Settling and die-off presumed to reduce contamination. Secondary contamination when brought from tank to house also significant. FC:FS ratio indicates that tank water contamination was from animals, but household contamination was from both animals and humans</td><td>[Wirojanagud, 1991]</td></tr>
</table>

[1] Note: Faecal Coliforms are a subset of thermo-tolerant coliforms and *E.Coli* are a subset of Faecal coliforms. Most test methods actually test for thermo-tolerant coliforms however authors tend to report either faecal coliforms or *E.coli* in their results. Due to this ambiguity, all subsets of thermo-tolerant coliforms are brought together in this table – however some authors do report a specific test for *E-Coli* so the type of indicator reported is shown.

<table>
<tr><th>Location</th><th>Indicator</th><th>Notes</th><th>Source</th></tr>
<tr><td>Thailand</td><td>FC</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers
<table>
<tr><td></td><td>% not meeting std (0 cell/100 ml)</td></tr>
<tr><td>Roof and gutter</td><td>42%</td></tr>
<tr><td>Tank and jar</td><td>57%</td></tr>
<tr><td>In-house</td><td>22%</td></tr>
</table>
Settling and die-off presumed to reduce contamination. Secondary contamination when brought from tank to house also significant. FC:FS ratio indicates that tank water contamination was from animals, but household contamination was from both animals and humans</td><td>[Wirojanagud, 1991]</td></tr>
<tr><td>Virgin Islands</td><td>FC</td><td>2 samples taken from outlet tap at each of 16 sites
Range: 0 – 2 000 cfu/100 ml
Range of averages: 0 – 1 076</td><td>[Ruskin et al., 1992]</td></tr>
<tr><td>Queensland</td><td>FC</td><td>Rain collected from roofs. GI and concrete tile roofs. Rural, industrial and urban (6 in all) 8 samples from each catchment over 5 months
“Averages”
<table>
<tr><td>Rural</td><td>Urban</td><td>Industrial</td></tr>
<tr><td>GI: 117, Con: 2</td><td>GI: 0 Con: 122</td><td>GI: 117 Con: 2</td></tr>
</table>
Not correlated with preceding dry days</td><td>[Thomas & Greene, 1993]</td></tr>
<tr><td>Victoria</td><td>E. Coli</td><td>4 concrete and 2 GI tanks. 11-14 samples over 15 months for each type
ND</td><td>[Thurman, 1995]</td></tr>
<tr><td>Virgin Islands</td><td>FC</td><td>36% samples positive
<1 – 770 cfu/100ml
average 61.7 (public) 13.6 (private)
No correlation with cysts
Correlated with TCs (P 0.0012)</td><td>[Crabtree et al., 1996]</td></tr>
<tr><td>Germany</td><td>E. Coli</td><td>972 samples from 102 DRWH systems
14.3%>100 cfy/100ml, 7%>1 000, 2.8%>10 000</td><td>[Holländer et al., 1996]</td></tr>
<tr><td>Victoria</td><td>E. Coli</td><td>22 tanks, 1-3 samples per tank in rural area
12/47 samples positive (26%),
Range 0-370 cfu/ml, median 4 (mean 15), low
Counts associated with high pH, zero counts associated with high zinc
Concrete tanks had higher E-coli, “inert” the lowest.</td><td>[Bannister et al., 1997]</td></tr>
<tr><td>Pretoria</td><td>FC</td><td>Comparative study (figures are in cfu/ml – but are suspiciously high for all sources – more likely they are actually cfu/100ml)
<table>
<tr><td>Rainwater</td><td>90 – 260</td></tr>
<tr><td>River</td><td>5 900 – 23 000</td></tr>
<tr><td>Borehole</td><td>10 – 700</td></tr>
<tr><td>Well</td><td>260 – 7000</td></tr>
</table></td><td>[Nevondo & Cloete, 1999]</td></tr>
<tr><td>Singapore</td><td>FC</td><td>38 samples collected from one rooftop over 2 years
Mean: 6.7 mg/l, SD 8.9</td><td>[Appan, 1999]</td></tr>
<tr><td>Kerala</td><td>E. Coli</td><td>Comparative study of 30 tanks and 10 traditional sources
<table>
<tr><td>Rainwater</td><td>94% +ve
70%<100 cfu/100ml</td></tr>
<tr><td>Other (river, pond, well, spring)</td><td>100% +ve
100%<100 cfu/100ml</td></tr>
</table></td><td>[Pushpangadan et al., 2001]</td></tr>
<tr><td>New Zealand</td><td>E. Coli</td><td><1-4 900
%iles: 25th 3,Median 15,75th 64, 95th 432
Higher levels associated with tiled roof</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>New Zealand</td><td>FC</td><td><1-840
%iles: 25th <1,Median 2,75th 33, 95th 244
Higher levels associated with season, plastic tanks</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>Denmark</td><td>E. Coli</td><td>2-4 Samples taken from 14 DRWH systems over a period of 6 months. Samples from tank and toilet bowl (not reported here due to significant confounding factors)
11/14 positive, range: 4-990 CFU/100ml</td><td>[Albrechtsen, 2002]</td></tr>
</table>

<table>
<tr><th>Location</th><th>Indicator</th><th>Notes</th><th>Source</th></tr>
<tr><td>Jordan</td><td>E. Coli</td><td>Comparative study of 255 traditional cisterns, 31 concrete tanks, 149 steel tanks (% of sample positive)
<table>
<tr><td>Traditional cistern</td><td>17%</td></tr>
<tr><td>Concrete tank</td><td>0%</td></tr>
<tr><td>Steel tank</td><td>3%</td></tr>
</table>
Pit latrine within 15m increased risk (OR 2.3)
Good overflow decreased risk (OR 0.4)
Buckets used to remove water in “cisterns”. Captive buckets decreased risk substantially (OR 0.1)</td><td>[Abo-Shehada et al., 2004]</td></tr>
<tr><td>Zambia</td><td>FC</td><td>1 school and 4 HHs in urban Zambia tested for WQ at rain, roof and tank.. For comparison several other water supply options were also sampled
<table>
<tr><td>Rain (CFU/100ml)</td><td><1-1</td></tr>
<tr><td>FF system (GI)</td><td>16-TNC</td></tr>
<tr><td>FF system (asb)</td><td>6-TNC</td></tr>
<tr><td>Tank</td><td><1-6</td></tr>
<tr><td>Piped water</td><td><1-50</td></tr>
<tr><td>Private boreholes</td><td><1-56</td></tr>
<tr><td>Shallow wells</td><td><1-2</td></tr>
</table></td><td>[Handia, 2005]</td></tr>
<tr><td>NSW</td><td>FC</td><td>Three sites surveyed over a period of time. Average values quoted
Site 1: Roof: 135, Outlet: 20, tank surface: 119
Site 2: Roof: 135, Outlet: 1
Site 2: Outlet: <1, tank surface: 108</td><td>[Coombes et al., 2005]</td></tr>
</table>

Table 3: Total coliforms (WHO recommendation: Actual number not considered important but changes considered significant)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Virgin Islands</td><td>30 cisterns tested
4/30 (but not correlated with HPCs)
Up to 44cm of sludge found in cisterns mainly plant debris, dust, animal faeces, decomposing animal remains
Highest nitrates associated with highest bacteria</td><td>[Rinehart et al., 1985]</td></tr>
<tr><td>Micronesia</td><td>One-time survey of 203 RWH systems.
70% samples positive
39% > 10 cfu/100ml
15% > 100 cfu/100ml
Median 4.4 cfu/100ml
Statistically significant factors reducing count were; screened inlet, tank type (metal drums scored had 2.5- 8.8x higher median counts), cistern volume, presence of covers
Non-significant factors were cistern cleaning and roof and gutter cleaning</td><td>[Dillaha & Zolan, 1985]</td></tr>
<tr><td>Virgin Islands</td><td>Bi-weakly sampling of 20 private homes – 271 samples and 5 public housing complexes – 75 samples.
Private homes, 73% +ve
Public housing; 52% +ve
Poor correlation with Pseudomas Aeruginosa (15% PA+ve, TC <1)</td><td>[Krishna, 1989]</td></tr>
<tr><td>Thailand</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers
<table>
<tr><td></td><td>% not meeting std (<2.2 cell/100 ml)</td></tr>
<tr><td>Roof and gutter</td><td>58%</td></tr>
<tr><td>Tank and jar</td><td>34%</td></tr>
<tr><td>In-house</td><td>78%</td></tr>
</table>
Settling and die-off presumed to reduce contamination. Secondary contamination when brought from tank to house also significant. FC:FS ratio indicates that tank water contamination was from animals, but household contamination was from both animals and humans</td><td>[Wirojanagud, 1991]</td></tr>
<tr><td>Virgin Islands</td><td>2 samples taken from outlet tap at each of 16 sites
Range: 0 – 5 334 cfu/100 ml
Range of averages: 0 – 2 834</td><td>[Ruskin et al., 1992]</td></tr>
</table>

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Germany</td><td>972 samples from 102 DRWH systems
41%>100 cfy/100ml, 17.7%>1 000, 2.1%>10 000</td><td>[Holländer et al., 1996]</td></tr>
<tr><td>Virgin Islands</td><td>58% samples positive
<1 – 3 140 cfu/100ml
average 430 (public) 350 (private)
No correlation with cysts
Correlated with HPCs (P 0.006)
Correlated with FCs (P 0.0012)</td><td>[Crabtree et al., 1996]</td></tr>
<tr><td>Victoria</td><td>22 tanks, 1-3 samples per tank in rural area
27/47 samples positive (57%), range: 0-16 000 cfu/100ml, median 4</td><td>[Bannister et al., 1997]</td></tr>
<tr><td>Singapore</td><td>38 samples collected from one rooftop over 2 years
Mean: 92 mg/l, SD 97.1</td><td>[Appan, 1999]</td></tr>
<tr><td>Pretoria</td><td>Comparative study<table>
<tr><td>Rainwater</td><td>470 - 1000</td></tr>
<tr><td>River</td><td>1 180 – 49 000</td></tr>
<tr><td>Borehole</td><td>110 – 2 100</td></tr>
<tr><td>Well</td><td>1 180 – 49 000</td></tr>
</table></td><td>[Nevondo & Cloete, 1999]</td></tr>
<tr><td>New Zealand</td><td><1-19 000
%iles: 25^{th} 2,Median 27,75^{th} 220, 95^{th} 970
Higher levels associated with turbidity, season, GI roof</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>Jordan</td><td>Comparative study of 255 traditional cisterns, 31 concrete tanks, 149 steel tanks (% of sample positive)<table>
<tr><td>Traditional cistern</td><td>49%</td></tr>
<tr><td>Concrete tank</td><td>32%</td></tr>
<tr><td>Steel tank</td><td>25%</td></tr>
</table></td><td>[Abo-Shehada et al., 2004]</td></tr>
<tr><td>Zambia</td><td>1 school and 4 HHs in urban Zambia tested for WQ at rain, roof and tank. For comparison several other water supply options were also sampled<table>
<tr><td>Rain (CFU/100ml)</td><td><1-27</td></tr>
<tr><td>FF system (GI)</td><td>30-TNC</td></tr>
<tr><td>FF system (asb)</td><td>33-TNC</td></tr>
<tr><td>Tank</td><td><1-30</td></tr>
<tr><td>Piped water</td><td><1-120</td></tr>
<tr><td>Private boreholes</td><td>41-TNC</td></tr>
<tr><td>Shallow wells</td><td>9-37</td></tr>
</table></td><td>[Handia, 2005]</td></tr>
<tr><td>NSW</td><td>Three sites surveyed over a period of time. Average values quoted
Site 1: Roof: 359, Outlet: 166, tank surface: 834
Site 2: Roof: 359, Outlet: 18
Site 2: Outlet: 200, tank surface: 1 050</td><td>[Coombes et al., 2005]</td></tr>
</table>

Table 4: Heterotrophic plate counts (WHO recommendation: Actual number not considered important but changes considered significant)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Virgin Islands</td><td>30 cisterns tested
100% positive
Up to 44cm of sludge found in cisterns mainly plant debris, dust, animal faeces, decomposing animal remains
Highest nitrates associated with highest bacteria</td><td>[Rinehart et al., 1985]</td></tr>
<tr><td>Thailand</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers<table>
<tr><td></td><td>% not meeting std (<500 cfu/ml)</td></tr>
<tr><td>Roof and gutter</td><td>91%</td></tr>
<tr><td>Tank and jar</td><td>60%</td></tr>
<tr><td>In-house</td><td>88%</td></tr>
</table>Settling and die-off presumed to reduce contamination. Secondary contamination when brought from tank to house also significant. FC:FS ratio indicates that tank water contamination was from animals, but household contamination was from both animals and humans</td><td>[Wirojanagud, 1991]</td></tr>
<tr><td>Hawaii</td><td>One-time sampling of 15 cisterns about 6” below surface and from HH tap.
All samples >125 000 CFU/100ml</td><td>[Fujioka et al., 1991]</td></tr>
</table>

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Virgin Islands</td><td>2 samples taken from outlet tap at each of 16 sites
Range of averages: 1.5E2 – 8.2E5 cfu/ml</td><td>[Ruskin et al., 1992]</td></tr>
<tr><td>Victoria</td><td>2 000 – 20 000 cfu/ml
No effect of inlet filter on measurements
No seasonal variation</td><td>[Thurman, 1995]</td></tr>
<tr><td>Virgin Islands</td><td><10 – 5.6E7 cfu/100ml
average 4.1E6 (public) 2.4E5 (private)
not correlated with cysts
correlated with turbidity (P 0.03)
correlated with TCs (P 0.006)</td><td>[Crabtree et al., 1996]</td></tr>
<tr><td>Victoria</td><td>22 tanks, 1-3 samples per tank in rural area
35/45 samples positive (78%), range: 0-10 000 cgu/ml median: 1 400
Not affected by tank type</td><td>[Bannister et al., 1997]</td></tr>
<tr><td>Pretoria</td><td>Comparative study<table><tr><td>Rainwater</td><td>10 – 16 000</td></tr><tr><td>River</td><td>7 750 – 45 600</td></tr><tr><td>Borehole</td><td>0 – 11 500</td></tr><tr><td>Well</td><td>2 910 – 50 800</td></tr></table></td><td>[Nevondo & Cloete, 1999]</td></tr>
<tr><td>New Zealand</td><td>1-130 000 cfu/100ml
%iles: 25^{th} 125,Median 570, 75^{th} 1 850, 95^{th} 14 500
Higher levels associated with GI roof, GI tank</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>NSW</td><td>Three sites surveyed over a period of time. Average values quoted
Site 1: Roof: 1 360, Outlet: 331, Tank surface: 3 256
Site 2: Roof: 1 360, Outlet: 784
Site 2: Outlet: 76, tank surface: 1 050</td><td>[Coombes et al., 2005]</td></tr>
</table>

PATHOGENS

Table 5: *Campylobacter Sp.*

Location	Type	Notes	Source
Germany	*Campylobacter jejuni*	142 samples from 34 DRWH systems No samples positive	[Holländer et al., 1996]
Victoria	*Campylobacter Sp.*	22 tanks, 1-3 samples per tank in rural area 6/42 (13%) from 5 tanks (23%) – 2 samples have no E.coli or TC other samples showed a wide range of E.coli and TC	[Bannister et al., 1997]
New Zealand	*Campylobacter jejuni*	115 household tanks sampled 0% samples positive	[Simmons et al., 2001]
Denmark	*Campylobacter Sp.*	2-4 Samples taken from 14 DRWH systems over a period of 6 months. Samples from tank and toilet bowl (not reported here due to significant confounding factors) 2/17 positive Toilet (RW)t: 2/10 positive Toilet (DW): 0/5 positive	[Albrechtsen, 2002]

Table 6: *Cryptosporidium Sp.*

Location	Type	Notes	Source
Virgin Islands	*Cryptyosporidium*	48% samples positive <1 – 70.29 oocysts/100 litres Average – 2.41 (2.78, private; 1.73, public) The 70.29 reading is well outside the range of the other readings – max 10.57 and may dominate the overall and private average	[Crabtree et al., 1996]
Victoria	*Cryptosporidium*	22 tanks, 1-3 samples per tank in rural area Not detected	[Bannister et al., 1997]
New Zealand	*Cryptosporidium*	50 household tanks sampled 4% samples positive	[Simmons et al., 2001]

Location	Type	Notes	Source
Denmark	*Cryptosporidium*	2-4 Samples taken from 14 DRWH systems over a period of 6 months. Samples from tank and toilet bowl 6/17 positive range: ND-50 cells/ml Toilet (RW)t: 1/10 positive, range: ND-10 cells/ml Toilet (DW): 0/5 positive	[Albrechtsen, 2002]
Jordan	*Cryptyosporidium parvum*	Comparative study of 255 traditional cisterns, 31 concrete tanks, 149 steel tanks (% of sample positive) Traditional cistern: 2% Concrete tank: 0% Steel tank: 0% Positive results were from 6 - 60 oocysts/litre Buckets used to remove water in “cisterns”. Captive buckets decreased risk substantially (P 0.09) Entry point raised above floor level substantially decreased risk (P 0.02)	[Abo-Shehada et al., 2004]

Table 7: *Giardia*

Location	Notes	Source
Virgin Islands	26% samples positive <1 – 3.8 cysts/100 litres Average – 1.01 (1.12, private; 1.05 public)	[Crabtree et al., 1996]
Victoria	22 tanks, 1-3 samples per tank in rural area Not detected	[Bannister et al., 1997]
New Zealand	50 household tanks sampled 0% samples positive	[Simmons et al., 2001]
Denmark	2-4 Samples taken from 14 DRWH systems over a period of 6 months. Samples from tank and toilet bowl 0/17 positive Toilet (RW)t: 0/10 positive Toilet (DW): 0/5 positive	[Albrechtsen, 2002]

Table 8: *Legionalla Sp.*

Location	Type	Notes	Source
Virgin Islands	*Legionella*	One-time sampling from 10 randomly selected cisterns Present in 8/10 samples Average 266 cells/ml	[Broadhead et al., 1988]
Virgin Islands	*Legionella-like* organisms	2 samples taken from outlet tap at each of 16 sites 8/35 (23%), 6/16 cisterns Associated with HPCs of >100 000 cfu/ml	[Ruskin et al., 1992]
Germany	*Legionella*	410 samples from 38 DRWH systems No samples positive	[Holländer et al., 1996]
New Zealand	*Legionella*	23 household tanks sampled 0% samples positive	[Simmons et al., 2001]
Denmark	*Legionella - pneumophila*	2-4 Samples taken from 14 DRWH systems over a period of 6 months. Samples from tank and toilet bowl 0/14 positive Toilet (RW)t: 0/7 positive Toilet (DW): 0/5 positive	[Albrechtsen, 2002]
Denmark	*Legionella Sp. (other)*	2-4 Samples taken from 14 DRWH systems over a period of 6 months. Samples from tank and toilet bowl 5/7 positive Toilet (RW)t: 5/5 positive Toilet (DW): 1/5 positive	[Albrechtsen, 2002]

Table 9: *Salmonella sp.*

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Virgin Islands</td><td>30 cisterns tested
Found in “many” cisterns</td><td>[Rinehart et al., 1985]</td></tr>
<tr><td>Thailand</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers
<table>
<tr><td></td><td>% not meeting std (0 cell/100 ml)</td></tr>
<tr><td>Roof and gutter</td><td>1/365 samples</td></tr>
<tr><td>Tank and jar</td><td>1/186 samples</td></tr>
<tr><td>In-house</td><td>0/99 samples</td></tr>
</table></td><td>[Wirojanagud, 1991]</td></tr>
<tr><td>Hawaii</td><td>One-time sampling of 11 cisterns
All samples negative using normal techniques but 2/11 +ve using very sensitive techniques (∴ low concentrations)</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>Victoria</td><td>4 concrete and 2 GI tanks. 11-14 samples over 15 months for each type
ND</td><td>[Thurman, 1995]</td></tr>
<tr><td>Germany</td><td>798 samples from 93 DRWH systems
0.13% samples positive (one only)</td><td>[Holländer et al., 1996]</td></tr>
<tr><td>Pretoria</td><td>Comparative study
<table>
<tr><td>Rainwater</td><td>19 - 200</td></tr>
<tr><td>River</td><td>7 000 – 16 000</td></tr>
<tr><td>Borehole</td><td>20 - 250</td></tr>
<tr><td>Well</td><td>50 - 825</td></tr>
</table></td><td>[Nevondo & Cloete, 1999]</td></tr>
<tr><td>New Zealand</td><td>115 household tanks sampled
0.9% samples positive</td><td>[Simmons et al., 2001]</td></tr>
</table>

Table 10: Other biological contamination

<table>
<tr><th>Location</th><th>Type</th><th>Notes</th><th>Source</th></tr>
<tr><td>Virgin Islands</td><td>Aerobacter</td><td>30 cisterns tested
“Found to be present in cisterns”</td><td>[Rinehart et al., 1985]</td></tr>
<tr><td>Thailand</td><td>Aeromonas Sp.</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers
<table>
<tr><td></td><td>% not meeting std (0 cell/100 ml)</td></tr>
<tr><td>Roof and gutter</td><td>0/365 samples</td></tr>
<tr><td>Tank and jar</td><td>1/186 samples</td></tr>
<tr><td>In-house</td><td>0/99 samples</td></tr>
</table></td><td>[Wirojanagud, 1991]</td></tr>
<tr><td>New Zealand</td><td>Aeromonas Sp.</td><td>125 household tanks sampled
16% samples positive
Correlated with indicators (P 0.007- 0.035)
Correlated with gastrointestinal symptoms (P: 0.021)
Correlated with tiled roofs (P: 0.020)</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>Denmark</td><td>Aeromonas Sp.</td><td>2-4 Samples taken from 14 DRWH systems over a period of 6 months. Samples from tank and toilet bowl
2/14 positive, range: <10-30 cells/ml
Toilet (RW)t: 3/7 positive, range: 10-4 400
Toilet (DW): 5/5 positive, range 10-8 800</td><td>[Albrechtsen, 2002]</td></tr>
<tr><td>Hawaii</td><td>Clostridium Perfringens</td><td>One-time sampling of 11 cisterns
8/11 samples +ve (range 0 – 5; mean 2 cfu/100ml)</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>Germany</td><td>Entrrococci</td><td>969 samples from 102 DRWH systems
25.2%>100 cfy/100ml, 7.8%>1 000, 0.5%>10 000</td><td>[Holländer et al., 1996]</td></tr>
<tr><td>Hawaii</td><td>F Strep</td><td>One-time sampling of 9 cisterns about 6” below surface and from HH tap.
Tank: 8/9 positive (range 0 – 1 400; mean 392 CFU/100ml)
HH; 8/9 positive (range 0 – 260; mean 53 CFU/100ml)</td><td>[Fujioka et al., 1991]</td></tr>
</table>

Location	Type	Notes	Source
Queensland	*Mycobacterium Sp.*	516 samples from 205 tanks taken over 2 years from GI tanks in three locations Toowoomba – 7/32 positive Rockhampton – 9/32 positive Fitzroy River basin – 46/141 positive Few tanks were positive on more than one occasion Indications are that *M. itracellulare* can survive in tank water No evidence of correlation with illness found (or specifically looked for)	[Tuffley & Holbeche, 1980]
Denmark	*Mycobactreium avium*	2-4 Samples taken from 14 DRWH systems over a period of 6 months. Samples from tank and toilet bowl 1/14 positive Toilet (RW)t: 0/7 positive Toilet (DW): 0/1 positive	[Albrechtsen, 2002]
Virgin Islands	*Pseudomonas*	30 cisterns tested “Found to be present in cisterns”	[Rinehart et al., 1985]
Virgin Islands	*Pseudomonas Aeruginosa*	Bi-weakly sampling of 20 homes – 271 private home and 75 public housing samples taken Private homes, 70% >1 Public housing; 44% >1 Poor correlation with Total Coliforms (15% PA+ve, TC <1)	[Krishna, 1989]
Germany	*Pseudomonas Aeruginosa*	710 samples from 104 DRWH systems 11.2% samples positive	[Holländer et al., 1996]
Denmark	*Pseudomonas Aeruginosa*	2-4 Samples taken from 14 DRWH systems over a period of 6 months. Samples from tank and toilet bowl Tank: 1/14 positive, range: <1-20 CFU/100ml Toilet (RW)t: 2/7 positive, range: 1-870 Toilet (DW): 0/5 positive	[Albrechtsen, 2002]
NSW	*Pseudomonas*	Three sites surveyed over a period of time. Average values quoted Site 1: Roof: 59 600, Outlet: 7 544, tank surface: 6 768 Site 2: Roof: 5 9600, Outlet: 1 673 Site 2: Outlet: 412, tank surface: 3 100	[Coombes et al., 2005]
Virgin Islands	*Serratia*	30 cisterns tested “Found to be present in cisterns”	[Rinehart et al., 1985]
Victoria	*Shigella*	4 concrete and 2 GI tanks. 11-14 samples over 15 months for each type Not detected	[Thurman, 1995]
Germany	*Staphylococcus aureus*	782 samples from 79 DRWH systems No samples positive	[Holländer et al., 1996]
Thailand	*Vibrio parahacmolyticus*	One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers % not meeting std (0 cell/100 ml) Roof and gutter 0/365 samples Tank and jar 0/186 samples In-house 1/99 samples	[Wirojanagud, 1991]
Germany	Yeasts	428 samples from 38 DRWH systems No samples positive	[Holländer et al., 1996]
Germany	*Yersinia*	338 samples from 34 DRWH systems No samples positive	[Holländer et al., 1996]

MINERALS

Table 11: Aluminium (WHO recommendation: 0.2mg/l in small supplies)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Texas</td><td>31 samples from 4 roof types and rain in rural Texas. Local industry includes paper mills, fertiliser and animal feed production. Houston is 240km south – predominant wind is southerly. Samples were not normally distributed. Some samples may be first-flush due to apparatus being allowed to overflow
Medians (mg/l):
<table>
<tr><td>Rain</td><td>Range: 0.008 – 2.047
Median: 0.251</td></tr>
<tr><td>Wood shingle</td><td>Range: 0.008 – 2.343
Median: 0.224</td></tr>
<tr><td>Composite shingle</td><td>Range: 0.008 – 6.736
Median: 0.181</td></tr>
<tr><td>Aluminium</td><td>Range: 0.008 – 4.077
Median: 0.169</td></tr>
<tr><td>GI</td><td>Range: 0.008 – 6.884
Median: 0.194</td></tr>
</table></td><td>[Chang et al., 2004]</td></tr>
<tr><td>Pretoria</td><td>Comparative study (mg/l)
<table>
<tr><td>Rainwater</td><td>0.014</td></tr>
<tr><td>River</td><td>0.029</td></tr>
<tr><td>Borehole</td><td>0.016</td></tr>
<tr><td>Well</td><td>0.026</td></tr>
</table></td><td>[Nevondo & Cloete, 1999]</td></tr>
</table>

Table 12: Ammonia (WHO recommendation: 1.5 mg/l)

Location	Notes	Source
NSW	Three sites surveyed over a period of time average values quoted (mg/l) Site 1: Rain:0.3 , Roof:0.4, Outlet:0.3, Tank surface:0.1 Site 2: Roof:0.4, Outlet:0.3	[Coombes et al., 2005]

Table 13: Arsenic (WHO recommendation: 0.01 mg/l)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>New Zealand</td><td>mg/l
<0.005-0.019 mg/l
%iles: 25th <0.005,Median <0.005, 75th 0.006, 95th 0.009
One high level reported from a house with tanalised timber as part of the system</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>Texas</td><td>Two types of roof tested (asphalt and GI) with two levels of proximity to a busy freeway (12m and 102m)
<table>
<tr><td>Near road (μg/l)</td><td>Away from road (μg/l)</td></tr>
<tr><td>Asp: 7.8, 7.8, 7.2
GI: 6.6, 9.3, 7.3</td><td>Asp:7.9, 9.1, 7.6
GI: 9.0, 11, 8.9</td></tr>
</table></td><td>[Van Metre & Mahler, 2003]</td></tr>
</table>

Table 14: Cadmium (WHO recommendation: 0.003 mg/l)

Location	Notes	Source
St Maarten	Study of atmospheric deposition, tank and distribution of 46 DRWH systems (μg/l) Rainfall; 0.52-1.06, mean: 0. Bottom: <0.02-157, median 0.99 Surface: <0.02-0.40, median 0.03 Tap: <0.02-30.2, median 0.12 Tank outlets ~15-25cm from floor Dissolved metals removed by increased pH and calcium (from cement tanks) Heightened levels in distribution systems are assumed to come from GI plumbing	[Gumbs & Dierberg, 1985]

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Kentucky, Tennessee, St Maarten island</td><td>Comparative study of three sites, (two with acid rain one without). 25 HH from Kentucky and Tennessee (acid rain) and 25 HH in St Maarten (no acid rain). Bulk deposition sampled near one HH in each region. roof runoff from one household in Kentucky. Sample from 0.5m below cistern surface and at cistern bottom (inc sediment) and tap water sampled at all sample sites
<table>
<tr><td>(μg/l)</td><td>St Maarten</td><td>Kentucky/ Tennessee</td></tr>
<tr><td>Bulk (mean)</td><td>0.17</td><td>0.44</td></tr>
<tr><td>Cistern surface (median)</td><td>0.10</td><td>0.19</td></tr>
</table>
Increased at roof runoff, decreased with storage, increased in plumbing. Sediment variable but “very high” cistern process assumed to be precipitation, sorption & sedimentation due to increased pH in tank</td><td>[Olem & Berthouex, 1989]</td></tr>
<tr><td>Germany</td><td>Study of runoff from 5 roof types for two storms (nmol/l)
<table>
<tr><td>Rainfall</td><td>10.7, 2.6</td></tr>
<tr><td>Tar felt</td><td>9.8, 1.8</td></tr>
<tr><td>Clay pantiles</td><td>5.6, 2.0</td></tr>
<tr><td>Asbestos cement</td><td>0.5, 2.0</td></tr>
<tr><td>Zinc sheet</td><td>15.5, 6.8</td></tr>
<tr><td>Gravel</td><td>0.9, 0.4</td></tr>
</table></td><td>[Quek & Forster, 1993]</td></tr>
<tr><td>Ohio</td><td>40 systems with concrete tanks in rural area sampled five times over 2 years. Each from the cold water tap, just below the water surface in the tank, Further samples were taken from 15cm from the bottom and from the sediment
Sediment exceeded Drinking water limits in 15% of systems; Particulate filters reduced levels (μg/l)
Cistern water all concentrations below detection limit (1 μg/l)
Tap water: 0 samples >10, 25% > detection limit</td><td>[Sharpe & Young, 1982]</td></tr>
<tr><td>Texas</td><td>Two types of roof tested (asphalt and GI) with two levels of proximity to a busy freeway (12m and 102m)
<table>
<tr><td>Near road (μg/l)</td><td>Away from road (μg/l)</td></tr>
<tr><td>Asp: 4.6, 3.3, 4.3
GI: 7.6, 6.3, 7.2</td><td>Asp: 3.6, 1.9, 2.5
GI: 5.5, 4.7, 7.4</td></tr>
</table></td><td>[Van Metre & Mahler, 2003]</td></tr>
<tr><td>Thailand</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers
All locations met WHO standard</td><td>[Wirojanagud, 1991]</td></tr>
<tr><td>Victoria</td><td>22 tanks, 1-3 samples per tank in rural area
<0.0005 mg/l</td><td>[Bannister et al., 1997]</td></tr>
</table>

Table 15: Calcium (WHO recommendation: 500 mg/l – aesthetic)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>NSW</td><td>Three sites surveyed over a period of time average values quoted (mg/l)
Site 1: Rain:2, Roof:2.7, Outlet:8.4, Tank surface:6.9
Site 2: Roof:2.7, Outlet:2.5</td><td>[Coombes et al., 2005]</td></tr>
<tr><td>St Maarten</td><td>Study of atmospheric deposition, tank and distribution of 46 DRWH systems (mg/l)
Rainfall; 0.8-2.8, mean: 1.9
Surface: 1.8-34.3, mean 11.4
Dissolved metals removed by increased pH and calcium (from cement tanks)</td><td>[Gumbs & Dierberg, 1985]</td></tr>
<tr><td>Kentucky, Tennessee, St Maarten island</td><td>Comparative study of three sites, (two with acid rain one without). 25 HH from Kentucky and Tennessee (acid rain) and 25 HH in St Maarten (no acid rain). Bulk deposition sampled near one HH in each region. roof runoff from one household in Kentucky. Sample from 0.5m below cistern surface and at cistern bottom (inc sediment) and tap water sampled at all sample sites
<table>
<tr><td>(μg/l)</td><td>St Maarten</td><td>Kentucky/ Tennessee</td></tr>
<tr><td>Bulk (mean)</td><td>49.5</td><td>50.9</td></tr>
<tr><td>Cistern surface (median)</td><td>447</td><td>775</td></tr>
</table>
Raised at roof runoff, raised further with storage, then stabilised, Sediment not tested.</td><td>[Olem & Berthouex, 1989]</td></tr>
</table>

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Germany</td><td>Study of runoff from 5 roof types for two storms (μmol/l)
<table>
<tr><td>Rainfall</td><td>86, 42</td></tr>
<tr><td>Tar felt</td><td>131, 153</td></tr>
<tr><td>Clay pantiles</td><td>222, 113</td></tr>
<tr><td>Asbestos cement</td><td>1002, 1047</td></tr>
<tr><td>Zinc sheet</td><td>115, 113</td></tr>
<tr><td>Gravel</td><td>830, 613</td></tr>
</table>
Increased levels from asbestos due to leaching from cement</td><td>[Quek & Forster, 1993]</td></tr>
</table>

Table 16: Chloride (WHO recommendation: 250 mg/l – aesthetic)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>NSW</td><td>Three sites surveyed over a period of time average values quoted (mg/l)
Site 1: Rain:7.5, Roof:14.9, Outlet:10.5, Tank surface:7.1
Site 2: Roof:14.9, Outlet:9.9</td><td>[Coombes et al., 2005]</td></tr>
<tr><td>St Maarten</td><td>Study of atmospheric deposition, tank and distribution of 46 DRWH systems (mg/l)
Rainfall; 8.2-13.6, mean: 11.3
Surface: 5.0-18.9, mean 9.2
Dissolved metals removed by increased pH and calcium (from cement tanks)</td><td>[Gumbs & Dierberg, 1985]</td></tr>
<tr><td>Zambia</td><td>1 school and 4 HHs in urban Zambia tested for WQ at rain, roof and tank. For comparison several other water supply options were also sampled
(mg/l)
<table>
<tr><td>Rain</td><td>1.0-3.0</td></tr>
<tr><td>FF system (GI)</td><td>0-5.0</td></tr>
<tr><td>FF system (asb)</td><td>0-9.0</td></tr>
<tr><td>Tank</td><td>0-17</td></tr>
<tr><td>Piped water</td><td>3.9-12</td></tr>
<tr><td>Private boreholes</td><td>10-18</td></tr>
<tr><td>Shallow wells</td><td>10-29</td></tr>
</table></td><td>[Handia, 2005]</td></tr>
<tr><td>Victoria</td><td>4 concrete and 2 GI tanks. 11-14 samples over 15 months for each type
Below detection limits</td><td>[Thurman, 1995]</td></tr>
</table>

Table 17: Chromium (WHO recommendation: 0.05 mg/l)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Victoria</td><td>22 tanks, 1-3 samples per tank in rural area
<0.003 mg/l</td><td>[Bannister et al., 1997]</td></tr>
<tr><td>St Maarten</td><td>Study of atmospheric deposition, tank and distribution of 46 DRWH systems (μg/l)
Rainfall; 0.8-5.7, mean: 2.4
Bottom: 0.4-453, median 3.3
Surface: <0.04-13.4, median 0.4
Tap: <0.04-13.2, median 0.4
Tank outlets ~15-25cm from floor
Dissolved metals removed by increased pH and calcium (from cement tanks)
Heightened levels in distribution systems are assumed to come from GI plumbing</td><td>[Gumbs & Dierberg, 1985]</td></tr>
<tr><td>Texas</td><td>Two types of roof tested (asphalt and GI) with two levels of proximity to a busy freeway (12m and 102m)
<table>
<tr><th>Near road (μg/l)</th><th>Away from road (μg/l)</th></tr>
<tr><td>Asp: 66, 67, 58
GI: 67, 80, 57</td><td>Asp:60, 56, 54
GI: 68, 57, 56</td></tr>
</table></td><td>[Van Metre & Mahler, 2003]</td></tr>
<tr><td>Thailand</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers
All locations met WHO standard</td><td>[Wirojanagud, 1991]</td></tr>
</table>

Table 18: Copper (WHO recommendation: 2 mg/l)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Victoria</td><td>22 tanks, 1-3 samples per tank in rural area (mg/l)
0.001-0.27, median: 0.011</td><td>[Bannister et al., 1997]</td></tr>
<tr><td>Texas</td><td>31 samples from 4 roof types and rain in rural Texas. Local industry includes paper mills, fertiliser and animal feed production. Houston is 240km south – predominant wind is southerly. Samples were not normally distributed. Some samples may be first-flush due to apparatus being allowed to overflow
Medians (mg/l):
<table>
<tr><td>Rain</td><td>Range: 0.001 – 0.174
Median: 0.021</td></tr>
<tr><td>Wood shingle</td><td>Range: 0.001 – 5.410
Median: 0.022</td></tr>
<tr><td>Composite shingle</td><td>Range: 0.001 – 0.126
Median: 0.018</td></tr>
<tr><td>Aluminium</td><td>Range: 0.001 – 0.248
Median: 0.020</td></tr>
<tr><td>GI</td><td>Range: 0.001 – 0.224
Median: 0.020</td></tr>
</table>
High values from shingle thought to be due to leaching of preservatives</td><td>[Chang et al., 2004]</td></tr>
<tr><td>Kentucky, Tennessee, St Maarten island</td><td>Comparative study of three sites, (two with acid rain one without). 25 HH from Kentucky and Tennessee (acid rain) and 25 HH in St Maarten (no acid rain). Bulk deposition sampled near one HH in each region. roof runoff from one household in Kentucky. Sample from 0.5m below cistern surface and at cistern bottom (inc sediment) and tap water sampled at all sample sites
<table>
<tr><td></td><td>St Maarten (μg/l)</td><td>Kentucky/ Tennessee (μg/l)</td></tr>
<tr><td>Bulk (mean)</td><td>9</td><td>19</td></tr>
<tr><td>Cistern surface (median)</td><td>Too low to detect</td><td>Too low to detect</td></tr>
</table>
Below detection limits for most measurements but sediment up to 240 μg/l cistern process assumed to be precipitation, sorption & sedimentation due to increased pH in tank</td><td>[Olem & Berthouex, 1989]</td></tr>
<tr><td>Germany</td><td>Study of runoff from 5 roof types for two storms
(mg/l)
<table>
<tr><td>Rainfall μmol/l</td><td>0.29, 0.10</td></tr>
<tr><td>Tar felt</td><td>0.12, 0.12</td></tr>
<tr><td>Clay pantiles</td><td>7.48, 3.70</td></tr>
<tr><td>Asbestos cement</td><td>0.20, 0.13</td></tr>
<tr><td>Zinc sheet</td><td>0.51, 0.32</td></tr>
<tr><td>Gravel</td><td>0.03, 0.12</td></tr>
</table>
Copper concentration on pantiles due to copper flashing, many roofs acted as a sink</td><td>[Quek & Forster, 1993]</td></tr>
<tr><td>New Zealand</td><td><0.002-4.5 mg/l
%iles: 25^{th} 0.01,Median 0.06, 75^{th} 0.29, 95^{th} 1.17
High levels associated with copper piping from tank to faucet and with one location</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>Texas</td><td>Two types of roof tested (asphalt and GI) with two levels of proximity to a busy freeway (12m and 102m)
<table>
<tr><td>Near road (μg/l)</td><td>Away from road (μg/l)</td></tr>
<tr><td>Asp: 98, 88, 84
GI: 99, 110, 96</td><td>Asp:89, 50, 53
GI: 85, 59, 77</td></tr>
</table></td><td>[Van Metre & Mahler, 2003]</td></tr>
<tr><td>Thailand</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers
All locations met WHO standard</td><td>[Wirojanagud, 1991]</td></tr>
</table>

Table 19: Iron (WHO recommendation: 0.3 mg/l – aesthetic)

Location	Notes	Source
Victoria	22 tanks, 1-3 samples per tank in rural area (mg/l) 0.01-3 median, .11	[Bannister et al., 1997]

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>NSW</td><td>Three sites surveyed over a period of time average values quoted (mg/l)
Site 1: Rain:<0.01, Roof:0.05, Outlet:0.01, Tank surface:0.01
Site 2: Roof:0.05, Outlet:<0.05</td><td>[Coombes et al., 2005]</td></tr>
<tr><td>Pretoria</td><td>Comparative study (mg/l)<table><tr><td>Rainwater</td><td>0.013</td></tr><tr><td>River</td><td>0.038</td></tr><tr><td>Borehole</td><td>0.049</td></tr><tr><td>Well</td><td>0.037</td></tr></table></td><td>[Nevondo & Cloete, 1999]</td></tr>
<tr><td>Victoria</td><td>4 concrete and 2 GI tanks. 11-14 samples over 15 months for each type
Below detection limits</td><td>[Thurman, 1995]</td></tr>
<tr><td>Thailand</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers
All locations met WHO standard</td><td>[Wirojanagud, 1991]</td></tr>
</table>

Table 20: Flouride (WHO recommendation: 1.5 mg/l)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Pretoria</td><td>Comparative study (mg/l)<table><tr><td>Rainwater</td><td>0.270</td></tr><tr><td>River</td><td>0.455</td></tr><tr><td>Borehole</td><td>0.371</td></tr><tr><td>Well</td><td>0.366</td></tr></table></td><td>[Nevondo & Cloete, 1999]</td></tr>
<tr><td>Zambia</td><td>1 school in urban Zambia. For comparison several other water supply options were also sampled (mg/l)<table><tr><td>RW Tank</td><td>0.16</td></tr><tr><td>Piped water</td><td>0.12-0.32</td></tr><tr><td>Private boreholes</td><td>0.25-4.8</td></tr><tr><td>Shallow wells</td><td>0.3-4.7</td></tr></table></td><td>[Handia, 2005]</td></tr>
</table>

Table 21: Lead (WHO recommendation: 0.01 mg/l)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Victoria</td><td>22 tanks, 1-3 samples per tank in rural area
0.002-0.073, median 0.007 (mg/l)
Correlated and rise after long periods without rain. Postulated that levels are due to lead-based solders in GI tanks</td><td>[Bannister et al., 1997]</td></tr>
<tr><td>Texas</td><td>31 samples from 4 roof types and rain in rural Texas. Local industry includes paper mills, fertiliser and animal feed production. Houston is 240km south – predominant wind is southerly. Samples were not normally distributed. Some samples may be first-flush due to apparatus being allowed to overflow
Medians (mg/l):<table><tr><td>Rain</td><td>Range: 0.025 – 0.116
Median: 0.025</td></tr><tr><td>Wood shingle</td><td>Range: 0.025 – 0.700
Median: 0.025</td></tr><tr><td>Composite shingle</td><td>Range: 0.025 – 0.203
Median: 0.025</td></tr><tr><td>Aluminium</td><td>Range: 0.025 – 0.134
Median: 0.025</td></tr><tr><td>GI</td><td>Range: 0.025 – 0.255
Median: 0.025</td></tr></table></td><td>[Chang et al., 2004]</td></tr>
<tr><td>NSW</td><td>Three sites surveyed over a period of time average values quoted
Site 1: Rain:<0.01 mg/l, Roof:0.014, Outlet:<0.01, Tank surface:<0.01
Site 2: Roof:0.01, Outlet:<0.01</td><td>[Coombes et al., 2005]</td></tr>
</table>

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>St Maarten</td><td>Study of atmospheric deposition, tank and distribution of 46 DRWH systems (μg/l)
Rainfall; 5.8-15.9, mean: 9.9
Bottom: 0.3-3 840, median 19.4
Surface: 0.1-75.1, median 0.9
Tap: <0.2-70.0, median 2.1
Tank outlets ~15-25cm from floor
Dissolved metals removed by increased pH and calcium (from cement tanks)
Heightened levels in distribution systems are assumed to come from GI plumbing</td><td>[Gumbs & Dierberg, 1985]</td></tr>
<tr><td>Zambia</td><td>1 school and 4 HHs in urban Zambia tested for WQ at rain, roof and tank. For comparison several other water supply options were also sampled (mg/l)
<table>
<tr><td>Rain</td><td><0.01</td></tr>
<tr><td>FF system (GI)</td><td><0.001-0.01</td></tr>
<tr><td>FF system (asb)</td><td><0.001-0.27</td></tr>
<tr><td>Tank</td><td><0.001-14</td></tr>
<tr><td>Piped water</td><td>-</td></tr>
<tr><td>Private boreholes</td><td><0.001</td></tr>
<tr><td>Shallow wells</td><td>-</td></tr>
</table>
High levels in the tank thought to be from lead solder left in gutter – but not corroborated by FF results – may be from accumulated material in bottom of tank and too low an outlet.</td><td>[Handia, 2005]</td></tr>
<tr><td>Kentucky, Tennessee, St Maarten island</td><td>Comparative study of three sites, (two with acid rain one without). 25 HH from Kentucky and Tennessee (acid rain) and 25 HH in St Maarten (no acid rain). Bulk deposition sampled near one HH in each region. roof runoff from one household in Kentucky. Sample from 0.5m below cistern surface and at cistern bottom (inc sediment) and tap water sampled at all sample sites
<table>
<tr><th></th><th>St Maarten (μg/l)</th><th>Kentucky/ Tennessee (μg/l)</th></tr>
<tr><td>Bulk (mean)</td><td>427</td><td>73</td></tr>
<tr><td>Cistern surface (median)</td><td><1</td><td>2.8</td></tr>
</table>
High St Maarten presumed to be due to many cars using leaded fuel.
Decreased at roof runoff, decreased with storage, increased in plumbing, particularly when left to stand. Sediment variable but “very high” cistern process assumed to be precipitation, sorption & sedimentation due to increased pH in tank</td><td>[Olem & Berthouex, 1989]</td></tr>
<tr><td>Germany</td><td>Study of runoff from 5 roof types for two storms (nmol/l)
<table>
<tr><td>Rainfall</td><td>162, 116</td></tr>
<tr><td>Tar felt</td><td>187, 164</td></tr>
<tr><td>Clay pantiles</td><td>73, 299</td></tr>
<tr><td>Asbestos cement</td><td>38, 199</td></tr>
<tr><td>Zinc sheet</td><td>85, 278</td></tr>
<tr><td>Gravel</td><td>6, 22</td></tr>
</table></td><td>[Quek & Forster, 1993]</td></tr>
<tr><td>Ohio</td><td>40 systems with concrete tanks in rural area sampled five times over 2 years. Each from the cold water tap, just below the water surface in the tank, Further samples were taken from 15cm from the bottom and from the sediment
Sediment exceeded Drinking water limits in 55%, 42% and 59% (3 samples) of cases; Particulate filters reduced levels (mean – 348 μg/l without, 32 μg/l with)
Corrosion of $CaCO_3$ presumed to cause lead to precipitate
Cistern water: max – 26 μg/l only two samples over detection limit
Tap water: 15% samples >50 μg/l, 25% > detection limit. Water left overnight had 22% > limit. Plastic plumbing substantially reduced concentrations</td><td>[Sharpe & Young, 1982]</td></tr>
<tr><td>New Zealand</td><td><0.003-0.14 mg/l
%iles: 25^{th} <0.003,Median <0.01, 75^{th} <0.01, 95^{th} 0.02
Correlated with lead in the catchment or conveyance (roof, flashing, guttering, downspout) (*P* 0.019) but not just with flashing or lead solder
Correlated with low (<7) pH (OR 2.09 and 3.95 for pH<6.5)</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>Texas</td><td>Two types of roof tested (asphalt and GI) with two levels of proximity to a busy freeway (12m and 102m)
<table>
<tr><th>Near road (μg/l)</th><th>Away from road (μg/l)</th></tr>
<tr><td>Asp: 160, 300, 390
GI: 120, 130, 120</td><td>Asp:370, 300, 220
GI: 170, 130, 150</td></tr>
</table></td><td>[Van Metre & Mahler, 2003]</td></tr>
</table>

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Thailand</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers
All locations met WHO standard</td><td>[Wirojanagud, 1991]</td></tr>
<tr><td>Queensland</td><td>Rain collected from roofs. GI and concrete tile roofs. Rural, industrial and urban (6 in all)8 samples from each catchment over 5 months
“Averages”
<table>
<tr><td>Rural</td><td>Urban (mg/l)</td><td>Industrial (mg/l)</td></tr>
<tr><td>GI: 0 Con: 0</td><td>GI: 0.012 Con: 0.05</td><td>GI: 0.1, Con: 0.09</td></tr>
</table>
Correlated with preceding dry days</td><td>[Thomas & Greene, 1993]</td></tr>
</table>

Table 22: Magnesium (WHO recommendation: none)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Victoria</td><td>22 tanks, 1-3 samples per tank in rural area (mg/l)
0.002-0.15 median: 0.016</td><td>[Bannister et al., 1997]</td></tr>
<tr><td>Texas</td><td>31 samples from 4 roof types and rain in rural Texas. Local industry includes paper mills, fertiliser and animal feed production. Houston is 240km south – predominant wind is southerly. Samples were not normally distributed. Some samples may be first-flush due to apparatus being allowed to overflow
Medians (mg/l):
<table>
<tr><td>Rain</td><td>Range: 0.053 – 4.739
Median: 0.487</td></tr>
<tr><td>Wood shingle</td><td>Range: 0.082 – 6.680
Median: 0.646</td></tr>
<tr><td>Composite shingle</td><td>Range: 0.023 – 5.063
Median: 0.368</td></tr>
<tr><td>Aluminium</td><td>Range: 0.004 – 1.478
Median: 0.292</td></tr>
<tr><td>GI</td><td>Range: 0.001 – 3.659
Median: 0.246</td></tr>
</table></td><td>[Chang et al., 2004]</td></tr>
<tr><td>St Maarten</td><td>Study of atmospheric deposition, tank and distribution of 46 DRWH systems
Rainfall: 0.91-1.65, mean: 1.30 mg/l
Surface: 0.19-3.20, mean 0.59 mg/l
Dissolved metals removed by increased pH and calcium (from cement tanks)</td><td>[Gumbs & Dierberg, 1985]</td></tr>
</table>

Table 23: Manganese (WHO recommendation: 0.4 mg/l)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Pretoria</td><td>Comparative study (mg/l)
<table>
<tr><td>Rainwater</td><td>0.019</td></tr>
<tr><td>River</td><td>0.054</td></tr>
<tr><td>Borehole</td><td>0.018</td></tr>
<tr><td>Well</td><td>0.043</td></tr>
</table></td><td>[Nevondo & Cloete, 1999]</td></tr>
<tr><td>Thailand</td><td>One-time event with 19 sample sets from DRWH systems in 3 villages. Each set sampled Roof tank and in-house containers (mg/l)
<table>
<tr><td></td><td>% not meeting std</td></tr>
<tr><td>Roof and gutter</td><td>20%</td></tr>
<tr><td>Tank and jar</td><td>0%</td></tr>
<tr><td>In-house</td><td>2%</td></tr>
</table>
Main source of contamination thought to be the roof-gutter system via leaching. Lower concentrations in upper layers of stored water than lower layers though to indicate settling via adsorption or precipitation. Some recontamination is also evident</td><td>[Wirojanagud, 1991]</td></tr>
</table>

Table 24: Nickel (WHO recommendation: 0.02 mg/l)

Location	Notes	Source
Victoria	22 tanks, 1-3 samples per tank in rural area (mg/l) 0.002-0.013, median: 0.005	[Bannister et al., 1997]

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Texas</td><td>Two types of roof tested (asphalt and GI) with two levels of proximity to a busy freeway (12m and 102m)
<table>
<tr><td>Near road (μg/l)</td><td>Away from road (μg/l)</td></tr>
<tr><td>Asp: 32, 29, 28
GI: 36, 55, 31</td><td>Asp:69, 30, 30
GI: 43, 45, 36</td></tr>
</table></td><td>[Van Metre & Mahler, 2003]</td></tr>
</table>

Table 25: Zinc (WHO recommendation: 3 mg/l aesthetic)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Victoria</td><td>22 tanks, 1-3 samples per tank in rural area (mg/l)
0-17, median 2.1
Correlated and rise after long periods without rain. Postulated that levels are due to lead-based solders in GI tanks
Not correlated with GI roofs</td><td>[Bannister et al., 1997]</td></tr>
<tr><td>Texas</td><td>31 samples from 4 roof types and rain in rural Texas. Local industry includes paper mills, fertiliser and animal feed production. Houston is 240km south – predominant wind is southerly. Samples were not normally distributed. Some samples may be first-flush due to apparatus being allowed to overflow
Medians (mg/l):
<table>
<tr><td>Rain</td><td>Range: 0.003 – 0.978
Median: 0.085</td></tr>
<tr><td>Wood shingle</td><td>Range: 0.039 – 109.7
Median: 9.717</td></tr>
<tr><td>Composite shingle</td><td>Range: 0.043 – 13.59
Median: 2.248</td></tr>
<tr><td>Aluminium</td><td>Range: 0.514 – 16.60
Median: 0.2.248</td></tr>
<tr><td>GI</td><td>Range: 0.124 – 212.3
Median: 8.219</td></tr>
</table>
High values from shingle thought to be due to leaching of preservatives</td><td>[Chang et al., 2004]</td></tr>
<tr><td>NSW</td><td>Three sites surveyed over a period of time average values quoted (mg/l)
Site 1: no data
Site 2: Outlet:3.9</td><td>[Coombes et al., 2005]</td></tr>
<tr><td>St Maarten</td><td>Study of atmospheric deposition, tank and distribution of 46 DRWH systems (μg/l)
Rainfall; 269-534, mean: 400
Bottom: 16-375 000, median 1080
Surface: 2-1160, median 84
Tap: 0.2-70, median 21
Tank outlets ~15-25cm from floor
Dissolved metals removed by increased pH and calcium (from cement tanks)
Heightened levels in distribution systems are assumed to come from GI plumbing</td><td>[Gumbs & Dierberg, 1985]</td></tr>
<tr><td>Zambia</td><td>1 school and 4 HHs in urban Zambia tested for WQ at rain, roof and tank. For comparison several other water supply options were also sampled
(mg/l)
<table>
<tr><td>Rain</td><td><0.001</td></tr>
<tr><td>FF system (GI)</td><td>0.14-3.16</td></tr>
<tr><td>FF system (asb)</td><td><0.001-0.025</td></tr>
<tr><td>Tank</td><td><0.001-0.96</td></tr>
<tr><td>Piped water</td><td>-</td></tr>
<tr><td>Private boreholes</td><td><0.001</td></tr>
<tr><td>Shallow wells</td><td>-</td></tr>
</table></td><td>[Handia, 2005]</td></tr>
</table>

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Kentucky, Tennessee, St Maarten island</td><td>Comparative study of three sites, (two with acid rain one without). 25 HH from Kentucky and Tennessee (acid rain) and 25 HH in St Maarten (no acid rain). Bulk deposition sampled near one HH in each region. roof runoff from one household in Kentucky. Sample from 0.5m below cistern surface and at cistern bottom (inc sediment) and tap water sampled at all sample sites
<table>
<tr><th></th><th>St Maarten (μg/l)</th><th>Kentucky/ Tennessee (μg/l)</th></tr>
<tr><td>Bulk (mean)</td><td>624</td><td>310</td></tr>
<tr><td>Cistern surface (median)</td><td>90</td><td>60</td></tr>
</table>
Similar at roof runoff, decreased with storage, increased in plumbing, particularly when left to stand. Sediment variable but “very high” cistern process assumed to be precipitation, sorption & sedimentation due to increased pH in tank</td><td>[Olem & Berthouex, 1989]</td></tr>
<tr><td>Germany</td><td>Study of runoff from 5 roof types for two storms (μmol/l)
<table>
<tr><td>Rainfall</td><td>1.61, 1.01</td></tr>
<tr><td>Tar felt</td><td>1.72, 1.56</td></tr>
<tr><td>Clay pantiles</td><td>0.89, 0.75</td></tr>
<tr><td>Asbestos cement</td><td>0.18, 0.51</td></tr>
<tr><td>Zinc sheet</td><td>664, 672</td></tr>
<tr><td>Gravel</td><td>145, 134</td></tr>
</table>
Elevated levels in zinc sheet due to leaching and gravel due to leaching of zinc flashing. Most roofs acted as a sink</td><td>[Quek & Forster, 1993]</td></tr>
<tr><td>New Zealand</td><td><0.0005-3.2 mg/l
%iles: 25th 0.14,Median 0.4, 75th 0.99, 95th 1.90</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>Texas</td><td>Two types of roof tested (asphalt and GI) with two levels of proximity to a busy freeway (12m and 102m)
<table>
<tr><th>Near road (μg/l)</th><th>Away from road (μg/l)</th></tr>
<tr><td>Asp: 1600, 1200, 1900
GI: 3800, 4600, 4500</td><td>Asp:900, 710, 930
GI: 4300, 3600, 6200</td></tr>
</table></td><td>[Van Metre & Mahler, 2003]</td></tr>
<tr><td>Queensland</td><td>Rain collected from roofs. GI and concrete tile roofs. Rural, industrial and urban (6 in all)8 samples from each catchment over 5 months
“Averages”
<table>
<tr><th>Rural (mg/l)</th><th>Urban (mg/l)</th><th>Industrial (mg/l)</th></tr>
<tr><td>GI: 1.3 Con: 0.02</td><td>GI: 1.10 Con: 0.03</td><td>GI: 3.5, Con: 1.6</td></tr>
</table>
Not correlated with preceding dry days</td><td>[Thomas & Greene, 1993]</td></tr>
</table>

Table 26: Other mineral contamination

<table>
<tr><th>Location</th><th>Type</th><th>Notes</th><th>Source</th></tr>
<tr><td>Texas</td><td>Mercury</td><td>Two types of roof tested (asphalt and GI) with two levels of proximity to a busy freeway (12m and 102m)
<table>
<tr><th>Near road (μg/l)</th><th>Away from road (μg/l)</th></tr>
<tr><td>Asp: 0.22, 0.23, 0.16
GI: 0.07, 0.06, 0.08</td><td>Asp:0.22, 0.31, 0.09
GI: 0.18, 0.06, 0.10</td></tr>
</table></td><td>[Van Metre & Mahler, 2003]</td></tr>
<tr><td>Virgin Islands</td><td>Metals</td><td>30 cisterns tested
In acceptable range Copper and iron dominant (copper presumed to be from piping, Iron from cistern walls or plumbing)</td><td>[Rinehart et al., 1985]</td></tr>
</table>

NUTRIENTS

Table 27: Nitrate (WHO recommendation: 50 mg/l for infants)

Location	Notes	Source
NSW	Three sites surveyed over a period of time average values quoted Site 1: Rain:0.2 mg/l, Roof:0.3, Outlet:<0.05, Tank surface:0.06 Site 2: Roof:0.3, Outlet:<0.05	[Coombes et al., 2005]

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Hawaii</td><td>One-time sampling of 5 cisterns about 6” below surface and 8 samples from household tap
<1 mg/l</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>Pretoria</td><td>Comparative study (mg/l)<table>
<tr><td>Rainwater</td><td>2.4</td></tr>
<tr><td>River</td><td>4.45</td></tr>
<tr><td>Borehole</td><td>1.34</td></tr>
<tr><td>Well</td><td>1.94</td></tr>
</table></td><td>[Nevondo & Cloete, 1999]</td></tr>
<tr><td>Victoria</td><td>22 tanks, 1-3 samples per tank in rural area
0.01-1.80 mg/l, median: 0.53
Trees slightly associated with nutrient</td><td>[Bannister et al., 1997]</td></tr>
<tr><td>Zambia</td><td>1 school and 4 HHs in urban Zambia tested for WQ at rain, roof, tank and tap. For comparison several other water supply options were also sampled
(mg/l)<table>
<tr><td>Rain</td><td><0.01-2.9</td></tr>
<tr><td>FF system (GI)</td><td><0.01-26</td></tr>
<tr><td>FF system (asb)</td><td><0.01-1.3</td></tr>
<tr><td>Tank</td><td><0.01-8.8</td></tr>
<tr><td>Piped water</td><td><0.01-2.4</td></tr>
<tr><td>Private boreholes</td><td>0.64-7.9</td></tr>
<tr><td>Shallow wells</td><td>0.56-6.8</td></tr>
</table></td><td>[Handia, 2005]</td></tr>
</table>

Table 28: Nitrite (WHO recommendation: 3 mg/l for infants, 0.2mg/l for long-term exposure)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>NSW</td><td>Three sites surveyed over a period of time average values quoted
Site 1: Rain:0.4mg/l , Roof:2.2, Outlet:1.0, Tank surface:0.6
Site 2: Roof:2.2, Outlet:1.4</td><td>[Coombes et al., 2005]</td></tr>
<tr><td>Pretoria</td><td>Comparative study (mg/l)<table>
<tr><td>Rainwater</td><td>0.006</td></tr>
<tr><td>River</td><td>0.087</td></tr>
<tr><td>Borehole</td><td>0.005</td></tr>
<tr><td>Well</td><td>0.041</td></tr>
</table></td><td>[Nevondo & Cloete, 1999]</td></tr>
<tr><td>Victoria</td><td>4 concrete and 2 GI tanks. 11-14 samples over 15 months for each type
Below detection limits</td><td>[Thurman, 1995]</td></tr>
<tr><td>Queensland</td><td>Rain collected from roofs. GI and concrete tile roofs. Rural, industrial and urban (6 in all) 8 samples from each catchment over 5 months
“Averages”<table>
<tr><td>Rural (mg/l)</td><td>Urban (mg/l)</td><td>Industrial (mg/l)</td></tr>
<tr><td>GI: 0.34 Con: 0.34</td><td>GI: 0.13 Con: 0.15</td><td>GI: 0.14, Con: 0.15</td></tr>
</table>Not correlated with preceding dry days</td><td>[Thomas & Greene, 1993]</td></tr>
</table>

Table 29: Phosphate (WHO recommendation: none)

Location	Notes	Source
Victoria	22 tanks, 1-3 samples per tank in rural area 0.005-0.36 mg/l, 0.028 Trees slightly associated with nutrient	[Bannister et al., 1997]
Singapore	38 samples collected from one rooftop over 2 years Mean: 0.1 mg/l, SD 0.6	[Appan, 1999]

Table 30: Sulphate (WHO recommendation: 25mg/l aesthetic)

Location	Notes	Source
NSW	Three sites surveyed over a period of time average values quoted Site 1: Rain:3.5/l , Roof:6.7, Outlet:7.3, Tank surface:4.9 Site 2: Roof:6.7, Outlet:5.9	[Coombes et al., 2005]

Location	Notes	Source
Zambia	1 school and 4 HHs in urban Zambia tested for WQ at rain, roof, tank and tap. For comparison several other water supply options were also sampled (mg/l) Rain: <0.01-22 FF system (GI): <0.01-2 FF system (asb): <0.01-7.6 Tank: <0.01-78 Piped water: <0.01-51 Private boreholes: 9.3-63 Shallow wells: 20-47.1	[Handia, 2005]
Victoria	4 concrete and 2 GI tanks. 11-14 samples over 15 months for each type Below detection limits	[Thurman, 1995]

Table 31: Other nutrient contamination

Location	Type	Notes	Source
Pretoria	Dissolved oxygen (DO)	Comparative study (mg/l) Rainwater: 3.40 River: 3.83 Borehole: 3.38 Well: 2.80	[Nevondo & Cloete, 1999]
Texas	PAH total	Two types of roof tested (asphalt and GI) with two levels of proximity to a busy freeway (12m and 102m) Near road (μg/l): Asp: 22, 12, 23; GI: 22, 24, 17 Away from road (μg/l): Asp:34, 19, 30; GI: 25, 13, 85	[Van Metre & Mahler, 2003]
Victoria	Pesticides	22 tanks, 1-3 samples per tank in rural area 1/10 samples positive (10%)	[Bannister et al., 1997]

CHEMICAL INDICATORS

Table 32: Colour (WHO recommendation: none)

Location	Notes	Source
Pretoria	Comparative study (hazen) Rainwater: 0.66 River: 8.33 Borehole: 1.16 Well: 5.50	[Nevondo & Cloete, 1999]

Table 33: Conductivity (WHO recommendation: none)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Texas</td><td>31 samples from 4 roof types and rain in rural Texas. Local industry includes paper mills, fertiliser and animal feed production. Houston is 240km south – predominant wind is southerly. Samples were not normally distributed. Some samples may be first-flush due to apparatus being allowed to overflow
Medians (μS/cm):
<table>
<tr><td>Rain</td><td>Range: 7– 79
Median: 19</td></tr>
<tr><td>Wood shingle</td><td>Range: 7 – 232
Median: 28</td></tr>
<tr><td>Composite shingle</td><td>Range: 6 – 179
Median: 22</td></tr>
<tr><td>Aluminium</td><td>Range: 2.2 – 57
Median: 10</td></tr>
<tr><td>GI</td><td>Range: 4 - 172
Median17</td></tr>
</table></td><td>[Chang et al., 2004]</td></tr>
<tr><td>Hawaii</td><td>One-time sampling of 9 cisterns about 6” below.
Range 27 – 125 μmhos/cm</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>Hawaii</td><td>One-time sampling of 14 cisterns about 6” below surface and from HH tap.
Range 23 – 128 μmhos/cm</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>Kentucky, Tennessee, St Maarten island</td><td>Comparative study of three sites, (two with acid rain one without). 25 HH from Kentucky and Tennessee (acid rain) and 25 HH in St Maarten (no acid rain). Bulk deposition sampled near one HH in each region. roof runoff from one household in Kentucky. Sample from 0.5m below cistern surface and at cistern bottom (inc sediment) and tap water sampled at all sample sites
<table>
<tr><td></td><td>St Maarten (μs/cm)</td><td>Kentucky/ Tennessee (μs/cm)</td></tr>
<tr><td>Bulk (mean)</td><td>36.5</td><td>31.9</td></tr>
<tr><td>Cistern surface (median)</td><td>74.6</td><td>101</td></tr>
</table></td><td>[Olem & Berthouex, 1989]</td></tr>
<tr><td>St Maarten</td><td>Study of atmospheric deposition, tank and distribution of 46 DRWH systems
Rainfall; 50-75, mean: 63 μS/cm
Surface: 63-320, mean 115 μS/cm
Dissolved metals removed by increased pH and calcium (from cement tanks)</td><td>[Gumbs & Dierberg, 1985]</td></tr>
</table>

Table 34: Hardness – $CaCO_3$ (WHO recommendation: none)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Victoria</td><td>4 concrete and 2 GI tanks. 11-14 samples over 15 months for each type
Range of means: 20.27 - 52.7 mg/l (old concrete much higher)</td><td>[Thurman, 1995]</td></tr>
<tr><td>Zambia</td><td>1 school in urban Zambia. For comparison several other water supply options were also sampled
(mg/l)
<table>
<tr><td>RW Tank</td><td>20</td></tr>
<tr><td>Piped water</td><td>112-184</td></tr>
<tr><td>Private boreholes</td><td>340-500</td></tr>
<tr><td>Shallow wells</td><td>28-528</td></tr>
</table></td><td>[Handia, 2005]</td></tr>
<tr><td>Singapore</td><td>38 samples collected from one rooftop over 2 years
Mean: 0.1 mg/l, SD 0.3</td><td>[Appan, 1999]</td></tr>
</table>

Table 35: pH (WHO recommendation: 6.5-9.5 aesthetic)

Location	Notes	Source
Singapore	38 samples collected from one rooftop over 2 years Mean; 4.1, SD 0.4	[Appan, 1999]
Victoria	22 tanks, 1-3 samples per tank in rural area 6.5-9.7, median 7.5	[Bannister et al., 1997]

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Texas</td><td>31 samples from 4 roof types and rain in rural Texas. Local industry includes paper mills, fertiliser and animal feed production. Houston is 240km south – predominant wind is southerly. Samples were not normally distributed. Some samples may be first-flush due to apparatus being allowed to overflow
Medians:
<table>
<tr><td>Rain</td><td>Range: 4.20 – 7.03
Median: 5.69</td></tr>
<tr><td>Wood shingle</td><td>Range: 3.33 – 6.89
Median: 5.03</td></tr>
<tr><td>Composite shingle</td><td>Range: 4.08 – 8.25
Median: 6.73</td></tr>
<tr><td>Aluminium</td><td>Range: 4.78 – 7.26
Median: 6.22</td></tr>
<tr><td>GI</td><td>Range: 3.62 – 7.41
Median: 6.63</td></tr>
</table></td><td>[Chang et al., 2004]</td></tr>
<tr><td>NSW</td><td>Three sites surveyed over a period of time average values quoted
Site 1: Rain:5.9. Roof:5.8, Outlet:6.1, Tank surface:6.2
Site 2: Roof:5.8, Outlet:5.7</td><td>[Coombes et al., 2005]</td></tr>
<tr><td>Hawaii</td><td>One-time sampling of 9 cisterns about 6” below.
Range 5.1 – 7.8</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>Hawaii</td><td>One-time sampling of 14 cisterns about 6” below surface and from HH tap.
Range 5.2 – 9.2 (rainfall 7.5)</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>St Maarten</td><td>Study of atmospheric deposition and tank at 46 DRWH systems
Rainfall; 6.05-6.30, mean: 6.13
Surface: 7.35-10.25, mean 8.1
Dissolved metals removed by increased pH and calcium (from cement tanks)</td><td>[Gumbs & Dierberg, 1985]</td></tr>
<tr><td>Zambia</td><td>1 school and 4 HHs in urban Zambia tested for WQ at rain, roof and tank. For comparison several other water supply options were also sampled
<table>
<tr><td>Rain</td><td>6.5-8.4</td></tr>
<tr><td>FF system (GI)</td><td>5.5-7.9</td></tr>
<tr><td>FF system (Asb)</td><td>6.5-7.4</td></tr>
<tr><td>Tank</td><td>6.1-9.9</td></tr>
<tr><td>Piped water</td><td>6.0-7.5</td></tr>
<tr><td>Private boreholes</td><td>6.3-7.9</td></tr>
<tr><td>Shallow wells</td><td>6.1-7.5</td></tr>
</table></td><td>[Handia, 2005]</td></tr>
<tr><td>Pretoria</td><td>Comparative study
<table>
<tr><td>Rainwater</td><td>6.85</td></tr>
<tr><td>River</td><td>8.20</td></tr>
<tr><td>Borehole</td><td>8.22</td></tr>
<tr><td>Well</td><td>6.30</td></tr>
</table></td><td>[Nevondo & Cloete, 1999]</td></tr>
<tr><td>Kentucky, Tennessee, St Maarten island</td><td>Comparative study of three sites, (two with acid rain one without). 25 HH from Kentucky and Tennessee (acid rain) and 25 HH in St Maarten (no acid rain). Bulk deposition sampled near one HH in each region. roof runoff from one household in Kentucky. Sample from 0.5m below cistern surface and at cistern bottom (inc sediment) and tap water sampled at all sample sites
<table>
<tr><td></td><td>St Maarten</td><td>Kentucky/ Tennessee</td></tr>
<tr><td>Bulk (mean)</td><td>6.54</td><td>4.41</td></tr>
<tr><td>Cistern surface (median)</td><td>7.0</td><td>7.61</td></tr>
</table>
Increase in pH of ~ 1.5 from roof, increase of ~0.5 in (masonry) tank, sediment vary variable, little change from plumbing</td><td>[Olem & Berthouex, 1989]</td></tr>
<tr><td>Kerala</td><td>Comparative study of 30 tanks and 10 traditional sources
<table>
<tr><td>Rainwater</td><td><6.5: 0%
6.5-8.5: 46%
>8.5: 54%</td></tr>
<tr><td>Other (river, pond, well, spring)</td><td><6.5: 60%
6.5-8.5: 40%
>8.5: 0%</td></tr>
</table></td><td>[Pushpangadan et al., 2001]</td></tr>
</table>

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Germany</td><td>Study of runoff from 5 roof types for two storms<table><tr><td>Rainfall</td><td>3.88, 3.87</td></tr><tr><td>Tar felt</td><td>3.81, 4.60</td></tr><tr><td>Clay pantiles</td><td>4.61, 5.49</td></tr><tr><td>Asbestos cement</td><td>7.36, 7.27</td></tr><tr><td>Zinc sheet</td><td>6.76, 6.54</td></tr><tr><td>Gravel</td><td>7.13, 6.76</td></tr></table>pH increased by dissolution of roofing materials and $CaCO_2$ in particles</td><td>[Quek & Forster, 1993]</td></tr>
<tr><td>Virgin Islands</td><td>30 cisterns tested
In acceptable range</td><td>[Rinehart et al., 1985]</td></tr>
<tr><td>New Zealand</td><td>5.2-11.4
%iles: 25^{th} 6.9,Median 7.3, 75^{th} 8.8, 95^{th} 9.7
Ferrocement water tanks associated with alkaline conditions</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>Queensland</td><td>Rain collected from roofs. GI and concrete tile roofs. Rural, industrial and urban (6 in all)8 samples from each catchment over 5 months
Rural GI 6.8-7 Con: 7.2-8.1
lower for urban (but not stated), all sited higher pH for conc.</td><td>[Thomas & Greene, 1993]</td></tr>
<tr><td>Victoria</td><td>4 concrete and 2 GI tanks. 11-14 samples over 15 months for each type
Range of means: 7.54 – 10.2 (new concrete tank much higher than norm)</td><td>[Thurman, 1995]</td></tr>
</table>

Table 36: Total Dissolved Solids (WHO recommendation: 1000 mg/l aesthetic)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>NSW</td><td>Three sites surveyed over a period of time average values quoted
Site 1: Rain:21 mg/l. Roof:57, Outlet:114, Tank surface:98.2
Site 2: Roof:57, Outlet:67</td><td>[Coombes et al., 2005]</td></tr>
<tr><td>Singapore</td><td>38 samples collected from one rooftop over 2 years
Mean: 19.5 mg/l, SD 12.5</td><td>[Appan, 1999]</td></tr>
<tr><td>Zambia</td><td>1 school and 4 HHs in urban Zambia tested for WQ at rain, roof and tank. For comparison several other water supply options were also sampled
(mg/l)<table><tr><td>Rain (mg/l)</td><td>2.8-13.5</td></tr><tr><td>FF system (GI)</td><td>2.5-15</td></tr><tr><td>FF system (asb)</td><td>36-43.6</td></tr><tr><td>Tank</td><td>8.5-102</td></tr><tr><td>Piped water</td><td>85.2-115</td></tr><tr><td>Private boreholes</td><td>273-462</td></tr><tr><td>Shallow wells</td><td>203-757</td></tr></table></td><td>[Handia, 2005]</td></tr>
<tr><td>Victoria</td><td>4 concrete and 2 GI tanks. 11-14 samples over 15 months for each type
range of means: 18.18 - 86.07 mg/l (old concrete much higher)</td><td>[Thurman, 1995]</td></tr>
</table>

Table 37: Total Suspended Solids (WHO recommendation: none)

Location	Notes	Source
Singapore	38 samples collected from one rooftop over 2 years Mean: 9.1 mg/l, SD 8.9	[Appan, 1999]
NSW	Three sites surveyed over a period of time average values quoted Site 1: Rain:8.4 mg/l, Roof:35, Outlet:1.6, Tank surface:1.4 Site 2: Roof:35, Outlet:19	[Coombes et al., 2005]

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Germany</td><td>Study of runoff from 5 roof types for two storms
(mg/l)<table><tr><td>Rainfall</td><td>14, 26</td></tr><tr><td>Tar felt</td><td>29, 75</td></tr><tr><td>Clay pantiles</td><td>24, 69</td></tr><tr><td>Asbestos cement</td><td>24, 96</td></tr><tr><td>Zinc sheet</td><td>43, 114</td></tr><tr><td>Gravel</td><td>14, 12</td></tr></table>High values from zinc sheet thought to be from easy wash-off from smooth surface. Good FF effect</td><td>[Quek & Forster, 1993]</td></tr>
</table>

Table 38: Turbidity (WHO recommendation: <5NTU aesthetic)

<table>
<tr><th>Location</th><th>Notes</th><th>Source</th></tr>
<tr><td>Virgin Islands</td><td>0.1- 7.3, average 1.1 NTU</td><td>[Crabtree et al., 1996]</td></tr>
<tr><td>Zambia</td><td>1 school and 4 HHs in urban Zambia tested for WQ at rain, roof and tank. For comparison several other water supply options were also sampled<table><tr><td>Rain</td><td><2</td></tr><tr><td>FF system (GI)</td><td><2</td></tr><tr><td>FF system (asb)</td><td><2</td></tr><tr><td>Tank</td><td><2</td></tr><tr><td>Piped water</td><td><2</td></tr><tr><td>Private boreholes</td><td>-</td></tr><tr><td>Shallow wells</td><td>-</td></tr></table></td><td>[Handia, 2005]</td></tr>
<tr><td>New Zealand</td><td>0.04-4.7NTU
%iles: 25th 0.30,Median 0.56, 75th 0.90, 95th 2.20
No correlation with bacterial indicator organisms</td><td>[Simmons et al., 2001]</td></tr>
<tr><td>Singapore</td><td>38 samples collected from one rooftop over 2 years
Mean: 4.6 NTU, SD 5.7</td><td>[Appan, 1999]</td></tr>
<tr><td>Victoria</td><td>22 tanks, 1-3 samples per tank in rural area
0.5-2.2 NTU median: 2.0</td><td>[Bannister et al., 1997]</td></tr>
<tr><td>Hawaii</td><td>One-time sampling of 14 cisterns about 6” below surface and from HH tap.
Range 0.1 – 2.7 NTU</td><td>[Fujioka et al., 1991]</td></tr>
<tr><td>Zambia</td><td>1 school and 4 HHs in urban Zambia tested for WQ at rain, roof and tank. For comparison several other water supply options were also sampled<table><tr><td>Rain (NTU)</td><td>0.14-10.3</td></tr><tr><td>FF system (GI)</td><td>0.20-2.81</td></tr><tr><td>FF system (asb)</td><td>0.45-13.2</td></tr><tr><td>Tank</td><td>0.20-3.9</td></tr><tr><td>Piped water</td><td>0.5-16.9</td></tr><tr><td>Private boreholes</td><td>0.26-7.9</td></tr><tr><td>Shallow wells</td><td>0.75-35.4</td></tr></table></td><td>[Handia, 2005]</td></tr>
<tr><td>Queensland</td><td>Rain collected from roofs. GI and concrete tile roofs. Rural, industrial and urban (6 in all)8 samples from each catchment over 5 months
“Averages”<table><tr><td>Rural</td><td>Urban</td><td>Industrial</td></tr><tr><td>GI: 1.2 Con: 0.6</td><td>GI: 0.8 Con: 6.5</td><td>GI: 3, Con: 5.5</td></tr></table>Correlated with preceding dry days</td><td>[Thomas & Greene, 1993]</td></tr>
</table>

APPENDIX E
FIRST-FLUSH DATA

TURBIDITY DATA

Table 32: Kabanyolo (Uganda) – Roadside, Tile

Sample							
L_1 (NTU)	L_2 (NTU)	L_3 (NTU)	L_4 (NTU)	L_5 (NTU)	k_w	L_0 (NTU)	R^2
2000	1900	900	300		n/a	n/a	n/a
1900	1800	520	280		0.88	2569	0.88
700	500	300	160		0.62	794	0.84
800	500	450	200		0.53	882	0.85
2000	520	490	320		0.88	2055	0.97
1000	850	300	170		0.81	1272	0.89
1850	1700	270	200		1.12	2675	0.96
1000	350	150	28		1.67	1451	0.99
480	200	100	25		1.32	638	0.97
400	300	240	180		0.30	398	0.74
180	100	50	35		0.75	197	0.91
100	25	18	10		1.31	126	0.98
800	250	200	120		0.91	856	0.96
1000	300	210	150		0.98	1099	0.96
350	100	70	15		1.47	471	0.98
800	400	200	70		1.09	1007	0.97
600	200	130	65		1.13	720	0.97
2000	1950	1900	1500		n/a	n/a	n/a
1400	800	600	250		0.69	1570	0.90
700	550	300	140		0.67	843	0.83
750	600	300	150		0.69	906	0.84

Table 33: Kabanyolo (Uganda) – Roadside, GI

Sample							
L_1 (NTU)	L_2 (NTU)	L_3 (NTU)	L_4 (NTU)	L_5 (NTU)	k_w	L_0 (NTU)	R^2
2000	1980	1500	500		n/a	n/a	n/a
1800	200	120	80		2.82	3500	1.00
800	700	200	110		0.92	1071	0.91
600	450	120	38		1.18	844	0.91
700	500	140	42		1.20	980	0.92
800	290	270	190		0.58	734	0.94
1000	800	700	200		0.59	1279	0.77
900	280	55	25		2.20	1506	1.00
360	180	40	8		1.67	541	0.98
320	280	100	75		0.67	382	0.89
160	50	30	15		1.27	203	0.98
75	40	8	<5		1.69	113	0.99
500	200	78	50		1.29	658	0.97

Sample							
L_1 (NTU)	L_2 (NTU)	L_3 (NTU)	L_4 (NTU)	L_5 (NTU)	k_w	L_0 (NTU)	R^2
600	220	100	55		1.32	788	0.98
300	95	30	9		1.95	470	1.00
600	350	200	95		0.79	693	0.91
200	75	50	35		0.82	209	0.94
2000	1800	1700	1200		n/a	n/a	n/a
1700	1400	450	300		0.82	2134	0.92
800	650	140	95		1.07	1100	0.96
760	500	170	100		0.98	962	0.95

Table 34: Kabanyolo (Uganda) – Compound (100m from road), GI

Sample							
L_1 (NTU)	L_2 (NTU)	L_3 (NTU)	L_4 (NTU)	L_5 (NTU)	k_w	L_0 (NTU)	R^2
560	520	200	34		0.97	795	0.97
50	42	19	15		0.53	55	0.84
45	32	18	5		0.84	57	0.94
78	43	20	9		0.99	96	0.96
80	65	14	8		1.11	112	0.95
17	9	5	<5		1.09	22	0.97
19	7	<5	<5		n/a	n/a	n/a
20	6	<5	<5		n/a	n/a	n/a
11	5	<5	<5		n/a	n/a	n/a
120	15	10	<5		2.95	233	1.00
20	14	<5	<5		1.59	31	0.99
16	10	<5	<5		n/a	n/a	n/a
18	10	<5	<5		n/a	n/a	n/a
25	15	7	<5		1.15	33	0.99
55	35	10	<5		1.39	79	1.00
70	30	19	<5		1.29	92	1.00
60	30	17	<5		1.23	79	1.00
1200	900	500	350		0.52	1287	0.82
700	650	150	80		1.01	983	0.91
200	160	40	9		1.22	288	0.93
205	160	45	10		1.18	291	0.93

Table 35: Kabanyolo (Uganda) – Compound (150m from road), Tile

Sample							
L_1 (NTU)	L_2 (NTU)	L_3 (NTU)	L_4 (NTU)	L_5 (NTU)	k_w	L_0 (NTU)	R^2
500	200	40	32		0.97	795	0.97
20	10	3	<5		0.53	55	0.84
28	20	10	<5		0.84	57	0.94
30	23	10	4		0.99	96	0.96
37	27	20	9		1.11	112	0.95
220	75	21	15		1.09	22	0.97
105	20	9	<5		1.98	31	1.00
30	17	5	<5		2.17	34	0.99
8	<5	<5	<5		n/a	n/a	n/a
10	<5	<5	<5		n/a	n/a	n/a
25	14	5	<5		1.04	38	0.99
20	11	<5	<5		n/a	n/a	n/a

Sample							
L_1 (NTU)	L_2 (NTU)	L_3 (NTU)	L_4 (NTU)	L_5 (NTU)	k_w	L_0 (NTU)	R^2
40	20	10	<5		0.56	43	0.78
30	10	<5	<5		n/a	n/a	n/a
20	<5	<5	<5		n/a	n/a	n/a
18	7	<5	<5		n/a	n/a	n/a
16	6	<5	<5		n/a	n/a	n/a
800	700	300	100		2.02	16	1.00
500	200	70	35		1.31	35	0.97
180	100	20	<5		1.73	31	1.00
185	105	30	5		1.44	265	0.97

Table 36: Colombo (Sri Lanka) – Near busy paved road, Tile

Sample							
L_1 (NTU)	L_2 (NTU)	L_3 (NTU)	L_4 (NTU)	L_5 (NTU)	k_w	L_0 (NTU)	R^2
1.70	0.78	0.47		0.32	1.74	2.01	0.96
5.05	0.35	0.45	0.83		3.34	20.13	0.99
4.66	3.28	2.03			1.55	5.82	0.91
0.64	0.08	0.04		0.01	4.40	1.32	1.00
2.12	2.46	0.95	0.48		1.04	3.51	0.81
3.22	2.95	2.71	2.61		0.31	3.28	0.84
1.48	1.73		0.56		1.09	2.12	0.81
0.65	0.04		0.20		2.76	3.11	0.94

Table 37: Colombo (Sri Lanka) – Near busy paved road, GI

Sample							
L_1 (NTU)	L_2 (NTU)	L_3 (NTU)	L_4 (NTU)	L_5 (NTU)	k_w	L_0 (NTU)	R^2
0.78	0.64	0.40			1.41	0.96	0.88
0.76	0.33	0.23			2.02	0.97	0.97
1.04	0.72	0.74			1.30	1.07	0.86
0.33	0.33	0.10			1.51	0.57	0.78
0.47	0.06	0.27	0.31	0.11	0.23	0.65	0.99
3.81	1.56	1.82			1.49	4.44	0.99
1.01	0.20				n/a	n/a	n/a
0.42	0.92				n/a	n/a	n/a
1.10	0.98	0.28			1.62	1.92	0.83

DISSOLVED SOLIDS DATA

Table 38: Colombo (Sri Lanka) – Near busy paved road, Asbestos

Sample							
L_1 (PPM)	L_2 (PPM)	L_3 (PPM)	L_4 (PPM)	L_5 (PPM)	k_w	L_0 (PPM)	R^2
61					n/a	n/a	n/a
110	57	41	31	24	0.98	130	0.98
63					n/a	n/a	n/a
80	72	62	35	46	0.49	97	0.95
58	47	44	30	26	0.45	65	0.92
56	42	41	32		0.53	56	0.88
105	68	67			1.19	126	0.93
67	53	43			1.13	81	0.92

Sample					k_w	L_0 (PPM)	R^2
L_1 (PPM)	L_2 (PPM)	L_3 (PPM)	L_4 (PPM)	L_5 (PPM)			
93	51	43	42	33	0.60	96	0.96
96	60	37	24	21	1.01	119	0.99
84	55	37	25	24	0.86	101	0.99
56	54	46	40	33	0.27	61	0.87
51	44	35	21	18	0.60	61	0.93
87	52	45	43	43	0.48	89	0.96
75	27	45	37	40	0.40	69	0.98
80	62	60	43	46	0.39	88	0.96
63	49	46	39	35	0.33	66	0.93
68	51	58			0.97	78	0.88
142	78	60	59		0.85	148	0.92
102	65	67	70	68	0.21	95	0.93
150	88				n/a	n/a	n/a
73	58	46	35	39	0.48	83	0.98
111	64	52	43	34	0.71	122	0.97
110	74	56	40	69	0.60	129	0.98
69	66	49	40	33	0.41	79	0.87
44	37	29	24	24	0.42	49	0.97
229	118	84	68		1.12	264	0.96
142	110	73	56	47	0.68	166	0.98
104	76	58	44	42	0.60	118	0.98
97	94	71			1.00	119	0.88
169	86	56	46	1	1.20	201	1.00
128	82	61	40	26	0.86	150	0.98
139	184	138	82	56	0.43	182	0.85
93	77	65	42	42	0.53	108	0.96
58	45	41	13	33	0.72	76	0.98
101	78	73	33	34	0.68	122	0.95
84	101	79	74		0.30	89	0.75
102	74	59	60	62	0.35	105	0.96
47	99	91	59	46	0.08	73	0.69
153	112	101	88	56	0.42	157	0.93
127	92	84	53		0.73	139	0.92
224	160	121	81		0.86	254	0.94
114	83	43	72		0.56	108	0.91
101	102	91	68		0.43	108	0.84
299	235	166	151		0.66	316	0.90
218	122	111	72		0.94	242	0.95
75	66		66		0.38	62	0.75
113	129	77	70		0.54	129	0.80
124	72	57	53		0.81	129	0.92

Table 39: Colombo (Sri Lanka) – Near busy paved road, GI

Sample					k_w	L_0 (PPM)	R^2
L_1 (PPM)	L_2 (PPM)	L_3 (PPM)	L_4 (PPM)	L_5 (PPM)			
72	75	33			1.19	94	0.83
52	39	30	22		0.76	57	0.93
62	41	37			1.23	75	0.94
56					n/a	n/a	n/a

Sample					k_w	L_0 (PPM)	R^2
L_1 (PPM)	L_2 (PPM)	L_3 (PPM)	L_4 (PPM)	L_5 (PPM)			
50	31	19	19	16	0.75	56	0.98
58	35				n/a	n/a	n/a
55	41				n/a	n/a	n/a
39	31	24			1.15	48	0.92
48	34	29	27	28	0.38	50	0.96
51	28	20	16		1.05	58	0.95
58	32	30	23	23	0.63	63	0.97
30	13	14	11	11	0.70	32	0.97
31	24	13	12	8	0.76	36	0.97
40	30	22	15	27	0.56	48	0.97
48	26	32	27	28	0.33	46	0.95
45	31	50			0.81	49	0.79
45	19	25	21	17	0.52	43	0.97
38	34				n/a	n/a	n/a
79	54				n/a	n/a	n/a
58	27	30	28	25	0.49	56	0.96
90					n/a	n/a	n/a
45	17	26	23	18	0.44	41	0.97
82	44	27	28	29	0.82	94	0.98
70	44	33	29	19	0.69	76	0.95
56	49	45	29	37	0.39	65	0.95
34	23	18	19	16	0.42	35	0.95
40	46	30	40	25	0.10	40	0.89
178	68	55	36	35	1.26	223	0.99
67	62	54	49	35	0.26	70	0.97
83	49	37	31	28	0.72	93	0.98
49	38	26	27	17	0.49	52	0.93
86	90	63	52	40	0.41	100	0.83
36	46	18	33	15	0.24	38	0.93
36	24	15	19	14	0.52	37	0.96
39	33	32	27	26	0.24	41	0.93
34	38	31	28	23	0.19	38	0.84
48	39	34	33		0.42	46	0.84
53	25	40	22		0.68	53	0.93
69	47	16	49	20	0.38	61	0.98
49	56	39	40	30	0.22	54	0.85
57	60				n/a	n/a	n/a
25	24	28	38	19	-0.22	20	0.90
70	68	56			0.95	85	0.87
175	138	114			1.12	212	0.92
61	66	59	71	37	-0.01	57	0.95
61	66	59	71	37	-0.01	57	0.95
46		31		24	1.06	54	0.98
47	36	27	26		0.60	48	0.88
94	82	62			1.08	115	0.91
47	53	44	63	43	-0.13	43	0.84

Table 40: Colombo (Sri Lanka) – Near busy paved road, Tile

Sample							
L_1 (PPM)	L_2 (PPM)	L_3 (PPM)	L_4 (PPM)	L_5 (PPM)	k_w	L_0 (PPM)	R^2
57	50	36	29		0.64	62	0.89
50	28	22			1.50	64	0.96
66	46				n/a	n/a	n/a
48	41	30	28	20	0.43	52	0.92
51					n/a	n/a	n/a
28	25				n/a	n/a	n/a
56					n/a	n/a	n/a
44	27				n/a	n/a	n/a
35	33	22	21	20	0.38	39	0.95
46	25	16	13		1.15	54	0.96
30	30	20	18	18	0.38	35	0.93
44	35	25	18	17	0.62	52	0.98
50	45	18	10	7	1.09	68	0.94
49	31	25	18	18	0.69	56	0.98
37	17	30	23	19	0.25	33	0.95
36	35	31	21		0.51	39	0.86
36	32	30	23	17	0.34	39	0.95
37	27	30			1.00	43	0.89
51	41				n/a	n/a	n/a
42	37	33	19	15	0.54	49	0.90
96					n/a	n/a	n/a
35	32	30	28	41	0.07	37	0.91
21	23	29	26	26	-0.12	22	0.75
39	37	25	19	23	0.48	47	0.96
43	34	29	22	18	0.48	47	0.97
34	32	21	17	11	0.55	40	0.90
72	31	32	27	18	0.76	76	0.97
29	36	29	21	23	0.25	36	0.81
55	27	29	28	23	0.47	53	0.95
80	65	52			1.11	97	0.92
63	41	38	27	24	0.58	69	0.96
47	42	22	21	11	0.68	55	0.95
103	87	56	36	28	0.75	126	0.94
62	50				n/a	n/a	n/a
29	25	27	14	5	0.53	33	0.99
43	17	19	16		0.93	44	0.96
42					n/a	n/a	n/a
38	35	30	26		0.43	39	0.84
37	38	17	17	11	0.65	45	0.92
67	36	22	5	7	1.52	93	1.00
32	36	31	21	16	0.34	38	0.86
72	91	76			0.78	90	0.77
47	54				n/a	n/a	n/a
52	50	37	37		0.43	53	0.82
88	84	81			0.88	104	0.85
64	47	31	24	27	0.68	75	0.99
64	47	31	24	27	0.68	75	0.99
34	25	22	20	11	0.43	35	0.96

Sample					k_w	L_0 (PPM)	R^2
L_1 (PPM)	L_2 (PPM)	L_3 (PPM)	L_4 (PPM)	L_5 (PPM)			
30	30	27	18	15	0.37	35	0.85
64	66	41			1.05	81	0.87
30	29	31	24		0.31	30	0.80

THERMOTOLERANT COLIFORMS

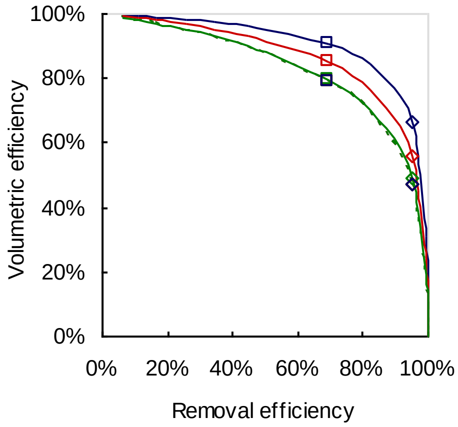
Table 41: Kandy (Sri Lanka)

Sample					k_w	L_0 (cfu/100ml)	R^2
L_1 (cfu/100ml)	L_2 (cfu/100ml)	L_3 (cfu/100ml)	L_4 (cfu/100ml)	L_5 (cfu/100ml)			
14	2				4.67	32	0.83
600	600	300	170		1.00	862	0.84
101	86				0.39	109	0.85

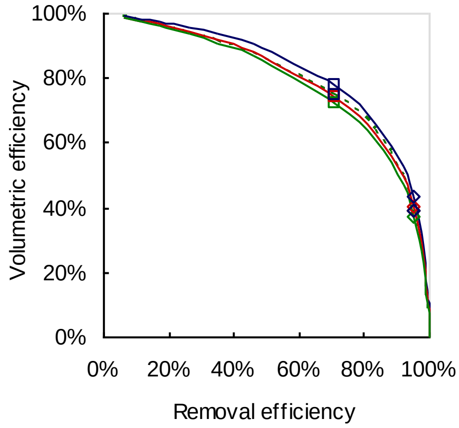
APPENDIX F: FF PERFORMANCE

Figure 42: Removal and volumetric efficiency of first-flush systems with varying demand

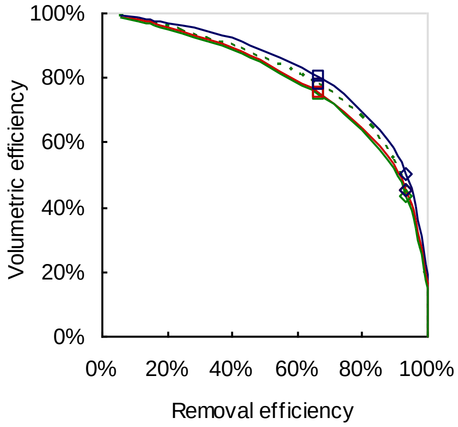
a. Corozal



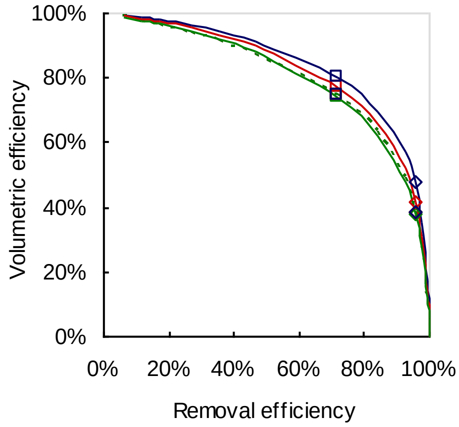
b. Big Lake



c. Blue Canyon



d. Kekaha



e. Newport

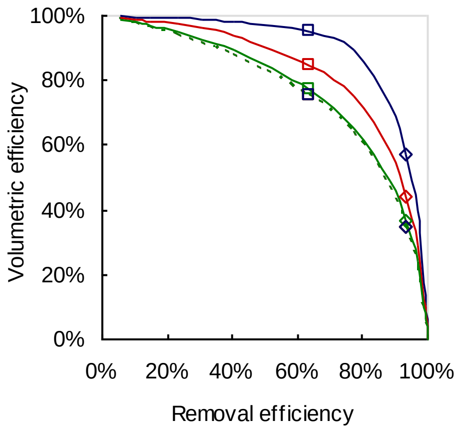
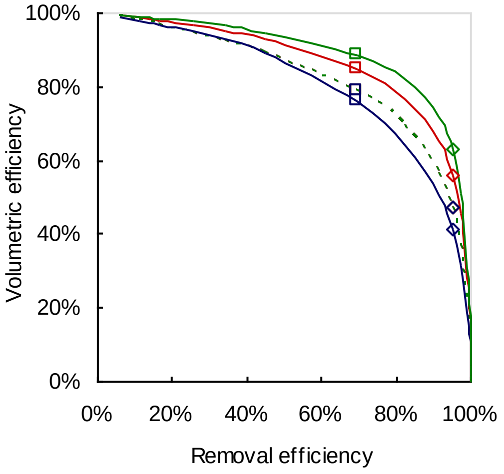
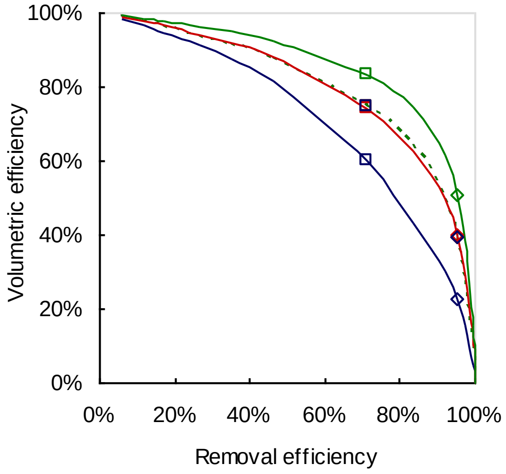


Figure 43: Removal and volumetric efficiency of first-flush systems with varying tank size

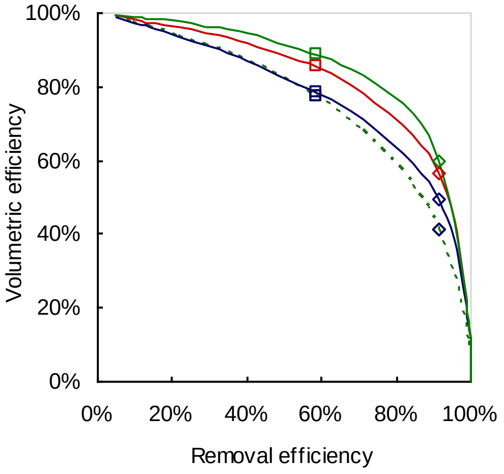
a. Corozal



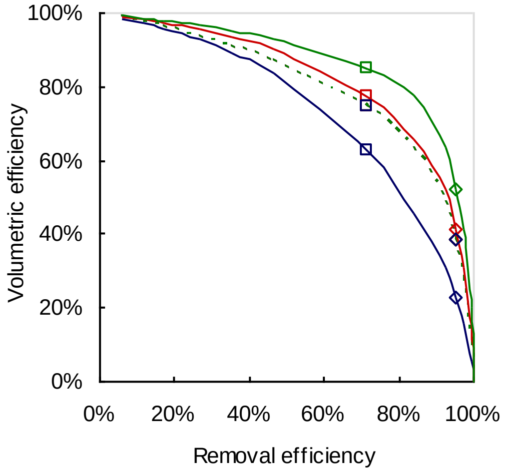
b. Big Lake



c. Blue Canyon



d. Kekaha



e. Newport

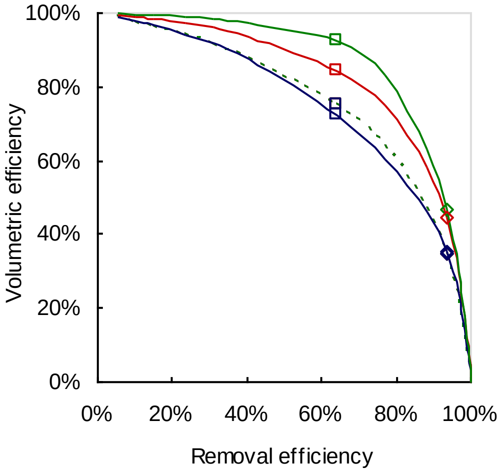
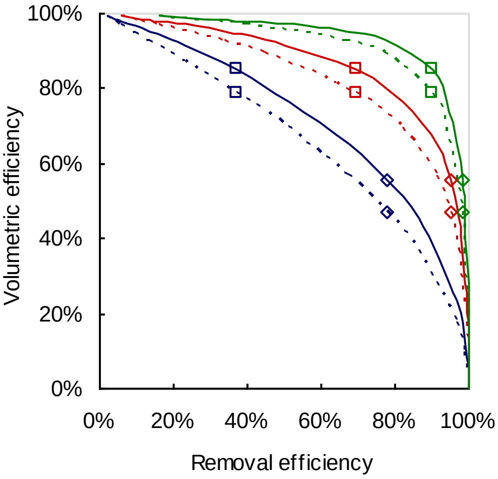
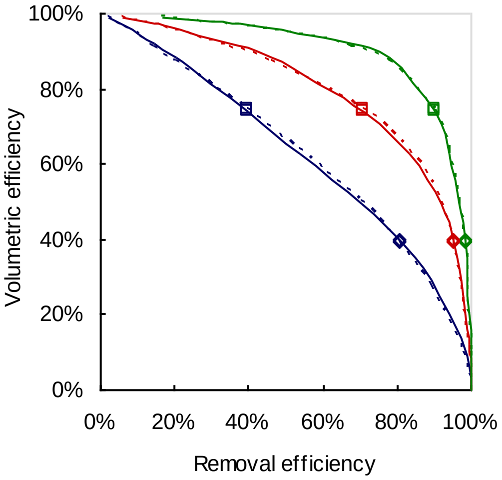


Figure 44: Removal and volumetric efficiency of first-flush systems with varying wash-off rates

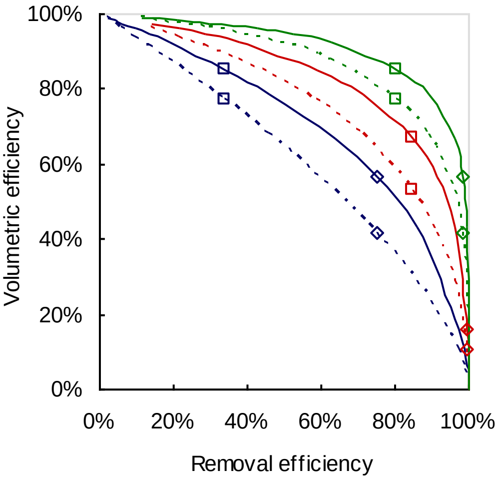
a. Corozal



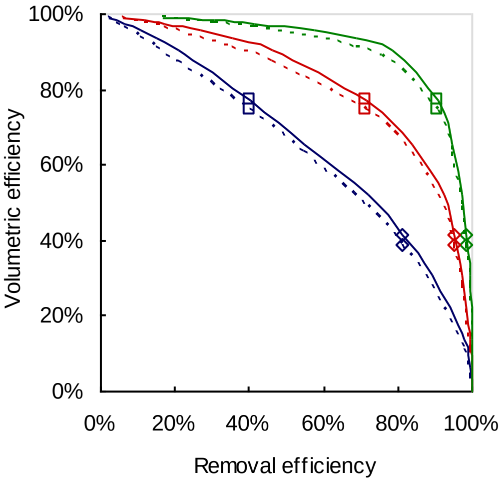
b. Big Lake



c. Blue Canyon



d. Kekaha



e. Newport

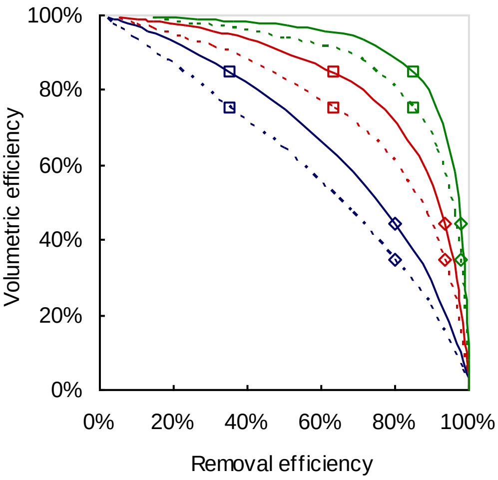


Figure 45: Removal and volumetric efficiency of first-flush systems with varying contaminant accumulation rates

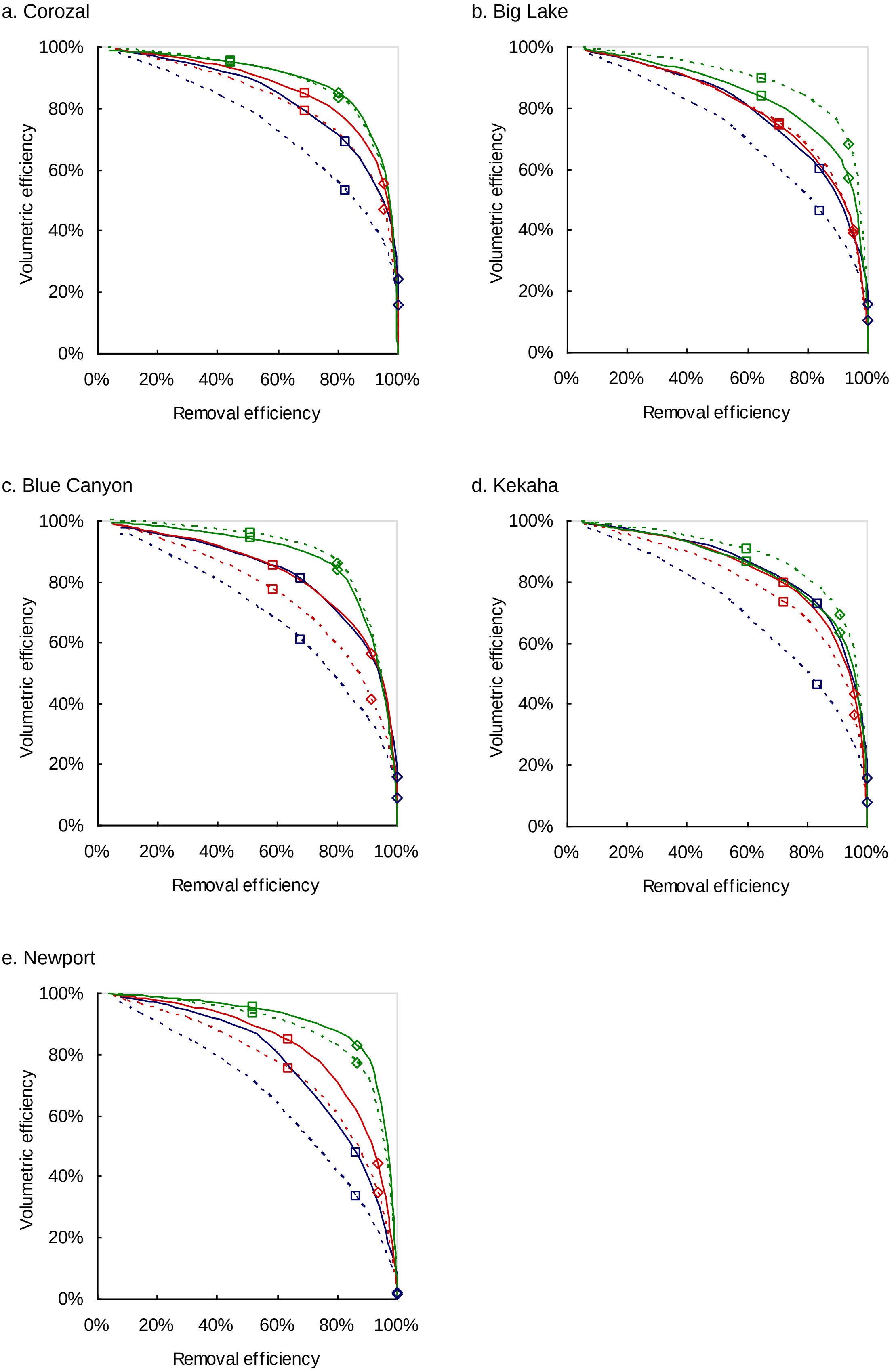
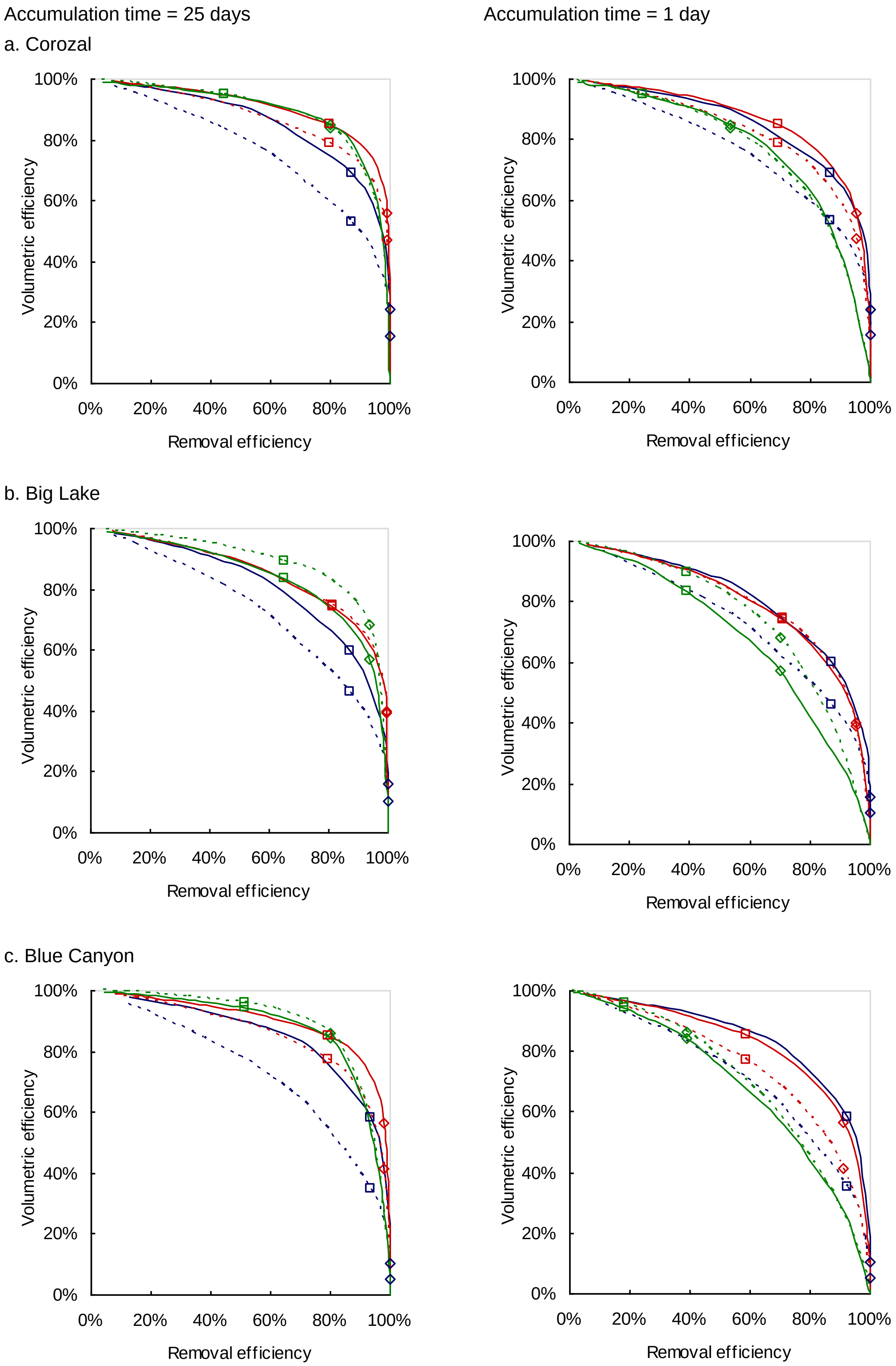


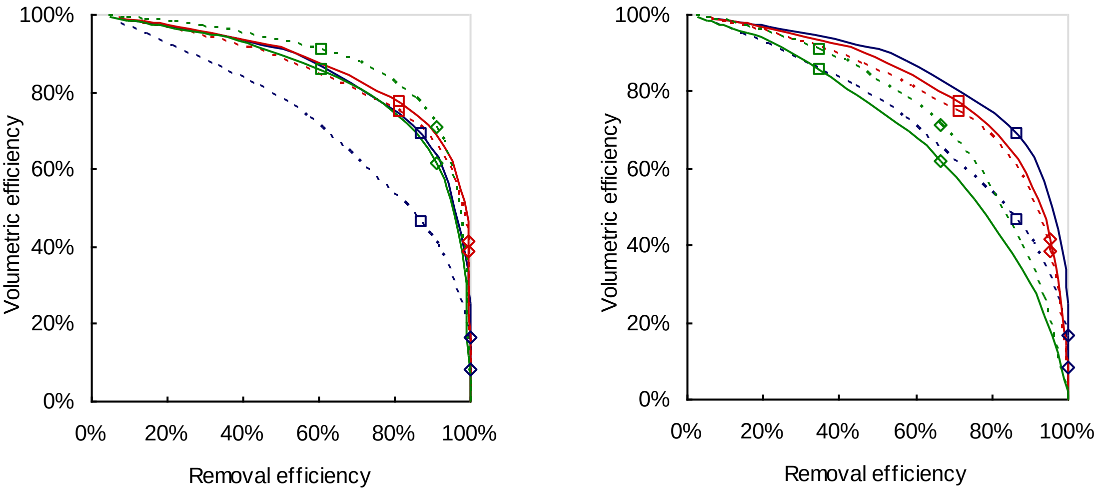
Figure 46: Removal and volumetric efficiency of first-flush systems with varying contaminant removal rates, and non-matching reset rates



Accumulation time = 25 days

Accumulation time = 1 day

d. Kekaha



e. Newport

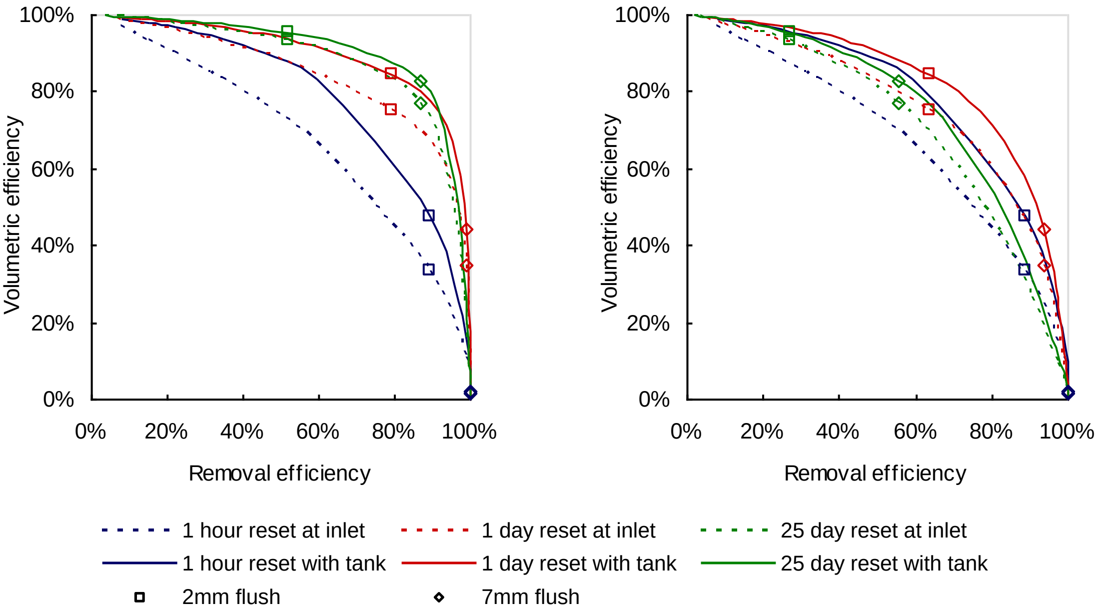
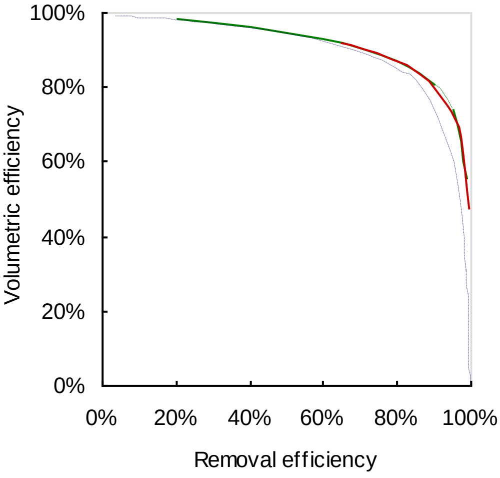


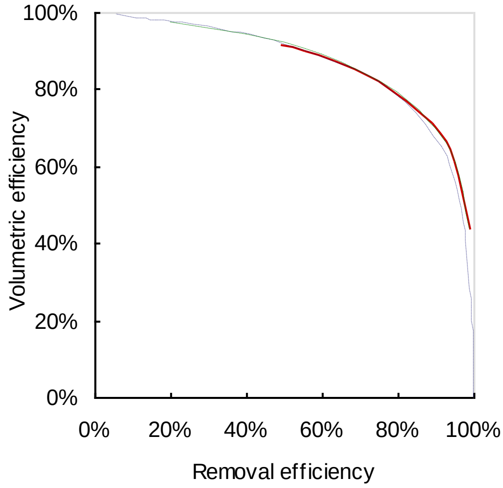
Figure 47: Removal and volumetric efficiencies of optimum, calculated and "matched" first flush devices

Accumulation time = 25 days

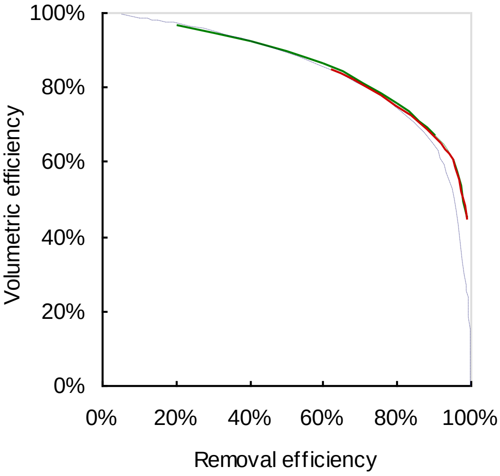
Accumulation time = 1 day

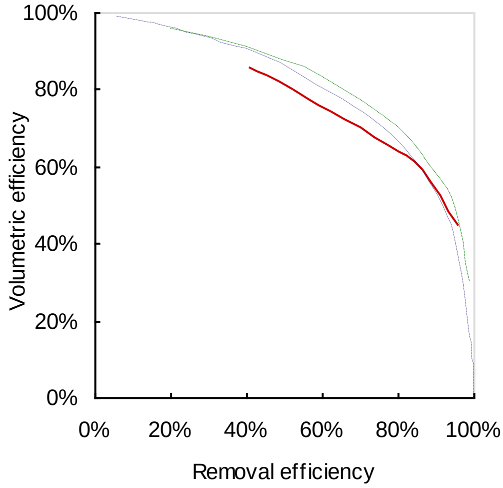
a. Corozal



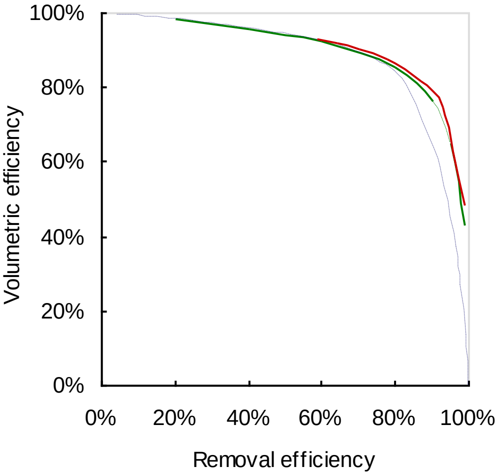


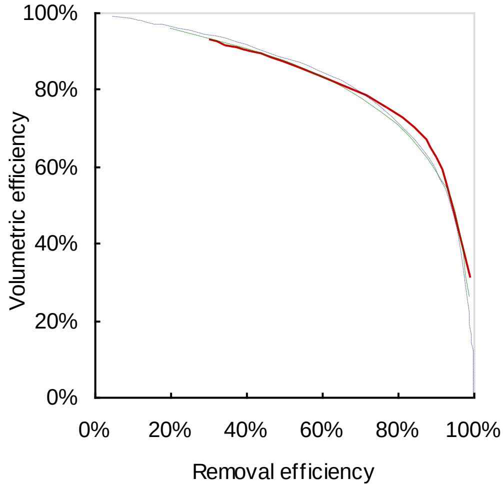
b. Big Lake





c. Blue Canyon

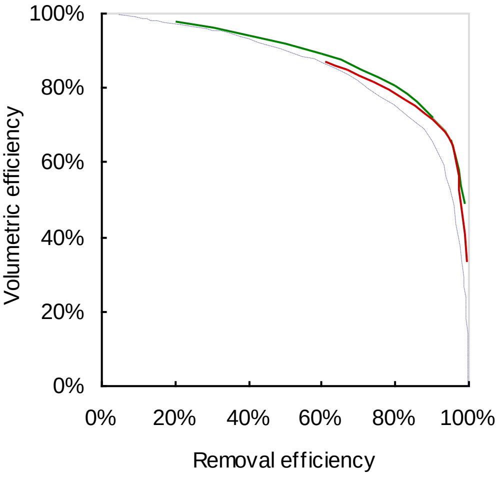


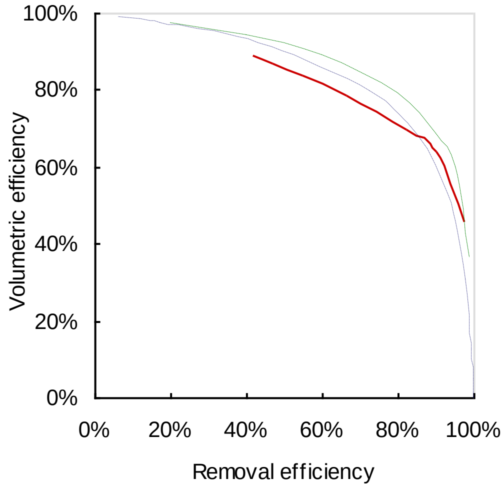


Accumulation time = 25 days

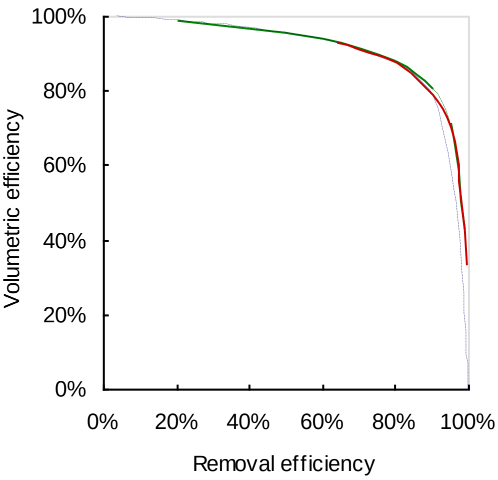
Accumulation time = 1 day

d. Kekaha





e. Newport



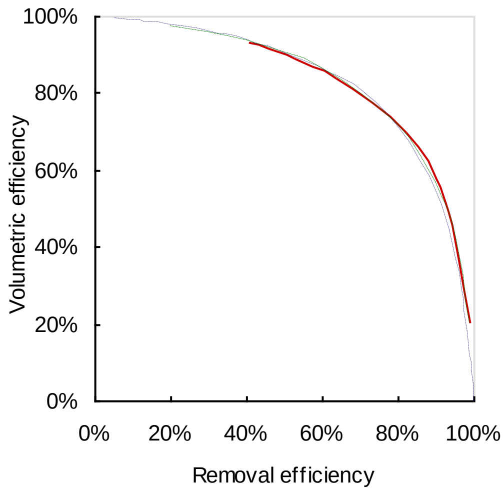


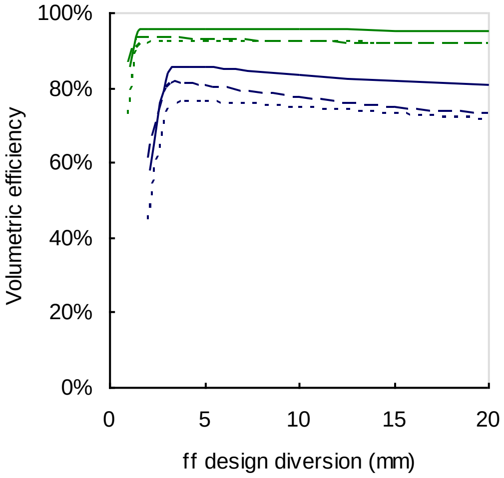
Figure 15.48: Volumetric efficiency of different first-flush devices with constant removal efficiency

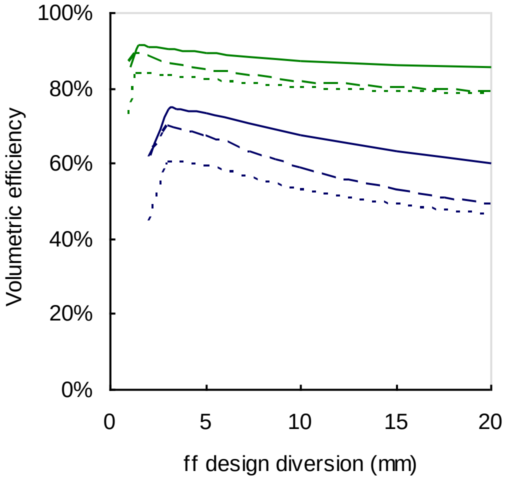
Accumulation time = 25 days

Accumulation time = 1 day

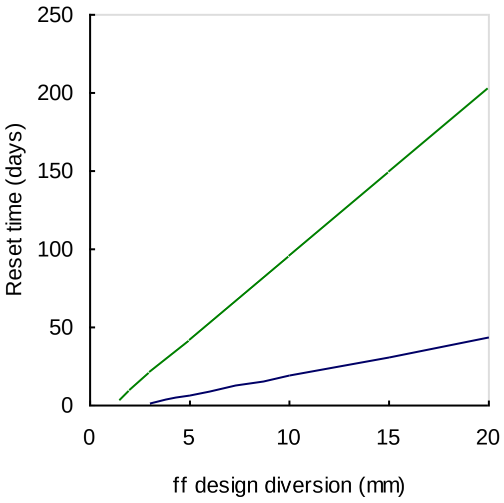
a. Corozal

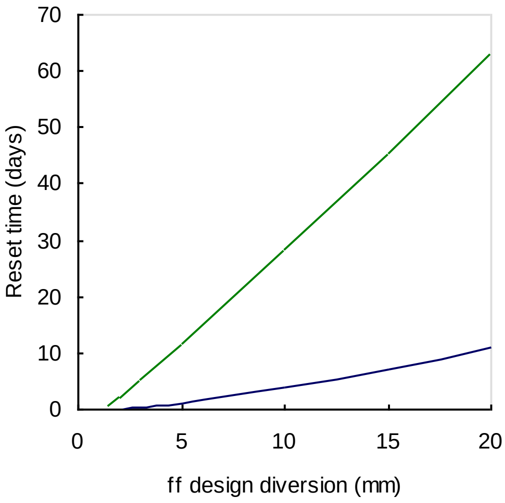
ff diversion





reset times



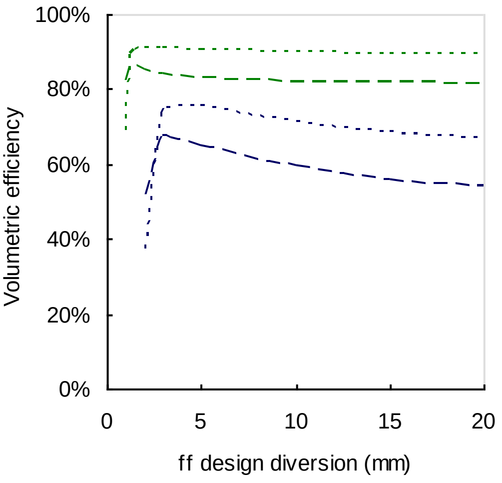


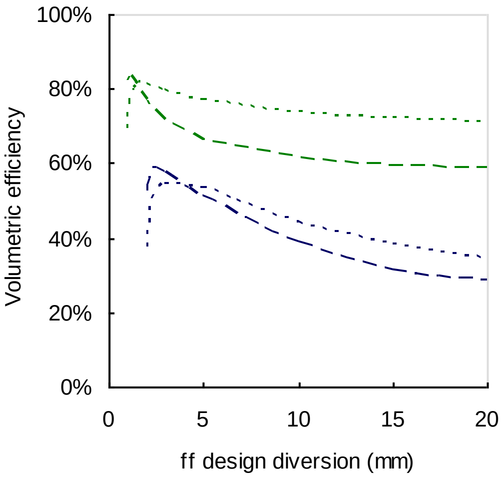
Accumulation time = 25 days

Accumulation time = 1 day

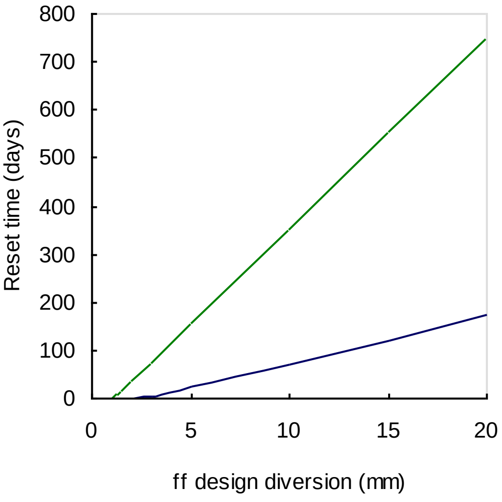
b. Big Lake

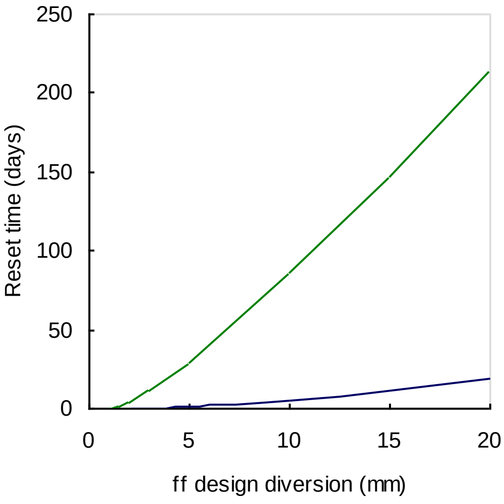
ff diversion





reset times



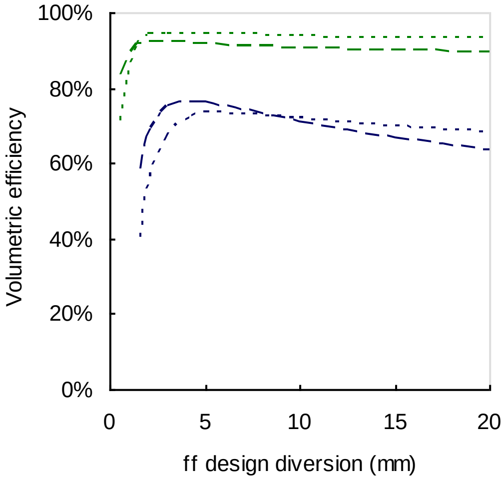


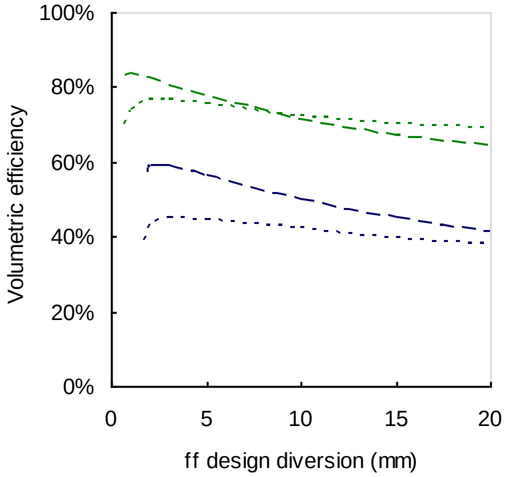
Accumulation time = 25 days

Accumulation time = 1 day

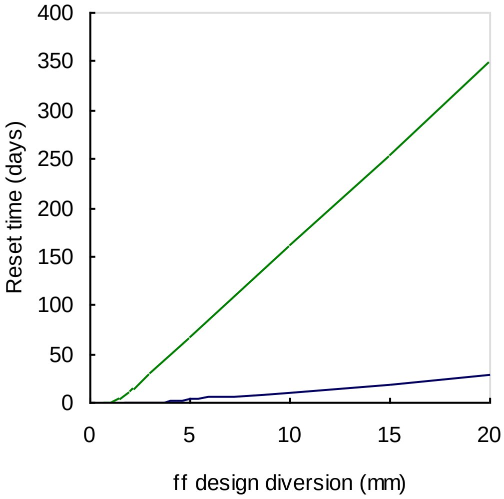
c. Blue Canyon

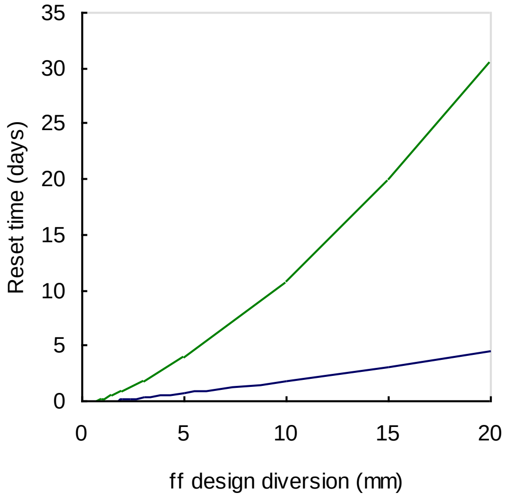
ff diversion





reset times



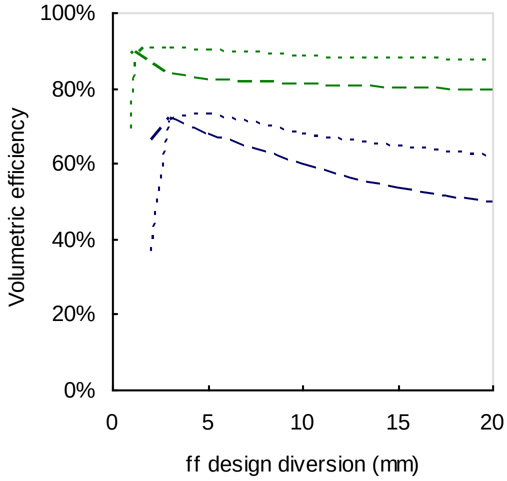


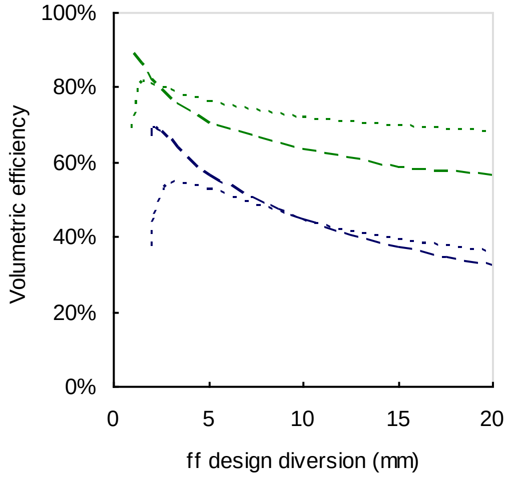
Accumulation time = 25 days

Accumulation time = 1 day

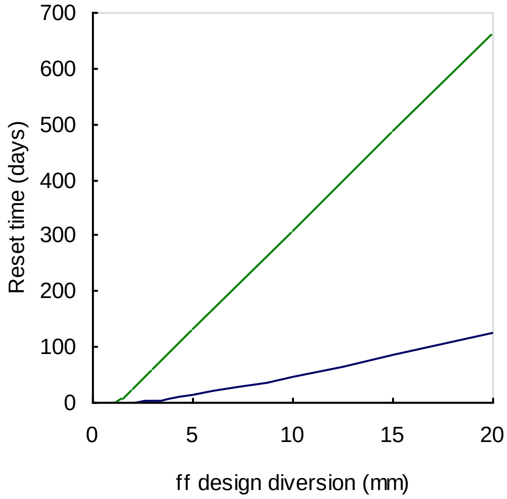
d Kekaha

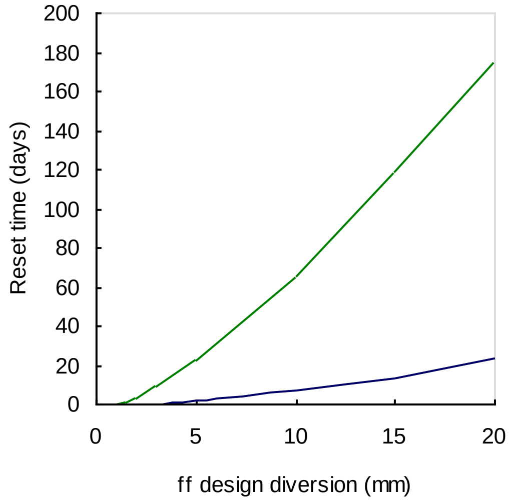
ff diversion





a Kekaha reset times



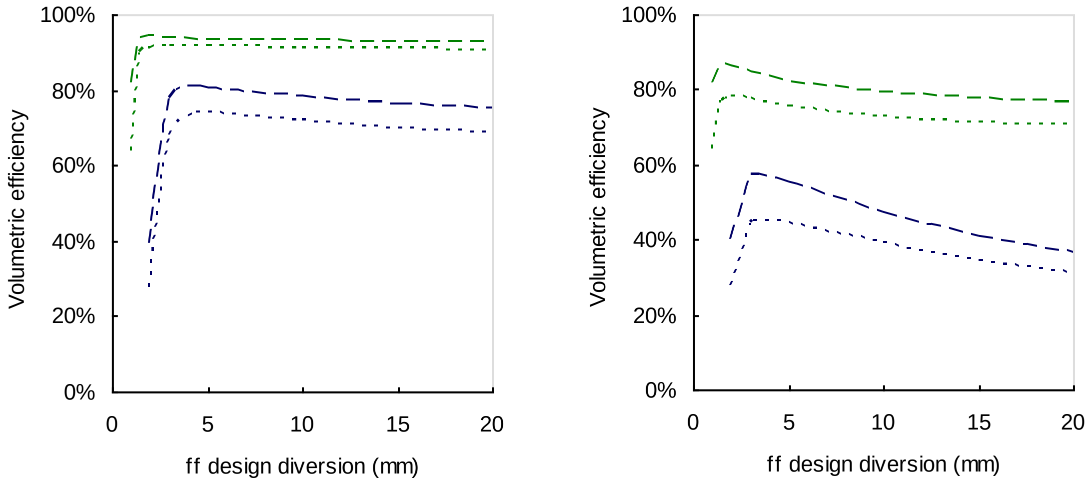


Accumulation time = 25 days

Accumulation time = 1 day

Newport

ff diversion



a Newport reset times