# Economics of Scale Analysis of Communal Rainwater Tanks

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The Urban Water Security Research Alliance (UWSRA) is a $50 million partnership over five years between the Queensland Government, CSIRO's Water for a Healthy Country Flagship, Griffith University and The University of Queensland. The Alliance has been formed to address South East Queensland's emerging urban water issues with a focus on water security and recycling. The program will bring new research capacity to South East Queensland tailored to tackling existing and anticipated future issues to inform the implementation of the Water Strategy.

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# FOREWORD

Water is fundamental to our quality of life, to economic growth and to the environment. With its booming economy and growing population, Australia's South East Queensland (SEQ) region faces increasing pressure on its water resources. These pressures are compounded by the impact of climate variability and accelerating climate change.

The Urban Water Security Research Alliance, through targeted, multidisciplinary research initiatives, has been formed to address the region’s emerging urban water issues.

As the largest regionally focused urban water research program in Australia, the Alliance is focused on water security and recycling, but will align research where appropriate with other water research programs such as those of other SEQ water agencies, CSIRO’s Water for a Healthy Country National Research Flagship, Water Quality Research Australia, eWater CRC and the Water Services Association of Australia (WSAA).

The Alliance is a partnership between the Queensland Government, CSIRO’s Water for a Healthy Country National Research Flagship, The University of Queensland and Griffith University. It brings new research capacity to SEQ, tailored to tackling existing and anticipated future risks, assumptions and uncertainties facing water supply strategy. It is a $50 million partnership over five years.

Alliance research is examining fundamental issues necessary to deliver the region's water needs, including:

- ensuring the reliability and safety of recycled water systems.
- advising on infrastructure and technology for the recycling of wastewater and stormwater.
- building scientific knowledge into the management of health and safety risks in the water supply system.
- increasing community confidence in the future of water supply.

This report is part of a series summarising the output from the Urban Water Security Research Alliance. All reports and additional information about the Alliance can be found at http://www.urbanwateralliance.org.au/about.html.

C. Davis

**Chris Davis**
Chair, Urban Water Security Research Alliance

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# EXECUTIVE SUMMARY

Rainwater harvesting systems are commonly used as decentralised systems to reduce the need for potable water from the mains water supply and counter the potential rise in water shortages in future. Rainwater tanks are an established feature in individual households in Queensland, with the Queensland Development Code MP 4.2 (QDC MP 4.2) making it mandatory for all households constructed after 2007 to achieve a 70 kL per year water savings target. Installation of a rainwater tank connected internally to toilets and the cold water washing machine tap and to an external tap is one way recommended to achieve this target. A few studies exist on the life cycle costing of individual rainwater tanks. However, literature on the life cycle costing for communal rainwater systems is difficult to obtain as it is an emerging approach to rainwater harvesting in Australia and across the globe.

Communal rainwater harvesting, collection and supply systems are planned and implemented for a group of houses at a cluster or development scale. In such systems, the rainwater from individual homes flows through downpipes to a collection system and then to a communal storage tank. The rainwater is then supplied back to homes with or without treatment through a reticulation system based on its intended use.

This study was aimed at developing an understanding and knowledge on the economics of scale of communal rainwater tank systems. The whole-of-life cost of individual rainwater tanks was also estimated to compare household rainwater systems with communal systems. A net present value (NPV) method of economic assessment was applied for assessing the costs for both individual and communal rainwater tanks, with the main components comprising the capital costs as well as the on-going costs; these include maintenance, replacement and operation costs.

The analysis in this study was based on a conceptual development on flat topography with a housing density of approximately 20 dwellings per hectare. The result highlighted an optimal housing scale of between 192 and 288 households.

Comparisons of NPV costs between individual and communal systems at the optimal household level showed individual tanks (at $8,568) to be lower in cost per household than the communal systems on flat land (at $10,150). However, the costs were more comparable for a communal system on land with at least a 0.5% slope (at $8,770).

The study showed that capital costs of individual rainwater tanks provide the highest cost contribution to life cycle costs, followed by maintenance and replacement costs for individual rainwater tanks. Operational cost contributed the least to life cycle costs, which is in agreement with another study by Stewart (2011). Sensitivity analysis on discount rates showed that lower rates have a greater impact on life cycle costs than higher discount rates.

Analysis of contributing cost components for the communal systems also showed capital cost to be the major contributor to life cycle costs, followed by maintenance cost and, depending on the number of households in the development, operation and replacement cost. Dis-economy of scale for pipe costs was shown with increasing numbers of households (i.e., increasing costs for pipes with increasing house numbers). However, this was counterbalanced to some extent by the economics of scale from storage and treatment units (decreasing cost per household with increasing house numbers).

A desktop analysis to understand the influence of land topography on overall NPV showed that overall costs would reduce by 14% even with a small slope of 0.5%. Sensitivity analysis carried out on discount rates for individual rainwater systems showed that lower rates affected results more than higher discount rates. However, no change was observed in the optimal scale of a communal rainwater tank system. Excluding the treatment system from the analysis (to provide water for non-potable uses only) showed an overall reduction in NPV by 4% at the optimal housing scale, although further analysis is required to investigate the impact of various parameters on the optimal scale of a communal system.

The results presented in this report assist in providing a prediction of the cost breakdown for both individual and communal rainwater systems as well as a methodology for carrying out future economies of scale analysis for different scenarios. Final results showed that, although individual systems cost less than communal systems, the latter's ability to provide a source of potable water could outweigh the higher costs involved.

# 1. INTRODUCTION

South East Queensland (SEQ) has only recently emerged from one of its worst drought periods in recent history; the Millennium Drought, which lasted from 2001 to 2009. At one point in 2007, the three dams supplying Brisbane and its surrounding areas had a combined storage capacity of only 17 percent (SEQWGM, 2010), highlighting the delicate water security situation within this region. With SEQ's population set to grow from the current 3.18 million to 4.59 million (medium series) by 2031 (OESR, 2012), it is anticipated that there will be additional pressure on the existing water supply networks to provide for residential use. Furthermore, with climate change predicted to cause more droughts in the future, SEQ is set to face further challenges in providing sufficient and dependable sources of water for its population.

To counter the potential rise in water shortage in the future, the region has drawn its attention on the need for alternative water supply solutions intended to reduce the need for potable water from the SEQ Water Grid. Such methods could be realised through the implementation of decentralised systems, which involve the collection, treatment and use of rainwater, stormwater, or wastewater at different spatial scales (Cook et al, 2009). Commonly used decentralised systems include rainwater harvesting systems, greywater recycling systems, wastewater recycling systems, stormwater systems, and groundwater bores.

Rainwater tanks are an established feature in individual households in Queensland, with 36% of all households fitted with a tank (Australian Bureau of Statistics, 2010). As per Queensland Development Code (QDC) Mandatory Part 4.2 (MP 4.2), all new Class 1 buildings connected to a reticulated town water supply system after January 2007 are required to achieve a water saving target of 70 kL/household/yr for the SEQ region. This can be achieved by: (a) a rainwater tank; (b) a greywater treatment plant; (c) alternative water substitution measures; or (d) a combination of options a, b, and c. The installation of rainwater tanks is the most common approach to achieve the water saving targets. It can be realised through the installation of a 5 kL tank connected to 100m2 roof or half of the available roof area, whichever is the lesser of the two. Dwellings with rainwater tanks are connected to supply non-potable demand to substitute for potable water, usually to outdoor taps, toilets and washing machine cold water taps. To ensure a continuous supply of water, the rainwater tank should have a back-up supply of either a trickle top-up or automatic switching device.

Another option for household rainwater harvesting system is in the use of communal rainwater tanks, which can collect, store and treat rainwater for either potable or non-potable purposes in a residential development at various scales. With communal tanks fitted mainly for internal household uses, continuous supply must be ensured to avoid disruptions to such uses. This can be either through top-up from municipal mains water or on-site bore supply as, being a climate dependent system, it will be difficult to achieve 100% reliability without a supplementary source. These systems can be used in developments where traditional mains supply is unavailable or difficult to be connected to provide a source of potable water supply for the community. Such a system could also address on-going maintenance issues and potential health risk associated with individual tanks, as the body corporate takes the responsibility of operation and maintenance, unlike individual rainwater tanks where the onus lies on each of the home owners. Based on their suitability, communal rainwater harvesting systems are being considered as potential options for potable applications in greenfield developments to reduce the reliance on fresh water resources.

Considering the reduction in dependence on the mains water supply which could be realised through the implementation of rainwater harvesting systems, it is worthwhile to have an understanding of the costs involved for both individual and communal systems. This investigation has been conducted to develop knowledge on the economics of scale of communal rainwater systems.

## 1.1. Life Cycle Costing Studies

### 1.1.1. Individual Systems

In individual rainwater tanks, costs are directly absorbed by each of the individual home owners; from the installation stage to the recurring costs of operation and maintenance for ensuring on-going services. Depending on the usage of such systems, the overall costs incurred by the householder vary.

A number of studies on the life cycle costing of individual rainwater tanks are available, for example by Marsden Jacob Associates (2007) and Stewart (2011). The former looked into the cost-efficiency of installing rainwater tanks as a substitute for mains water use while the latter assesses the life cycle costs of various existing water supply schemes, including rainwater tanks. Both studies quantified the cost of rainwater based on a per kilolitre (kL) volume basis.

Marsden Jacob Associates (2007) broke costs into the individual components, with tanks accounting 30% of whole-of-life costs and water pumps, including replacements, comprising 35% of the cost component. Other costs included 25% for installation and plumbing and 10% for operation and maintenance. Stewart (2011) showed that capital costs (tanks, pumps, etc) made up the majority of the costs (80% – 82%) followed by pump and tank replacements costs. Operation costs for running a domestic-sized pump made the least contribution to costs.

### 1.1.2. Communal Systems

Literature on the life cycle costing for communal rainwater harvesting systems is difficult to obtain as it is a relatively new and emerging process in Australia and even globally.

In traditional, centralised water or sewage pipe systems, economics of scale is a well established feature of the infrastructure component, often encouraging designers to make their systems as large as possible in the belief that by sharing the cost, lower cost advantages could be passed on to the customers (Clark, 1997). Such a consideration also applied to a communal rainwater system will result in an optimal housing scale (numbers) where the minimum costs would occur, through balancing of the different costs components.

Although not addressing the costs associated with communal rainwater systems, Clark (1997) used a simple sewer design model and historic pipe cost data from South Australia Water to demonstrate the diseconomies of scale which is prevalent in pipe collection systems and how it is counterbalanced by economics of scale of other components in a system to provide total system costs. Analysis into Clark's study by Fane et al. (2002) found that below 500 connections, treatment costs were dominant and an economy of scale existed, whilst beyond 10,000 connections, a slight diseconomy of scale was evident.

Booker (1999) investigated into the economics of greywater collection, treatment and reuse systems, which can be considered a guide to conduct similar assessments. The study also demonstrated a diseconomy of scale in pipe networks affecting system size of more than 12,000 connections, whilst treatment units were the dominating costs for less than 1,200 connections. His study also notes that the cost curves for all the scenarios are quite flat, making it difficult to identify the most economically viable scale to recycle greywater, although minimum costs occured between 1,200 and 12,000 connections. An analysis carried out by Pinkham et al. (2004) states that this flat curve indicates low sensitivity to the size of households for systems within this range. Similarly, in his study, Clark (1997) highlights the relatively low difference in life cycle costs for wastewater systems with between 500 to one million connections, with minimum costs observed at between 10,000 and 25,000 households.

Sensitivity testing on the discount rates indicated modification in life cycle costs, but did not make much difference to the scale of servicing at which the minimum total system cost was located (Clark, 1997). Clark (1997) concludes that whilst his findings are believed to reflect the average situation, local factors will influence costs varying significantly from the averages. Likewise, Pinkham et al. (2004) states that the optimum scale will vary with local conditions and design requirements.

## 1.2. Study Objective

The aim of this study is to conduct an economic assessment of an individual rainwater tank and also the economics of scale of a communal rainwater harvesting system, on the basis of cost per dwelling. To achieve this goal, a desktop study was conducted to quantify the whole-of-life costs of both these systems using the net present value (NPV) method of life cycle costing.

Cost data for the components utilised in the individual and communal systems were obtained from suppliers and a recommended discount rate applied to produce the net present value of the two systems over the selected analysis period. Considerations for carrying out the economic assessment included capital costs of installing the systems, operation and maintenance costs, replacement costs, the life of associated equipment and infrastructure, discount rates as well as other related costs required for the smooth operation of the systems.

Using certain assumptions, conceptual designs of communal rainwater systems for various housing layouts/scale were carried out. Based on the conceptual design, cost data for various system components were sourced. Net present values (NPV) for each of these layouts were estimated to assess the optimum scale at which it will be economically viable to implement communal rainwater systems. Although a certain degree of uncertainty lies in the final outcomes due to the range of assumptions made, the study has used available literature to come up with the best conclusions.

# 2. LIFE CYCLE COSTING

## 2.1. Introduction

Life cycle costing (LCC) is used as an economic analysis tool in the estimation of the total cost of a system over its life span or over the period of service provided (Swamee and Sharma, 2008). The New South Wales Treasury (2004) defines LCC of an asset as *"the total cost throughout its life including planning, design, acquisition and support costs and any other costs directly attributable to owning or using the asset".*

For individual and/or communal rainwater systems, this approach would involve all costs incurred over the analysis period, i.e., the capital costs at the time of construction (e.g., pumps, tanks, pipes, etc.) and the ongoing costs to keep the system in operating condition over the period. Whilst the former cost is the initial outlay to commission the project, the latter cost is incurred on an ongoing basis for the smooth operation and maintenance of a system. An additional recurring cost which also needs inclusion is the price of replacing components at the end of their useful life; also known as replacement costs.

There are a variety of methods to bring these two costs, capital and ongoing, to the same units so that they can be combined to find the life cycle cost of the systems. These are the capitalisation method, the annuity method and the net present value (NPV) method. As the NPV is the most commonly used method to determine current values of future investments (Swamee and Sharma, 2008), this method was chosen over others in this study and is described in the following section.

## 2.2. Net Present Value

The NPV method attempts to determine present values of future investments if the future cost and income are known and a suitable discount rate, which brings future prices to current level, applied. The net present cost, $P_{NC}$, of a component can be derived from Equation 1:

$$P_{NC} = F.(1+i)^{-n} \tag{1}$$

*Where:*

$F$ = future cost, $i$ = discount rate and $n$ = life of the component.

It is common practice to assume that present *(P)* and future *(F)* costs of a component are the same over the analysis period, due to the uncertainties in predicting the future costs. The above equation is true if the life of the component is equal to or exceeds the analysis period, but requires modifications if the component's useful life is less than the analysis period. For instance, if the life cycle of a component is *n* years and the economical analysis is to be carried out over *4n* years, the net present value of the element can be expressed from Equation 2:

$$P_{NC} = P + P.(1+i)^{-n} + P.(1+i)^{-2n} + P.(1+i)^{-3n} \tag{2}$$

For annual costs of expenditure *(A)*, such as maintenance and operation costs, the net present value ($P_{NA}$) can be estimated using the Equation 3:

$$P_{NA} = A.\left[\frac{(1+i)^n - 1}{i(1+i)^n}\right] \tag{3}$$

Hence, a NPV over the life cycle a component can be estimated by applying these equations. For this study, the useful life of the components was considered to be zero beyond the analysis period.

### 2.2.1. Analysis Period

To successfully apply the NPV method of life cycle costing, suitable analysis periods for the individual and communal rainwater systems have to be selected. The length of the analysis should consider the components with the longest life, although if it's useful period is particularly long, it is more appropriate to carry it out for a shorter timeframe (DFA, 2006).

A realistic useful life of a household for a family to fully utilise the benefits of rainwater tanks is 50 years, and this was chosen as the analysis period for the study. Other components used in the communal systems which had longer useful life, e.g. pipes (80 years), were not taken into consideration as life cycle studies carried out over an extended period of time will make future costs insignificant relative to the present cost (DFA, 2006).

### 2.2.2. Life Cycle of Components

The useful life cycle of components is an important factor in carrying out a LCC analysis, as it assists in estimating the number of replacements that are needed for each component over the course of the analysis period. The NPVs of the replacements are included in the final cost analysis and represent an important part of the study.

Estimates on the useful life of components were obtained from product suppliers and literature review from Sharma et al. (2006) and Marsden Jacob Associates (2007). Main components for communal rainwater systems and individual rainwater tanks are shown in Table along with their estimated life cycle.

**Table 1: Useful life of main individual components.**

| Communal Rainwater System | | Individual Rainwater Tanks | |
|---|---|---|---|
| Component | Useful Life (years) | Component | Useful Life (years) |
| Collection and recirculation pipe network | 80 | Rainwater Tank (PVC) | 25 |
| Rainwater Tank (Concrete) | 50 | Household Plumbing | 50 |
| Vertical multistage pumps | 12 | Domestic pump | 10 |
| Holding Tank (PVC) | 25 | | |
| Sand filters and UV | 12 | | |
| Chlorination | 12 | | |

### 2.2.3. Discount Rate

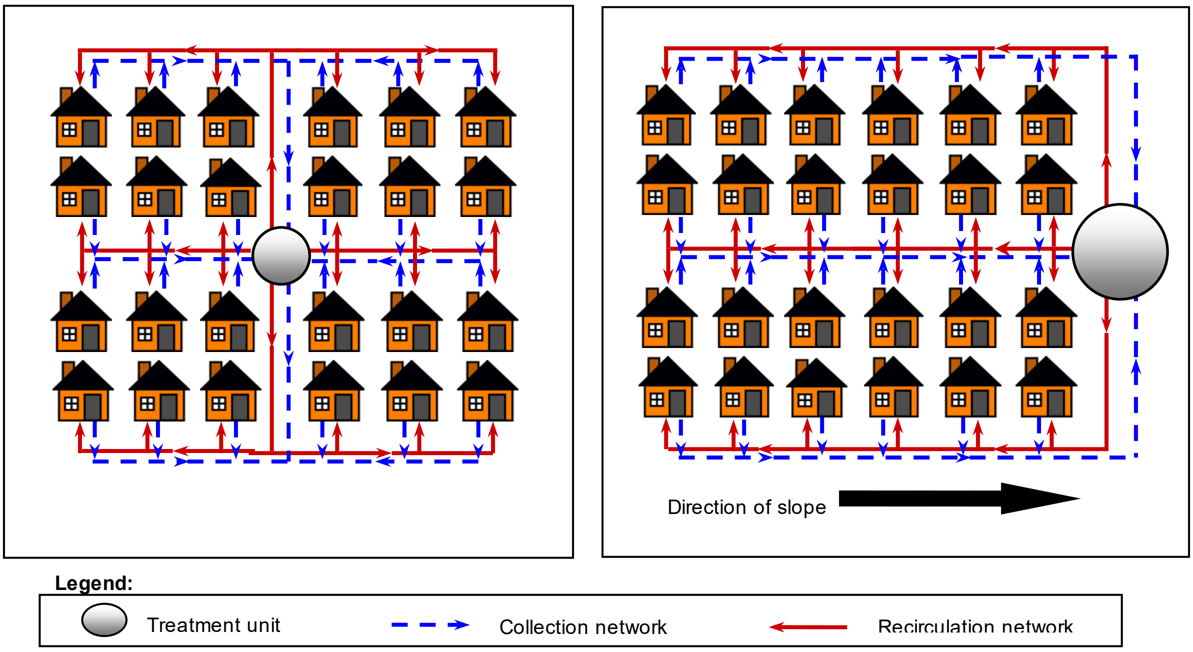
Discount rates describe the reduction in value of a product in the future compared to current prices. Some factors affecting discount rates include market rates of interest and inflation, both of which are usually positive, as well as certain adjustments to account for risks and other factors as highlighted by Australian Government's Office of Best Practice Regulation (OBPR, 2007). Logically, this means that something of a certain current value would be worth less in the future. For example, if a dollar is spent in a year's time, then at a current discount rate of 5% would mean it has a present value of $0.95.

Although there is uncertainty in the appropriate discount rates to be used, the OBPR (2007) recommends a rate of 7%. This uncertainty over the rate to be used may have a significant impact on the calculated NPV. The OBPR (2007) recommends carrying out sensitivity testing on discount rates to establish their significance on final NPV values. Recommended values for sensitivity analysis are at higher rates of 11% and lower rates of 3%. This recommended discount rate has been applied for all analysis carried out in this study, with sensitivity analysis carried out using the proposed values.

# 3. COMMUNAL RAINWATER SYSTEMS DESIGN

## 3.1. Housing Layouts

There are a number of options for the layout of communal rainwater systems with collection and distribution networks, storage and treatment units based on the local topography. For a development in a flat terrain, the storage tank can be located in the centre of the development as shown in Figure 1(a) to minimise the depth of laying rainwater collection pipes as the length of collection pipes to storage tank will be reduced. On the other hand, the storage tank and treatment unit can also be located on one side of the development based on the direction and magnitude of slope, as shown in Figure 1(b). The pipes in the collection system are laid at a gradient to develop sufficient gravitational energy for the rainwater to flow. As the length of pipes increases, so will the depth of pipe laying and pipe sizes.



(a) Treatment unit at the centre of the development (b) Treatment unit at the end of the development

**Figure 1: Examples of layouts for communal rainwater systems.**

In this study, a flat topography scenario of development has been considered for analysis with the storage tank and the treatment unit located in the centre of the development. Collection and recirculation pipes are designed with different hydraulic approaches as the former is a gravity flow based network and gradually runs deeper, whilst the latter is a pressurised system where more pressure will be required to supply water at greater distances. Main pipes for both systems are buried along the footpath or in green strip of the roads; lateral connections off the houses are connected to main pipes.

Housing development layouts ranging from 4 to 576 allotments were developed to assist in the economic analysis. Plotting final costs values on a per household basis against the associated housing layouts would provide the optimal number of houses for which the cost per house would be economical.

Housing plot sizes and road widths were measured from new developments in Fitzgibbon and Bracken Ridge in SEQ, using the online Google Earth software. Allotment sizes were found to be approximately 16 by 25 metres (front width x length) and road widths about 8 metres. Housing layouts were developed using these measurements for 4, 8, 16, 24, 48, 96, 192, 288, 384 and 576 homes to assist in the economic assessment and are shown in Figures 2 and 3. The density of the housing development was maintained at around 20 dwellings per hectare.

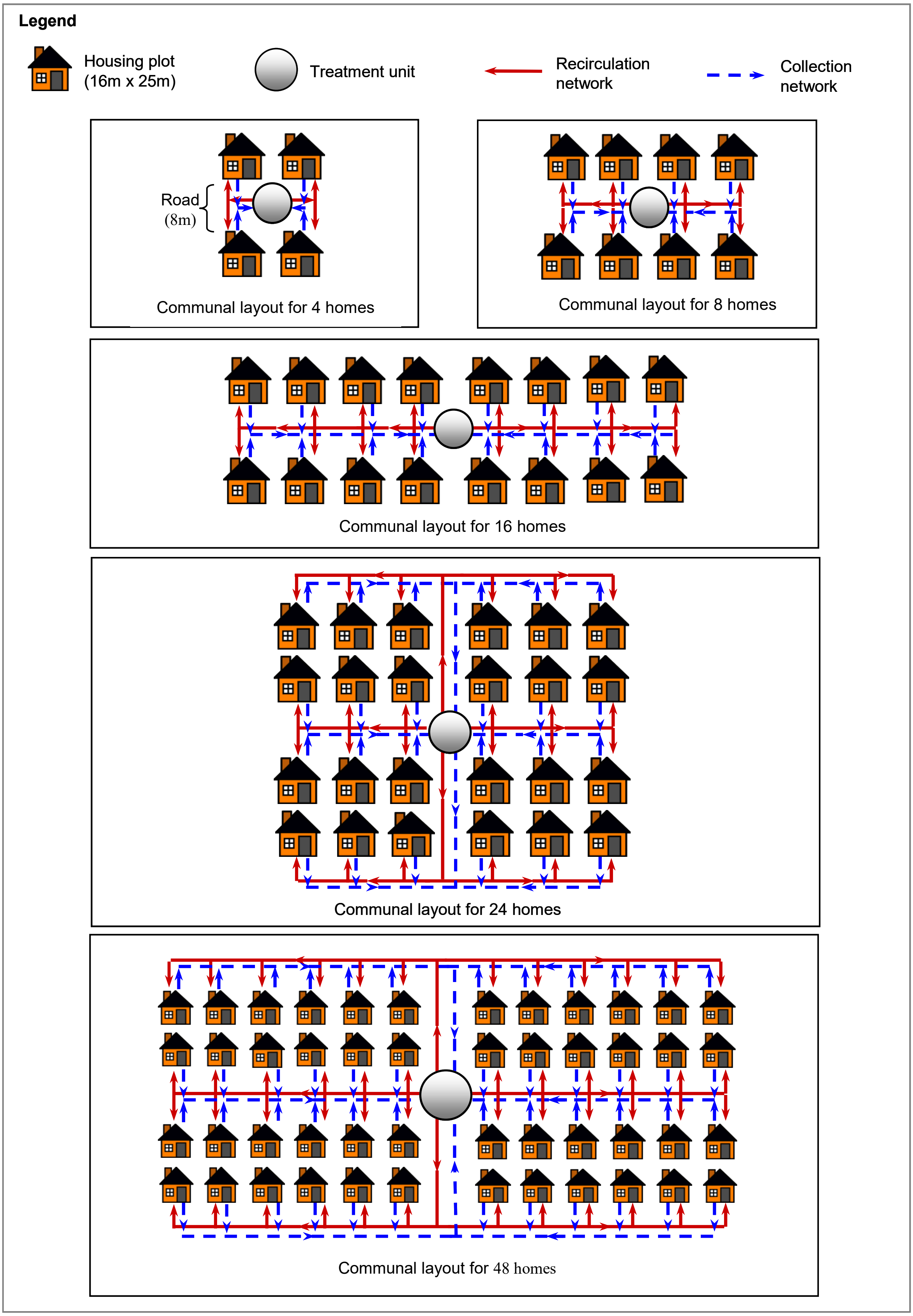
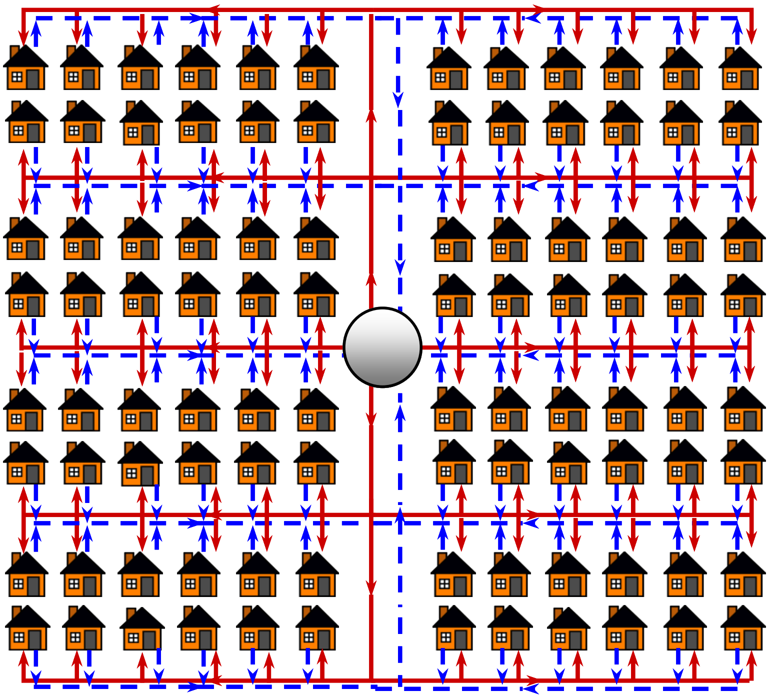
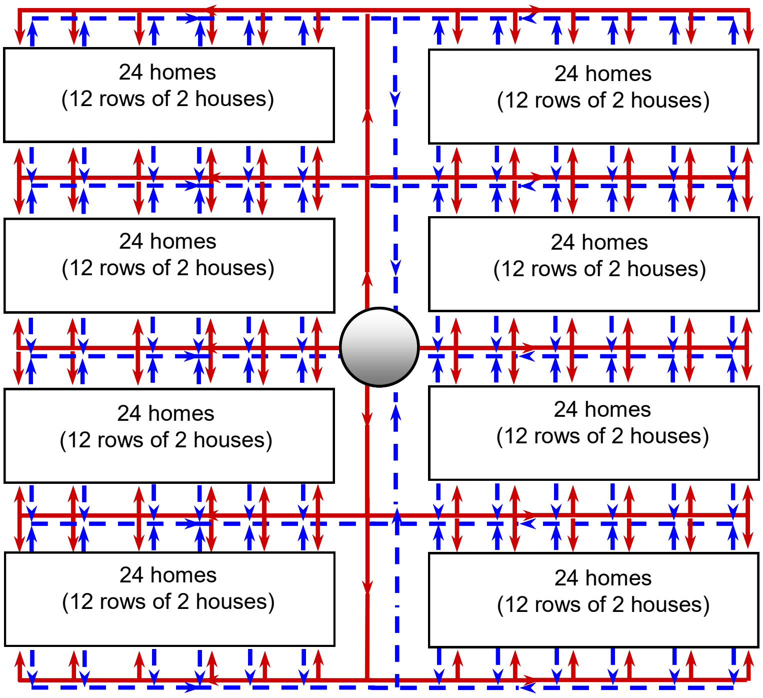


**Figure 2:** **Housing layouts used in the study (4 – 48 homes).**



Communal layout for 96 homes:
- Houses divided into 8 sections, with 6 rows of 2 houses each



Communal layout for 192 homes:
- similar to 96 homes but with 12 rows of 2 homes in each section instead of 8 rows of 2 houses (See layout of 96 homes)

Communal layout for 288 – 576 homes similar to 192 homes layout:
- 288 homes – 36 houses in each section, i.e. 18 rows of 2 houses;
- 384 homes – 48 houses in each section, i.e. 24 rows of 2 houses;
- 576 homes – 72 houses in each section, i.e. 36 rows of 2 houses.

**Figure 3: Housing layouts used in the study (96 – 576 homes).**

## 3.2. Design of the Communal System

As there are no specified guidelines on the design of a communal rainwater harvesting system, to ensure the most efficient design for the housing layouts, guidance has been taken from various literature and sources, including the design of the communal rainwater tank system at Capo di Mote (CDM), a residential development at Mount Tambourine in the hinterland of the Gold Coast, SEQ.

Household roofs are the catchments for rainwater, which is transported via a gravity collection pipe network to a large common rainwater tank for storage. However, for housing layouts where the collection pipes have a depth of more than 1.5 metres at its outflow, the rainwater will first enter a sump well. Rainwater is pumped from a sump pump to the storage tank which is designed partially underground based on local conditions.

The topography of the area is assumed to be flat. A transfer pump will pump the rainwater through a series of treatment units (filtration, UV disinfection and chlorination), to treat the stored water to potable standards, and then into a holding tank. From there a recirculation pump will distribute water to the households as per the demand (see Section 3.2.1). A basic diagram of the layout is shown in Figure 4.

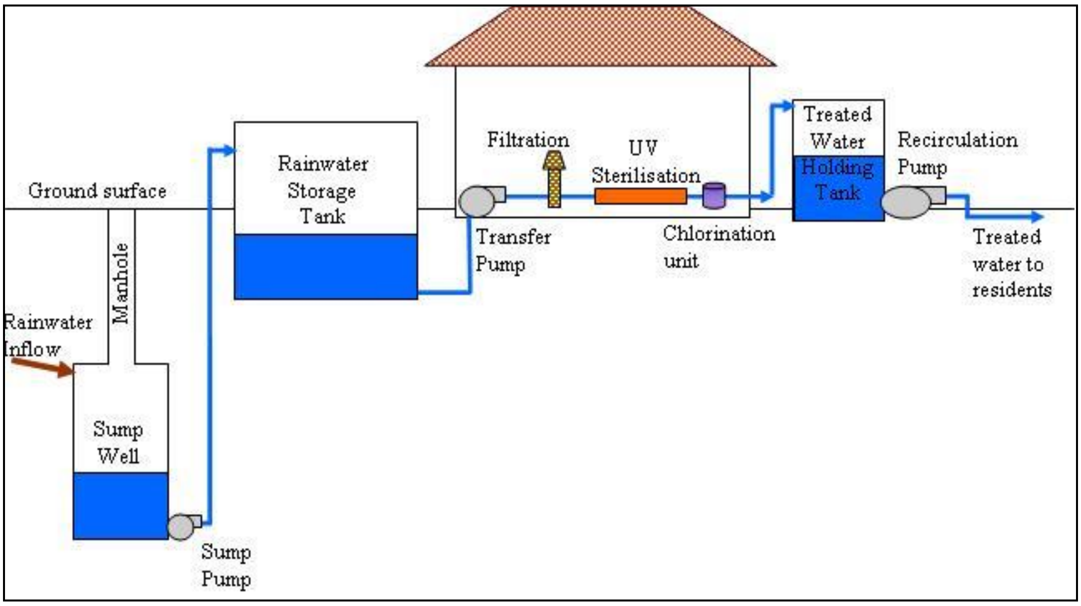


**Figure 4: Basic layout of communal rainwater treatment system.**

Basic design of pipes, tanks, pumps, and treatment units for the 10 different layouts of communal rainwater systems were conducted to provide guidance on the scale issues and are described in the subsequent sections.

### 3.2.1. Water Usage

The analysis for all households is based on the consideration that the systems are to be used for potable use. Beal *et al.* (2010) undertook the SEQ Residential End Use Study (SEQREUS), which focused on estimating per capita water usage for different local government areas in SEQ and also provided an end use breakdown on the average per capita water requirements for the region. The breakdown in per capita consumption from the SEQREUS is displayed in the Figure 5.

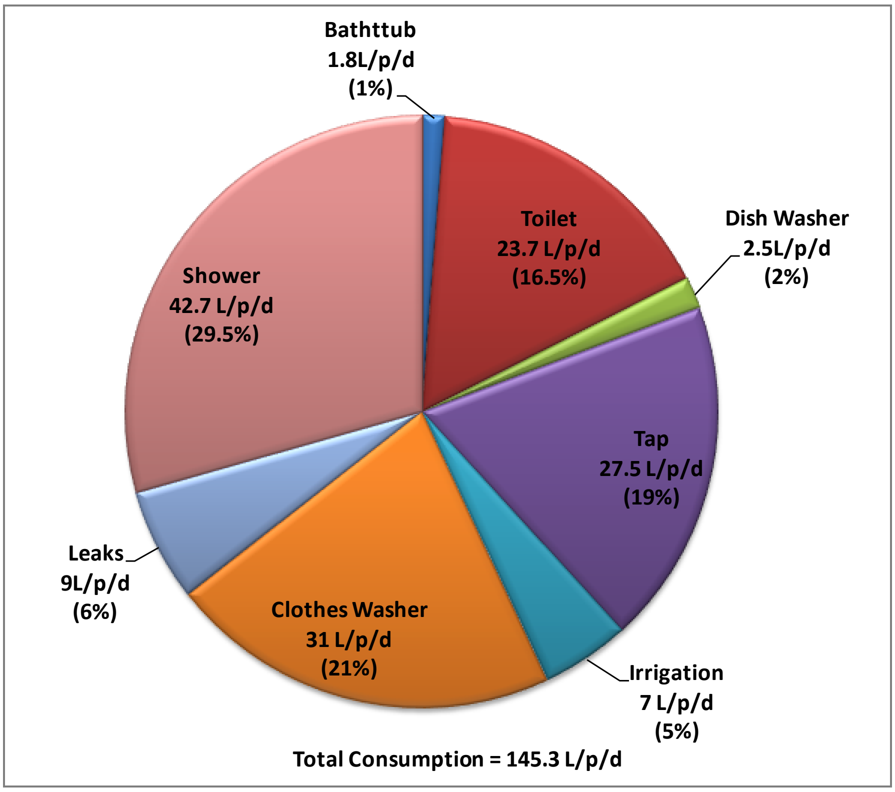


**Figure 5: Breakdown in average daily per capita water consumption for SEQ (Beal *et al*, 2010).**

Potable water to be used includes taps, shower, dishwasher and hot water for laundry (Table 2). Shower usage was taken as the combined consumption of shower plus bathtub usage, whilst hot water for laundry was estimated to comprise 25% of the total water used for laundry (EBMUD, 2008).

**Table 2: Estimated potable water usage for the analysis.**

| Appliance | Water Usage (L/p/day) |
|---|---|
| Taps | 27.5 |
| Shower | 44.5 |
| Dishwasher | 2.5 |
| Hot water for Laundry<br>(25% of total laundry use; i.e. 25% of 31 L/p/day) | 7.8 |
| **Total potable water demand (L/p/day)** | **82.3** |

## 3.2.2. Estimating Peak Flow Demands

Estimation of peak water demand is an important part for the design of pipes and pumps in any water supply network. However, it is difficult to estimate water demands accurately due to a number of factors including: climate; economic and social factors; pricing; land use; and industrialisation of area (Swamee and Sharma, 2008). Moreover, water demand is not normally constant throughout the day and usually has two peaks, one in the morning and another in the evening. For this reason, peak hour factors are required for the design of a pipe network system to meet such demands.

Peak hour factor (PHF) is defined as the ratio of peak hour demand on peak day over average hour demand in the same day (Swamee and Sharma, 2008); with peak day demand being the maximum day demand over a 12 month period. Table 3 shows a range of peak factors in DERM (2010) water supply and sewerage planning guidelines.

**Table 3: DERM (2010) guidelines on Peak Hour Factors (PHF) for water and sewerage supply.**

| Equivalent Persons | Peak Hour Factor (PHF) |
|---|---|
| > 5000 | 3.6 – 4.0 |
| < 5000 | 3.6 – 4.5 |
| Arid areas (where internal water use is less than 30% of total water consumption) | 3.6 – 5.0 |

These peak factors are normally for larger areas and higher peaking factors may be required for smaller schemes, depending on the standard of service available (DERM, 2010). The Water Services Association of Australia (WSAA) in its Water Supply Code of Australia (2004) provides PHF of:

- 2 for populations over 10,000;
- 5 for populations under 2000; and
- for populations between 2000 and 10,000, peak hour factors can be interpolated.

Design guidelines recommend higher peak factors, hence higher peak flows, for smaller schemes with a much lower population. Swamee and Sharma (2008) state that these peak factors are guidelines only and local information on flow data are required to be collected for peak factor selection. For this reason, peak flows were analysed from three studies of diurnal patterns carried out in SEQ including:

- A peak flow usage of 35 L/hr/person, or 0.583 L/min/person, from Beal *et al.* (2011);
- Analysis on diurnal pattern for the CDM site of communal tanks for 46 homes produced peak flows of 180 L/min for 75 persons translating to 0.48 L/min/person (Cook *et al.*, 2012); and
- 20 homes analysis on diurnal patterns in SEQ by Umapathi *et al.* (2012) showed each home had a peak flow of on average of 10-12 L/min/3.1 persons; approximately 3.23 – 3.87 L/min/person.

The overall housing profile of Capo di Monte (CDM) differed to the normal profile of SEQ, with the population made up of older people and having a lower than average occupancy rates of 1.65 persons per household; compared to SEQ's median age of 36 years and an average occupancy rate of 2.6 (OESR, 2012). It highlights that the CDM analysis would be unsuitable for use as there would be differing trends in the water usage due to the difference in the demographic profile of the area.

To further analyse the peak flows for different numbers of households, households from the analysis of rainwater tanks from 20 homes (Umapathi, 2012) were chosen at random and their total water usage added up; i.e. potable and non-potable usage. This was then converted to flow rates per person and plotted against the respective number of occupants. Figure  shows the relation between peak flow rates and total number of occupants, with Beal *et al.* (2011) findings included. Total water usage was used in the peak flow plots, rather than just using peak potable flow data from potable usage, as this would ensure that there would be no underestimation in peak flows for the respective housing layouts.

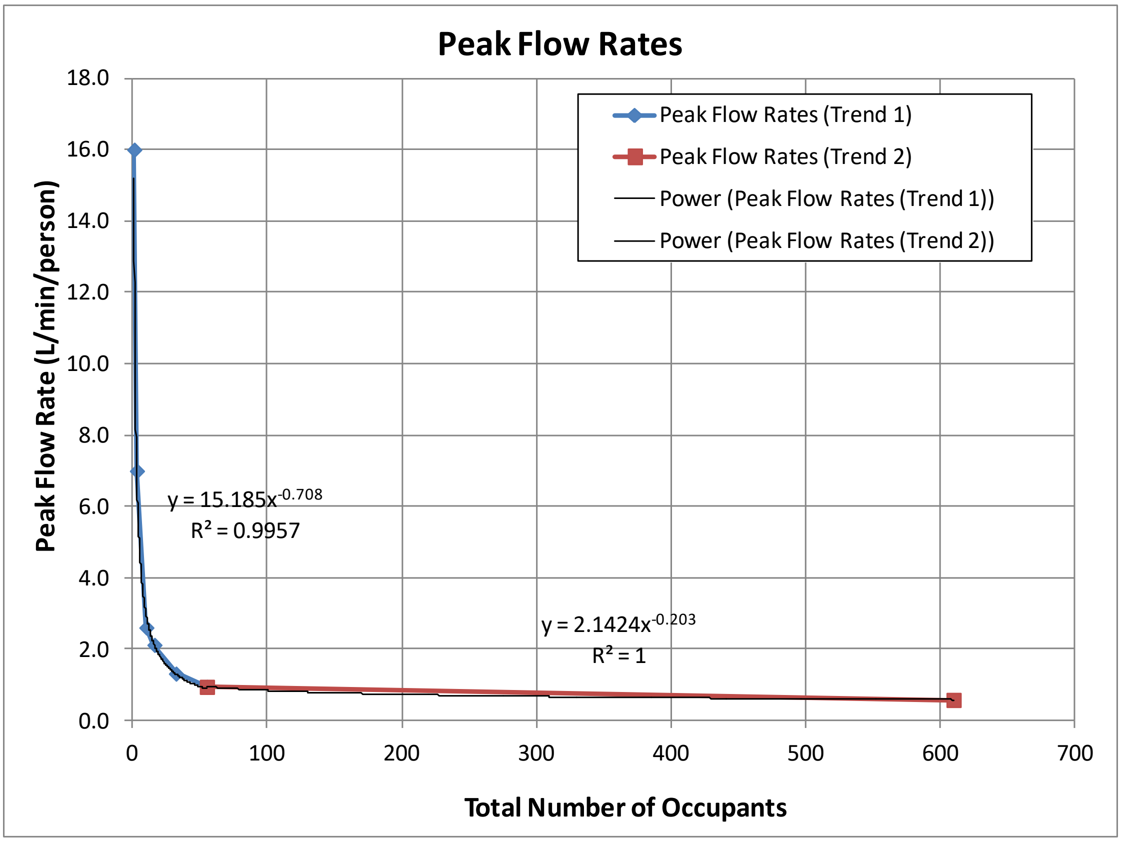


**Figure 6:** **Peak Flow Rate vs Total Number of Occupants.**

Adding a trendline to the plot provided two expressions for peak flow rates with respect to the total number of occupants. The resulting equations, $y = 15.185x^{-0.708}$ ($R^2 = 0.9957$), for 1 to 55 people, and $y = 2.1424x^{-0.203}$ ($R^2 = 0.9957$), for between 55 and 610 people, were used to calculate peak flows for the different household layouts. Beyond 610 people, an average of 0.583 L/min/person (Beal *et. al.,* 2011) was used, as this was the maximum population analysed and there is uncertainty to what the trend may be beyond this population size. Table shows the calculated peak values, with a 20% safety factor to account for any unexpected rise in peaks, used in the sizing up of the recirculation pumps for the analysis; with occupancy rates of 2.6 heads per household.

**Table 4:** **Peak flow rates of housing layouts in arbitrary model.**

| No. Of homes in arbitrary model | Total Occupants (rate = 2.6) | Peak Flow rate (L/min/person) | Peak Flow rate (L/hr/person) | Peak Flow Rate ~ factor of 20% (L/hr/person) | Peak Flow rate to be supplied by pump, ($m^3$/hr) |
|---|---|---|---|---|---|
| 4 | 10.4 | 2.893 | 173.6 | 209 | 2.2 |
| 8 | 20.8 | 1.771 | 106.3 | 128 | 2.7 |
| 16 | 41.6 | 1.084 | 65.1 | 79 | 3.3 |
| 24 | 62.4 | 0.926 | 55.5 | 67 | 4.2 |
| 48 | 124.8 | 0.804 | 48.3 | 58 | 7.3 |
| 96 | 249.6 | 0.699 | 41.9 | 51 | 12.8 |
| 192 | 499.2 | 0.607 | 36.4 | 44 | 22.0 |
| 288 | 748.8 | 0.583 | 34.98 | 42 | 31.5 |
| 384 | 998.4 | 0.583 | 34.98 | 42 | 42.0 |
| 576 | 1497.6 | 0.583 | 34.98 | 42 | 62.9 |

### 3.2.3. Roofs and Downpipes

Although QDC MP4.2 provides guidance on minimum roof areas for individual houses, there are none provided for communal rainwater systems. Hence, the assumption taken in this report is that roofs are fully connected to the network via downpipes; i.e. 100% connectivity, with household connector from down pipes to the rainwater collection system governing flow. The hydraulic analysis of Capo di Monte (CDM) rainwater collection system is provided in Cook *et al.* (2012) (see Appendix A) and provides some of the assumptions used in this report. A brief outline of the CDM hydraulic analysis is as follows.

- The SWMM hydraulic model package was used to carry out the hydraulic analysis of the rainwater collection system of CDM with model calibration and validation done to provide confidence in the model's outcomes.
- Calibration and validation runs carried out with effective roof areas (runoff coefficients) of 85%, 87.5% and 90% matched well with observed data, with the Rational Method further validating the outcomes; with 87.5% being a reasonable estimate for runoff coefficient.
- Gutters and downpipes were shown to be designed to convey flows up to a 20-year ARI storm.
- However, the structure controlling the amount of flow entering the collection system is the connector of the downpipes to the pipe network.
- Calibration and validation showed that these connectors limited the maximum rainfall intensity harvested to 2 mm/5 min (2.4 mm/6 min).

Roof areas for new dwellings, from measurements of new housing plots, were found to be 220 m$^2$; similar in size to CDM. Downpipes from roofs will be connected to in-ground pipes, which in turn will be connected to the main collection network through connectors. For the purpose of this study, gutters and downpipes were assumed to be adequately designed to a 20-year ARI, 5-minute duration storm as recommended by DERM (2007). Design peak flows entering into the rainwater collection can be calculated using the Rational Method (DERM, 2007) in Equation 4:

$$Q = \frac{C.I.A}{3.6 \times 10^6} \tag{4}$$

*Where:*

Q = peak flow rate (m$^3$/s), C = runoff coefficient, I = intensity of rainfall (mm/hr) and A = area of catchment (m$^2$).

The connectors are assumed to limit the peak rainfall intensity contribution to 2.4 mm/6 min, with reference to the design at CDM. Applying the Rational Method with this rainfall intensity and roof area of 220 m$^2$, and the runoff coefficient taken as 87.5%, due to evaporation, wind effects, splashing and spillage (as used in the CDM analysis), the maximum flow able to enter the collection pipe from each dwelling is 0.00128 m$^3$/s. This flow has been used to size the rainwater collection system.

### 3.2.4. Rainwater Collection Pipes

Rainwater collection pipes will convey roof water to the storage tank via gravity. Manning's equation was applied in sizing the collection pipes using the following pipe diameters; 75mm, 100mm, 150mm, 225mm, 300mm and 375mm, with polyvinyl chloride (PVC) as the chosen pipe material (Manning roughness value = 0.01). The Manning's equation (Rossman, 2010) is given in Equation 5 as:

$$Q = \frac{1}{n}.A.R^{2/3}.S^{1/2} \tag{5}$$

which expresses the relationship between flow rate (Q), Manning's roughness coefficient (*n*), cross-sectional area (A), hydraulic radius (R), and slope (S).

The slope of the pipes is taken to be 0.5% as this gives a minimum pipe full velocity of > 0.7m/s, as outlined in DERM's (2007) QUDM manual, for most of the pipes (Refer to Table 7.11.1, *"Acceptable flow velocities for pipes and box sections*", DERM, 2007). Calculated sizes of pipes and their respective required lengths for each housing layout are shown in Table 5.

**Table 5: Pipe sizes and lengths for housing layouts.**

| No. Houses | Length of Pipes Required (m) | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Lateral Pipe (75mm) | | | Main Pipe (<1.5m depth) | | | | Main Pipe (1.5m - 3.0m Depth) | | | | Main Pipe (3.0m - 4.5m Depth) | | |
| | <1.5m (Depth) | 1.5m - 3.0m (Depth) | 3.0m - 4.5m (Depth) | 75 mm | 100 mm | 150 mm | 225 mm | 225 mm | 300 mm | 375 mm | 225 mm | 300 mm | 375 mm | 450 mm |
| 4 | 16 | 0 | 0 | 0 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 8 | 32 | 0 | 0 | 0 | 32 | 16 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 16 | 64 | 0 | 0 | 64 | 80 | 140 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 24 | 96 | 0 | 0 | 64 | 144 | 172 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 48 | 192 | 0 | 0 | 64 | 160 | 304 | 140 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 96 | 592 | 192 | 0 | 64 | 224 | 560 | 188 | 0 | 116 | 0 | 0 | 0 | 0 | 0 |
| 192 | 592 | 176 | 0 | 64 | 224 | 832 | 480 | 396 | 116 | 0 | 0 | 0 | 0 | 0 |
| 288 | 592 | 560 | 0 | 64 | 224 | 832 | 480 | 1188 | 168 | 116 | 0 | 0 | 0 | 0 |
| 384 | 592 | 944 | 0 | 64 | 224 | 832 | 480 | 1456 | 860 | 0 | 116 | 0 | 0 | 0 |
| 576 | 592 | 1150 | 560 | 64 | 224 | 832 | 480 | 1728 | 1056 | 0 | 256 | 912 | 284 | 116 |

**Depth of Pipes**

Depths of pipes are an important factor as they affect the pipe cost, with deeper laid pipes costing more. (See section 4.2.1 for more details on pipe costs). To estimate depths of pipes laid, the minimum cover to the crown of the pipe has been taken to be 0.6 metres with pipe gradients being 0.5%. Only depths of main pipes are evaluated, with lateral connected pipes taken to be at the depth of the main pipe to which it is connected.

### 3.2.5. Rainwater Storage Tank

UVQ (Mitchell and Diaper, 2006), an urban water balance and contaminant balance analysis tool, was used in sizing the rainwater storage tanks. Rainfall intensity data from 1991 to 2010 at Brisbane Airport (Station ID: 040842) was obtained from the Bureau of Meteorology (BoM) to conduct water balance modelling for estimating tank sizes.

With the connectors limiting peak rainfall intensity to 2.4 mm/6 min, the pluvial data was censored such that the maximum allowable rainfall intensity available for rainwater harvesting was set at this value. This will ensure that only rainfall intensities lower than or equal to 2.4 mm/6 min will contribute to the rainwater tank, as would be the case in an actual scenario. Beyond this, flow is assumed to overflow into the stormwater system.

From the results of the UVQ modelling, a range of tank sizes along with their average annual volumetric reliability, which is the tank's ability to satisfy demand from the active storage, for four households was plotted to determine the best sized tank to be used for the communal system (Figure 7). It is assumed that all households have the same daily demand pattern. The total demand per capita per day was found to be 82.3 L, from the breakdown illustrated in Table 2.

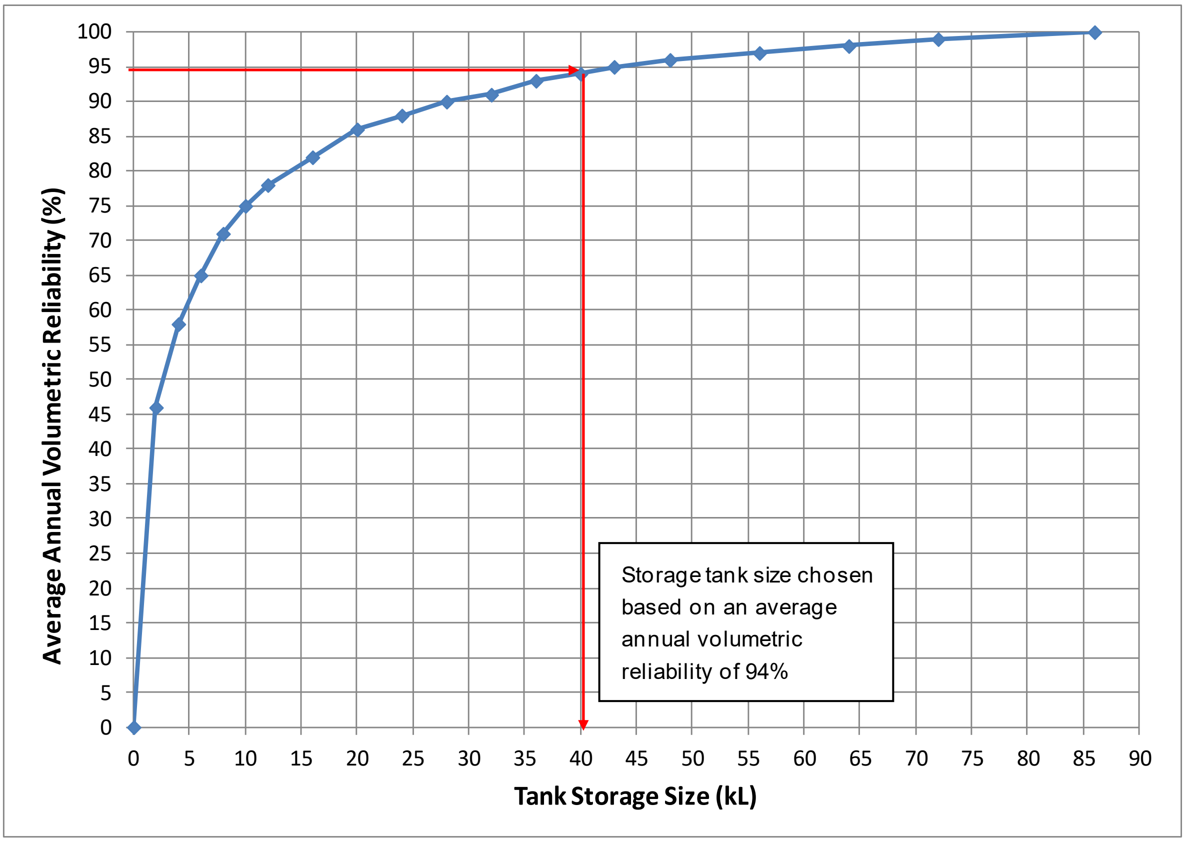


**Figure 7:** **Volumetric reliability for various sized tank storages for 4 households.**

As potable water demand is considered to be constant for all scenarios, sizing of tanks for cumulative housing layouts has been considered on a linear scale; e.g., if the size of a tank for four households is 10 kL, then size of 16 households is 40 kL, 100 kL for 40 households, etc. The size of a storage tank, based on an annual average reliability of 94%, is estimated to be 40 kL for four households. The chosen reliability (94%) is the approximate point on the graph where the gain in the tank's reliability becomes marginal with increase in tank storage size (Figure 7). In addition, with the communal system to be used as a potable source of water, a high reliability is required to maintain a constant water supply to tenants. However, to attain 100% reliability, the tank size would have to be increased to 85 kL, which is more than twice the chosen tank volume. Such a tank size would take up approximately double the space and incur higher start-up costs and there has to be a trade-off in providing a tank of decent reliability against the cost factors. Furthermore, if a probability distribution is conducted, then it may be possible to achieve 100% reliability, however, this has not been taken into consideration in this analysis. In any case, the remaining 6% is assumed to be provided from the back-up supply from other sources, e.g., bore water (as in the case for CDM) or mains water supply. Estimated collection tank sizes for the rest of the layouts are shown in Table 6.

**Table 6:** **Approximate sizes of collection tanks.**

| **No. of Households** | 4 | 8 | 16 | 24 | 48 | 96 | 192 | 288 | 384 | 576 |
|---|---|---|---|---|---|---|---|---|---|---|
| **Approx. storage size (kL)** | 40 | 80 | 160 | 240 | 480 | 960 | 1,920 | 2,880 | 3,840 | 5,760 |

### 3.2.6. Sump Well and Pump

Sump wells are required to avoid deep excavations for construction of rainwater storage tanks, which would be a significant initial expense. For the purpose of this study, sump wells were required for pipe outlets exceeding 1.5 m depth, i.e., they were required for housing layouts with more than or equal to 48 homes. From DERM (2010) Planning Guidelines for Water Supply and Sewerage, a wet well storage capacity for sewage treatment is provided by Equation 6:

$$Wet\,Well\,Storage\left(m^3\right)=\frac{0.9\times Pump\,rate\ (L/s)}{N} \tag{6}$$

*Where:*

N = number of pump starts per hour; 12 starts for motors less than 50 kW and 5 starts for more than 50 kW. Equation 7 is also provided to estimate the pump rate:

$$Pump\,Rate\ (L/s)=C_1\times ADWF\ (L/s) \tag{7}$$

*Where:*

$C_1$ = 15 x (Equivalent Population)$^{-0.1587}$, not lower than 3.5 and ADWF = Average Dry Weather Flow.

Assuming the peak flow of rainwater (1.283 L/s/household) from each of the households to be the ADWF and pump sizes used are no bigger than 50 kW, the sump well size for rainwater storage tanks can be estimated using Equation 6. Although the formula gives feasible estimates of storage capacities of sump wells for each of the housing layouts, pump sizes are grossly overestimated as shown in Table 7.

**Table 7: Sump well and pump sizes from DERM (2010) guidelines.**

| No. of Homes | Peak Flow (L/s) | $C_1$ | Pumping Rate (L/s) | Pumping Rate ($m^3$/hr) | Sump Well Size ($m^3$) |
|---|---|---|---|---|---|
| 48 | 62 | 6.97 | 430 | 1,546 | 32.2 |
| 96 | 123 | 6.25 | 770 | 2,771 | 57.7 |
| 192 | 246 | 5.60 | 1,379 | 4,964 | 103.4 |
| 288 | 370 | 5.25 | 1,939 | 6,982 | 145.5 |
| 384 | 493 | 5.01 | 2,470 | 8,894 | 185.3 |
| 576 | 739 | 4.70 | 3,475 | 12,509 | 260.6 |

The equation oversizes the pump rates since it does not allow for overflow from the sump well due to water being pumped out at a higher rate than the inflow into the well. This is not necessarily required in the design of a sump well for rainwater harvesting purpose. Instead of having a continuous removal of rainwater, as it flows in, it seems sensible to treat the well as an additional storage facility, with the sump pump removing the water based on its pumping rate. During days of heavy rainfall, overflow from the sump well would be allowed as well.

With this in mind, a modified methodology of using the UVQ tool to size up the sump wells was created, using the same rainfall data utilised in sizing the rainwater storage tanks. In this instance, the size of the sump well is represented by the *"Storage Capacity"* parameter of the rainwater tank in the UVQ tool. The daily volume extracted by the sump pump from the sump well in times of rainy days is represented by the *"Water Usage"* parameter. The flow rate of the sump pump was chosen based on this water removed and a pump was selected based on this rate and the required head.

To ensure that the pump fills the rainwater tanks with the yearly required volume of rainwater, the annual demand obtained from the rainwater tank sizing should match with this method of sump well sizing. In addition, the amount of spillage for both methods should also closely match as further validation that the sump well is sized to the necessary requirements. This method will allow spillage to occur as the extraction rate of the pump could be at times lower than the inflow rate into the sump well. Also, it will provide allowance for the water to be stored in the well after the rainfall event has stopped, even whilst the pump is running. Sizes of sump wells and pumps are listed in Table 8.

**Table 8: Sump well sizes with associated pump rates.**

| No. of Houses | Rainwater tank sizing | | Sump well and pump sizing | | | |
|---|---|---|---|---|---|---|
| | Modelled Spillage (kL/year) | Actual Supply (kL/year) | Sump Well Size (kL) | Pump Rate per day (kL/day) | Modelled Spillage (kL/year) | Supplied water (kL/year) |
| 48 | 4,447 | 3,519 | 60 | 29.3 | 4,416 | 3,572 |
| 96 | 8,894 | 7,038 | 80 | 73.6 | 8,897 | 7,080 |
| 192 | 17,752 | 14,111 | 100 | 177.7 | 17,842 | 14,112 |
| 288 | 26,682 | 21,115 | 120 | 284.5 | 26,743 | 21,188 |
| 384 | 35,575 | 28,153 | 140 | 394.4 | 35,554 | 28,354 |
| 576 | 53,363 | 42,230 | 180 | 606.6 | 53,453 | 42,409 |

### 3.2.7. Treatment Unit

The treatment unit will consist of a transfer pump pumping rainwater through a sand and carbon filter, UV sterilisation and chlorination to ensure water is treated to potable standards, before being stored in a holding tank for residents' use. Water would be filtered to at least 10 microns in the filters before being passed through the UV disinfectant, which will destroy bacteria, viruses and algae residing in the water. The final treatment, using chlorination, will ensure complete sterilisation and that the stored water is fit for potable use.

### 3.2.8. Holding Tank

As there were no guidelines available for sizing tanks to store treated rainwater, the site at CDM was evaluated and was found to be capable of providing its residents with approximately 4.5 days supply of water (daily demand of 5.33 kL) without any top-up to its 23.7 kL holding tank, provided it is full at the start. Holding tanks for various layouts have been designed to store approximately three days worth of potable water supply without top-up.

Average potable water consumption estimates per person per day were required in the calculation of holding tanks sizes. The total potable water usage of 82.3 L/p/d was used in sizing the treated water holding tanks for the different housing plans in the arbitrary model, with approximate tank sizes shown in Table . At this stage linear projections were made for the sizing of holding tanks, however different demand generators and variability in household demand could be considered to estimate the tank size. These could be smaller than the one estimated based on linear projections.

**Table 9: Estimated holding size tanks.**

| Households | Occupants | Total Demand (kL) | Size of Holding Tank (kL) |
|---|---|---|---|
| 4 | 11 | 0.91 | 2.7 |
| 8 | 21 | 1.73 | 5.2 |
| 16 | 42 | 3.46 | 10.4 |
| 24 | 63 | 5.18 | 15.6 |
| 48 | 125 | 10.29 | 31 |
| 96 | 250 | 20.58 | 62 |
| 192 | 500 | 41.15 | 123 |
| 288 | 749 | 61.64 | 185 |
| 384 | 999 | 82.22 | 247 |
| 576 | 1498 | 123.29 | 370 |

### 3.2.9. Recirculation Pipe Sizes

From Gold Coast City Council's (2008) Planning Scheme Policies' guidelines, the minimum diameter of potable water mains (recirculation) pipes is to be 100 mm throughout all the household layouts. Checks have been carried out to ensure that the pipe diameters suffice for all the layouts using DERM (2010) guidance, which states that the velocity for the potable line is not to exceed 2.5 m/s.

### 3.2.10. Pump Sizing

All recirculation pumps selected for the different housing layouts were from the Grundfos variable speed range. These pumps were chosen as they can automatically change impeller speeds, depending on differing flow demands for required heads, thus providing a higher pumping efficiency. The Grundfos single speed range was chosen for the transfer pump and the sump pump, as there is no variation in water demand in their usage, and furthermore, it is a cheaper option than the variable speed range.

Pumping head requirements for each of the housing layouts can be defined using the following modified Bernoulli's equation (Equation 8):

$$H_p = Z_L + H_L + H_T - Z_p \tag{8}$$

*Where:*

$H_p$ = required pump head (m), $Z_L$ = Elevation of pipe outlet (m), $H_L$ = Head Loss in pipe (m), $H_T$ = terminal head, or minimum pressure*, at property (m), $Z_p$ = Elevation of pump (m). **In accordance with Gold Coast City Council (2008) Planning Scheme Policies, the minimum pressure, or terminal head, at the property boundary for potable water is taken to be 22 meters.*

Swamee and Sharma (2008) note that pipe head loss consists of two parts; head loss due to friction and head loss due to the change in shape of the pipeline (also known as minor losses). Minor losses are neglected in the conceptual design of pipe networks. Thus, it can be assumed that friction losses are the main losses observed in the pipe networks and account for the main head loss in them. Friction losses are given by Darcy-Weisbach formula (Swamee and Sharma, 2008) (Equation 9):

$$H_L = \frac{8.f.L.Q^2}{\pi^2.g.D^5} \tag{9}$$

*Where:*

f = friction factor, L = length of pipe (m), Q = flow in pipe (m), g = gravitational acceleration (m/s$^2$) and D = diameter of pipe (m).

Sizing of recirculation pumps was done through the online Grundfos (2012) WebCaps application, which allows users to input flow and head requirements. As well as recommending pumps for the desired settings, the application also provides the approximate shaft and pump efficiencies of the pumps as well as being a validation tool for the manually calculated hydraulic power. The hydraulic power, $P_H$ (kW), required in pumping the water to the residents can be calculated from Equation 10:

$$P_H = \frac{\rho.Q.g.H_p}{1000} \tag{10}$$

*Where:*

$\rho$ = density of water (kg/m$^3$), Q = flow (m$^3$/s), g = gravitational acceleration (m/s$^2$) and $H_p$ = required pump head (m).

The pump efficiency ($\eta_p$) is the ratio between the hydraulic power ($P_H$) and the power delivered by the pump shaft ($P_2$). The power delivered by the shaft is read from the pump curve of the chosen pump. The pump efficiency is given by Equation 11:

$$\eta_p = \frac{P_H}{P_2} \tag{11}$$

Similarly, the shaft efficiency ($\eta_s$) is the ratio of energy delivered by a pump's shaft ($P_2$) to the energy supplied to drive it, i.e., the mains power ($P_1$), both of which are read from a pump curve, and is given by Equation 12:

$$\eta_s = \frac{P_2}{P_1} \tag{12}$$

The overall efficiency ($\eta_o$) of the pump is essentially a multiple of the pump ($\eta_p$) and the shaft efficiency ($\eta_s$). Table shows the chosen recirculation pumps for each of the housing layouts along with their respective power requirements and efficiencies. Two pumps were required in the 576 house model to provide for the demands of the residents.

**Table 10: Pump outputs for selected recirculation pumps.**

| No. of homes | Peak Flow Rate ($m^3$/hr) | Pump Head, $H_P$, (m) | Pump Model | Pump Size, (W) | Hydraulic Power $P_H$, (W) | Power to Shaft $P_2$, (W) | Mains Power $P_1$, (W) | Pump Eff., $\eta_p$ | Shaft Eff., $\eta_s$ | Overall Eff., $\eta_o$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 4 | 2.2 | 24 | CRE3-7 | 550 | 144 | 251 | 377 | 57.3% | 66.6% | 38.2% |
| 8 | 2.7 | 24 | CRE3-7 | 550 | 177 | 307 | 448 | 57.5% | 68.5% | 39.4% |
| 16 | 3.4 | 24 | CRE5-8 | 1100 | 222 | 380 | 564 | 58.5% | 67.4% | 39.4% |
| 24 | 4.2 | 24 | CRE5-8 | 1100 | 275 | 456 | 661 | 60.2% | 69.0% | 41.6% |
| 48 | 7.3 | 24 | CRE10-3 | 1100 | 477 | 728 | 996 | 65.6% | 73.1% | 47.9% |
| 96 | 12.8 | 24 | CRE15-2 | 2200 | 837 | 1,240 | 1,510 | 67.5% | 82.1% | 55.4% |
| 192 | 22.0 | 24 | CRE20-3 | 4000 | 1,439 | 2,120 | 2,580 | 67.9% | 82.2% | 55.8% |
| 288 | 31.5 | 24 | CRE32-2 | 4000 | 2,060 | 2,840 | 3,410 | 72.5% | 83.3% | 60.4% |
| 384 | 21.0 | 24 | CRE20-3 | 4000 | 1,373 | 2,000 | 2,450 | 68.7% | 81.6% | 56.1% |
|  | 21.0 | 24 | CRE20-3 | 4000 | 1,373 | 2,000 | 2,450 | 68.7% | 81.6% | 56.1% |
| 576 | 31.5 | 24 | CRE32-2 | 4000 | 2,060 | 2,840 | 3,410 | 72.5% | 83.3% | 60.4% |
|  | 31.5 | 24 | CRE32-2 | 4000 | 2,060 | 2,840 | 3,410 | 72.5% | 83.3% | 60.4% |

The transfer pump used for housing layouts from 4 to 96 homes was of the same model, a different model was chosen for the 192 and 288 homes layouts and a separate one for the 576 house layout. Flow rates for both pumps were assigned a constant of 2.0 $m^3$/hr, 4.5 $m^3$/hr and 8.0 $m^3$/hr respectively and required a minimum pumping head of 8 m. Table shows the power requirements and efficiencies of the chosen pumps.

**Table 11: Pump outputs for selected transfer pumps.**

| Peak Flow Rate ($m^3$/hr) | Generated Pump Head, $H_P$, (m) | Pump Model | Pump Size, (W) | Hydraulic Power $P_H$, (W) | Power to Shaft $P_2$, (W) | Mains Power $P_1$, (W) | Pump Eff., $\eta_p$ | Shaft Eff., $\eta_s$ | Overall Eff., $\eta_o$ |
|---|---|---|---|---|---|---|---|---|---|
| 2.0 | 11.7 | CRI3-2 | 370 | 64 | 131 | 193 | 48.7% | 67.9% | 33.0% |
| 4.5 | 10.0 | CRI5-2 | 370 | 123 | 246 | 321 | 49.8% | 76.6% | 38.2% |
| 8.0 | 8.4 | CRI10-1 | 370 | 183 | 302 | 388 | 60.6% | 77.8% | 47.2% |

Sump pumps were not required for housing layouts with less than 48 homes. For housing layouts with 48 and 96 homes, the same pump model was selected, whilst a bigger pump was chosen for the larger layouts (Table 12). In the case of the 576 house model, two pumps were required to empty the sump well.

**Table 12: Pump outputs for selected sump pumps.**

| Peak Flow Rate ($m^3$/hr) | Generated Pump Head, $H_P$, (m) | Pump Model | Pump Size, (W) | Hydraulic Power $P_H$, (W) | Power to Shaft P2, (W) | Mains Power P1, (W) | Pump Eff., $\eta_p$ | Shaft Eff., $\eta_s$ | Overall Eff., $\eta_o$ |
|---|---|---|---|---|---|---|---|---|---|
| 8.0 | 8.4 | CRI10 - 1 | 370 | 183 | 302 | 388 | 60.6% | 77.8% | 47.2% |
| 20.0 | 10.1 | CRI20 - 1 | 1100 | 550 | 883 | 1050 | 62.3% | 84.1% | 52.4% |

### 3.2.11. Power Consumption

Operating costs for the rainwater harvesting system consist mainly of the total power used within the system. To estimate operational costs and its overall effect on the life cycle analysis, it is vital that energy consumed within the harvesting system is estimated. Energy consuming equipment within the communal rainwater harvesting system includes the various pumps and the UV system, with the former being the main consumer. A simple method of estimating the daily energy consumption of a pump is through Equation 13:

$$E_d = P_1 \times O_d \tag{13}$$

*Where:*

$E_d$ = daily energy consumption (Wh), $P_1$ = mains power consumption (W), $O_d$ = daily operating duration (hours).

The previous section tabulated the energy consumed (mains power) by each of the different pumps used in the system, which is the pump (operating phase) energy consumption. However, there has been no distinction between the energy consumed during the pump start-up and operating phase, as the former uses more energy than the latter.

A study by Ward *et al.* (2012) from Exeter University attempted to estimate the total power used by a 1.1 kW rainwater harvesting pump, taking into consideration the energy used in the start-up phase in addition to the operating phase. They have estimated that Equation 13 underestimates the total power used by up to 60%. By applying this factor, the modified version of estimating the power consumption can be shown in Equation 14 by:

$$E_{d.f} = \frac{E_d}{0.6} \, or \, 1.667.E_d \tag{14}$$

*Where:*

$E_{d.f}$ = factored daily energy consumption (Wh).

Equation 14 will be applied to determine the annual total power consumed by the pumps.

# 4. COST DATA

## 4.1. Individual Rainwater Tanks

Costs for rainwater tanks are made up of two main parts; the capital costs of purchasing and installing a rainwater tank plus its accessories, and the ongoing costs, which includes the maintenance and operation costs. Cost data for these items were sought from a number of sources, mainly through verbal communication and e-mail exchanges with suppliers and plumbers as well as through their websites. The average price for each component was used in the life cycle assessment of individual rainwater tanks, with individual supplier prices presented in Appendix B.

### 4.1.1. Rainwater Tanks

QDC MP 4.2 states that rainwater tanks for Class 1 individual dwelling has to be at least 5 kL for a Class 1 detached building. Costs data for rainwater tanks of this size were obtained from a number of rainwater tank dealers in and around the SEQ region, with average cost being used for the life cycle assessment.

The price of the tanks comprised a 5 kL round plastic polyethylene tank, delivery charges and basic accessories. Accessories included: screened downpipe rain-heads (three in total; one for each connected downpipe); a first flush system; a pump cover; fittings and flexible coupling for connecting the pump to the tank; screens for leaf and debris; mosquito screens or flap valves; and a vermin trap. Gutters are assumed to be already in place within the property diverting water into the stormwater system and so are not included in the prices.

The average cost of purchasing a standard above ground, round polyethylene 5 kL rainwater tank with the required accessories came up to $1,529. Prices are generally higher for slimline, rectangular, steel or underground tanks and have not been taken into consideration in this study.

### 4.1.2. Pumps

As each rainwater tank dealer had their own stocks of pumps, pumps which were priced were not all of similar brands and makes, although care has been given to ensure that they had similar specifications. Pump power ranged from 0.55 kW to 0.7 kW, and all were equipped with an auto-switching device and a backflow prevention device. Pumps fitted with auto mains switch-over cost on average $842.

### 4.1.3. Plumbing

Plumbing costs were obtained from tank dealers and plumbers offering the service and included connecting the rainwater tank to two toilet cisterns, washing machine and the garden tap. Average plumbing costs for the region came up to $888.

### 4.1.4. Tank and Pump Installation

As well as the tank, pump and plumbing costs, other capital costs included installation costs of rainwater tanks, cost of laying concrete to provide a solid foundation base on which to place the rainwater tank and electric installation to connect up the pump (Table 13).

**Table 13: Tank and Pump Installation Costs.**

| Component | Average Cost |
|---|---|
| Concrete Laying | $700 |
| Tank Installation | $367 |
| Electrical Installation (pumps) | $200 |

### 4.1.5. Maintenance Costs

Maintenance of the tanks, which include cleaning of gutters, first flush devices and rain-heads, were assumed to be carried out regularly by the home owner. For the purpose of the study, annual maintenance costs (Table 14) included the hiring of a professional service to ensure thorough and proper maintenance activities were carried out, including checks on pumps, filters, rainwater tap signage and water quality in addition to the householders maintenance activities. Desludging activities was to be carried out every three years on the recommendations of rainwater tank maintenance providers.

**Table 14: Maintenance costs for an individual rainwater tank system.**

| Component | Frequency | Average Cost |
|---|---|---|
| Sediment check/cleaning | Three Years | $162 |
| Gutter maintenance, etc | Annually | $80 |
| Check signage, pumps, filters, water quality | Annually | $50 |

### 4.1.6. Electricity

Annual energy consumption estimates were obtained from monitored energy data for 20 homes located in SEQ (Umapathi *et al.*, 2012). Pumps varied in size from 0.45 kW to 0.8 kW, with annual power consumptions ranging from 10 kW to 104 kW, depending on pump sizes and household occupancy. Average annual consumption for a pump with auto mains switchover was estimated to be 64 kW, which was used in the estimation of yearly electricity prices.

Electricity tariffs for residential properties were obtained from energy providers in Queensland. Standard electricity charges were used, i.e., no variation in prices for off-peak and peak hours, to ensure consistency in the pricings and was found to be $0.2276/kWh. Annual electricity service charges are not included in the cost estimates as it is taken to be already a part of the householder's normal utility bills.

## 4.2. Communal Rainwater Tanks

Communal rainwater systems span over a larger scale than individual tanks and involve more components in its set up, thus requiring more cost data to be gathered. Along with costs for rainwater tanks and pumps, additional data included the cost of laying pipes, obtaining and installing the water treatment units as well as further operation and maintenance costs for such equipment. Cost of the land for setting up the treatment units has not been considered in this analysis as this will be part of the development cost.

### 4.2.1. Unit Cost of Pipes

Prices per unit length of pipes were obtained from Gold Coast Water's (2008) *Unit Rate Reviews*, but were based on June 2008 rates and required updating to reflect on current conditions (December 2011).

The Australian Bureau of Statistics (ABS) provides measures of inflation through a list of indexes, e.g., Consumer Price Index, Producers Price Index and Labour Price Index. For the purpose of scaling prices to current values, the Producer Price Index (PPI) was applied, with Index Number 3011, "*House construction Queensland*", under Table 15, *"Selected output of division E construction, subdivision and class index numbers"*, (Australian Bureau of Statistics, 2012) chosen as appropriate for use for this purpose. The formula used to update prices is given in Equation 15:

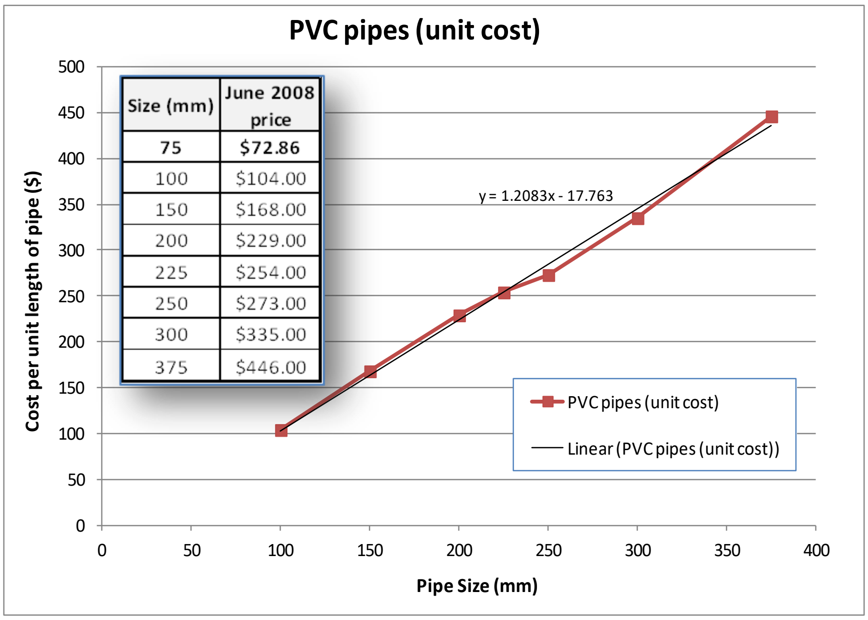
$$\% \; change \; in \; price = \frac{Current \; Year \; Index - Base \; Year \; Index}{Base \; Year \; Index} \times 100\% \tag{15}$$

Index values were taken from Table 15 of the PPI Index, Index Number 3011, *"House construction Queensland"* (ABS, 2012) and using the above equation, the percentage change in prices, compared to June 2008, was obtained to adjust prices to December 2011 values (Table 15).

**Table 15: Producer Price Index value adjusted to December 2011 prices.**

| | |
|---|---|
| Dec 2011 Index = | 171.4 |
| June 2008 Index = | 161.7 |
| Percentage change in prices = | 6% |

The price for laying a 75 mm polyvinyl chloride (PVC) pipe was not available in the report and was obtained from a regression analysis, which showed a direct linear relation between unit cost and pipe size. This relationship and the unit prices for laying potable water mains for polyvinyl chloride (PVC) pipes are shown in Figure 8.



| Size (mm) | June 2008 price |
|---|---|
| 75 | $72.86 |
| 100 | $104.00 |
| 150 | $168.00 |
| 200 | $229.00 |
| 225 | $254.00 |
| 250 | $273.00 |
| 300 | $335.00 |
| 375 | $446.00 |

**Figure 8: Relationship between cost and pipe size.**

### Depth of Pipes

Costs of pipes vary with depth, with per unit prices increasing as pipes are buried deeper in the ground. Prices shown in the previous figure are valid for depths of up to 1.5 metres and Table 16 shows a list of factors which need to be applied to various pipe diameters at a range of depths.

**Table 16: Cost factors for varying depths of pipes.**

| Depths | Pipe diameter <300mm | Pipe diameters >=300mm |
|---|---|---|
| Up to 1.50m | 1.00 | 1.00 |
| 1.5m – <3.0m | 1.19 | 1.25 |
| 3.0m – <4.5m | 1.34 | 1.40 |
| > 4.5m | 1.47 | 1.54 |

**Short Length Pipes**

Gold Coast Water's (2008) report also notes that shorter pipe lengths (< 200 m) incur higher costs than long runs due to lower use of machinery and manpower. As a result, allowances have to be provided for pipes of lengths shorter than 200 m in their cost estimates and are shown in Table 17.

**Table 17: Cost factors for short lengths pipes.**

| Pipe Length | Factor |
|---|---|
| 0 – 50m | 2.0 |
| 51 – 100m | 1.7 |
| 101 – 200m | 1.5 |

Table 18 shows unit prices for pipes which have been applied with short length factors.

**Table 18: Cost of pipes per unit length applying short length factors.**

| Size (mm) | Dec 2011 prices | | | |
|---|---|---|---|---|
| | PVC<br>< 50m | PVC<br>(51m - 100m) | PVC<br>(101m -200m) | PVC<br>> 200m |
| 75 | $154.46 | $131.29 | $115.85 | $77.23 |
| 100 | $220.48 | $187.41 | $165.36 | $110.24 |
| 150 | $356.16 | $302.73 | $267.12 | $178.08 |
| 200 | $485.47 | $412.65 | $364.11 | $242.74 |
| 225 | $538.47 | $457.70 | $403.86 | $269.24 |
| 250 | $578.75 | $491.94 | $434.06 | $289.38 |
| 300 | $710.19 | $603.66 | $532.64 | $355.10 |
| 375 | $945.51 | $803.68 | $709.13 | $472.75 |
| 450 | $1,742.62 | $1,481.23 | $1,306.96 | $871.31 |

### 4.2.2. Pump Costs

Cost data for chosen recirculation pumps were sought directly from Grundfos pump dealers, and were mainly available as package systems, whereby the pump comes as a complete package unit including start-up equipment such as bases, manifolds, control panel, valves, etc. For the purpose of this study, a package deal consisting of a duty and a standby pump was chosen for each system, and a quotation also requested for the individual pumps to determine replacement costs.

Individual pump costs were available only for the single speed range of Grundfos pumps chosen for the transfer pump. Prices for start-up equipment, including lockable main isolator, individual pump circuit breakers, run/fault light stainless steel folded base, manifolds and isolation valves, were estimated from the packaged cost provided for the variable speed pumps. The setup for the transfer pump would be of a similar nature, with a duty and standby pump in place. Cost data for both sets of pumps, as well as estimated separate costs for the equipment, are shown in Table 19.

**Table 19: Cost of selected pumps.**

| Pump Usage | Pump Models | Pump Power, (W) | Single Pump Cost | Additional Pump Equipment |
|---|---|---|---|---|
| Recirculation Pump | CRE3-7 | 550 | $3,380 | $3,685 |
| | CRE5-8 | 1100 | $3,655 | $4,305 |
| | CRE10-3 | 1100 | $3,780 | $5,945 |
| | CRE15-2 | 2200 | $4,225 | $5,585 |
| | CRE20-3 | 4000 | $6,550 | $5,380 |
| | CRE32-2 | 4000 | $8,420 | $5,380 |
| Transfer Pump | CRI3-2 | 370 | $1,070 | $3,685 |
| | CRI5-2 | 370 | $1,185 | $3,685 |
| Sump Pump | CRI10-1 | 370 | $1,575 | $3,685 |
| | CRI20-1 | 1100 | $2,055 | $3,685 |

### 4.2.3. Pump Installation

In addition to pump installation costs, pump commissioning is also required to ensure that they are put into operation quickly and effectively and are as energy efficient as possible. Table 20 displays the average costs for the installation and commissioning of the pumps, obtained from pump installers.

**Table 20: Installation and commission costs of pumps.**

| Item | Average Cost |
|---|---|
| Installation of Pump | $1,650 |
| Commissioning of Pumps | $660 |

### 4.2.4. Underground Concrete Tanks

Underground concrete tanks are used for the sump wells and the rainwater collection tanks, both of which have been sized for the individual housing layouts using UVQ. Only housing layouts of 48 homes or more required sump wells where pipe depth was more than 1.5 metres. Costs of concrete per cubic meter for constructing and installing an underground tank came to $717 based on average concrete tank prices obtained from suppliers. Concrete volumes were calculated for each tank size and costs tabulated from this. Costs of concrete tanks are shown in Table 21 and include installation. The excavation costs are accounted separately.

### 4.2.5. Inspection Shaft

Inspection shafts are required to allow entry into the sump wells for maintenance purposes. For the purpose of this study, a shaft diameter of 1.5 m was chosen for all housing layouts, with depths calculated from the roof of the sump well to ground level. Unit rates of inspection shafts installation were obtained from Gold Coast Water's (2008) rate reviews on manholes and are shown in Table 21, with rates brought to current prices.

**Table 21:** **Underground concrete tank prices.**

| **Cost of Sump Pumps** | | | | | | | |
|---|---|---|---|---|---|---|---|
| No. of households | Storage Volume Required (kL) | Diameter (m) | Height (m) | Volume of tank provided (kL) | Volume of concrete required ($m^3$) | Cost per tank | Tanks required |
| 48 | 60 | 5.0 | 3.1 | 60.9 | 16.9 | $12,200 | 1 |
| 96 | 80 | 5.8 | 3.1 | 81.9 | 21.1 | $15,100 | 1 |
| 192 | 100 | 6.5 | 3.1 | 102.9 | 25.0 | $18,000 | 1 |
| 288 | 120 | 7.0 | 3.1 | 119.3 | 28.0 | $20,100 | 1 |
| 384 | 140 | 7.7 | 3.1 | 144.4 | 32.4 | $23,300 | 1 |
| 576 | 180 | 8.7 | 3.1 | 184.3 | 39.3 | $28,300 | 1 |
| **Costs of Storage Tanks** | | | | | | | |
| No. of households | Storage Volume Required (kL) | Diameter (m) | Height (m) | Volume of tank provided (kL) | Volume of concrete required ($m^3$) | Cost per tank | Tanks required |
| 4 | 40 | 4.5 | 2.6 | 41.4 | 13.24 | $9,500 | 1 |
| 8 | 80 | 5.8 | 3.1 | 81.9 | 21.05 | $15,100 | 1 |
| 16 | 160 | 8.2 | 3.1 | 163.7 | 38.23 | $27,400 | 1 |
| 24 | 240 | 10.0 | 3.1 | 243.5 | 49.26 | $35,300 | 1 |
| 48 | 480 | 13.3 | 3.5 | 482.6 | 81.74 | $58,700 | 1 |
| 96 | 960 | 13.3 | 3.5 | 482.6 | 81.74 | $58,700 | 2 |
| 192 | 1920 | 13.3 | 3.5 | 482.6 | 81.74 | $58,700 | 4 |
| 288 | 2880 | 13.3 | 3.5 | 482.6 | 81.74 | $58,700 | 6 |
| 384 | 3840 | 14.3 | 4 | 642.4 | 97.00 | $69,600 | 6 |
| 576 | 5760 | 14.3 | 4.5 | 722.7 | 101.09 | $72,500 | 8 |

**Table 22:** **Manhole costs for different depths.**

| **Inspection Shaft Depth** | **2008 Rates** | **2011 Rates (indexed)** |
|---|---|---|
| 1.0m – 1.9m | $5,235 | $5,550 |
| 2.0m – 3.0m | $7,556 | $8,010 |
| 3.0m – 4.0m | $10,339 | $10,960 |
| 4.0m – 5.0m | $13,226 | $14,020 |

### 4.2.6. Excavation Costs

Excavation costs were estimated using Rawlinson Group's (2011) *"Construction guide for housing, small commercial and industrial buildings"*, and was tabulated on a per cubic meter basis to obtain prices up till December 2010. The costs were then scaled using the indexed method outlined in Section 4.2.1 to obtain current prices (Table 23). The resulting excavation cost was for depths up to one metre; depths factors have been applied for deeper excavations using the pipe depths factors from Table 16.

### 4.2.7. Excavation Volumes

In the case of storage tanks and sump wells, excavation widths were provided with an allowance of a meter on the diameter of the tanks. Excavation depths were taken as the depth of the tank with a one meter allowance, plus the depth of the inflow of the collection pipe into the tank; either the storage or sump wells. Excavation volumes were then calculated based on these estimations for each of the housing layout and are shown in Table 24.

**Table 23: Excavation costs.**

| Item | | Description |
|---|---|---|
| *Dec 2010 prices* | | |
| Excavate trench = | $65.70/m$^3$ | Excavate Trench in clay |
| Labour charge = | $20.00/m$^3$ | 0.4 operator hours (using backhoe) per cubic metre at operator rate of $50 per hour |
| Additional carting = | $2.20/m$^3$ | Additional tip charges for distance exceeding 10km; assumed to be a further 5 km |
| Total = | $87.90/m$^3$ | |
| *Dec 2011 prices* | | |
| Dec 2011 index = | 171.4 | *Index Numbers obtained from ABS (2012), PPI, Table 15, Index No. 3011, House Construction Queensland* |
| Dec 2010 index = | 172.0 | |
| Percentage Change = | -0.35% | |
| Dec 2011 price = | $87.59 | |
| | | |
| GST add cost = | 10% | |
| **Price inclusive of GST** | **$96.35/m$^3$** | |

**Table 24: Volume of excavations required for tanks.**

| No. of Households | Sump Pump Volume of Excavation (m$^3$) | Storage Tank Volume of Excavation (m$^3$) |
|---|---|---|
| 4 | N/A | 103 |
| 8 | N/A | 180 |
| 16 | N/A | 351 |
| 24 | N/A | 509 |
| 48 | 160 | 718 |
| 96 | 219 | 1,435 |
| 192 | 288 | 2,871 |
| 288 | 356 | 4,306 |
| 384 | 421 | 5,516 |
| 576 | 635 | 8,090 |

### 4.2.8. Plant Room

Capital costs involved in a communal water treatment plant would include the construction of a small building to house the water treatment units and pumps to protect them from the weather. A similar layout is present in CDM, although the building houses both the treatment units for recycled water and rainwater. A construction cost of the structure was provided by the manager in CDM and it is assumed that half of this value would be sufficient in constructing a similar building to house rainwater treatment units (Table 25).

**Table 25: Construction cost of plant room (December 2006).**

| Item | Approx. Cost |
|---|---|
| Cost of Constructing CDM building; housing both recycled water and rainwater treatment units | $72,000 |
| Estimated cost of constructing housing for rainwater treatment units (half of CDM costs) | $36,000 |

The obtained cost is relevant for all housing layouts as the equipment would more or less take up the same space with no anticipated increase in space required to house them. However, costs provided were for December 2006 and required scaling to update them to current levels (Table 26).

**Table 26: Updated costs of plant room (December 2011).**

| ***Dec 2011 prices*** | | **Description** |
|---|---|---|
| Dec 2011 index = | 164.8 | *Index Numbers obtained from ABS (2012), PPI, Table 15, Index No. 30, Building Construction Queensland* |
| Dec 2006 index = | 152.0 | |
| Percentage Change = | 8.42% | |
| Dec 2011 price = | **$39,032** | |

### 4.2.9. Water Treatment

The collected rainwater will be treated to potable standards using a filtration and ultraviolet (UV) system and supplemented by a chlorination facility. Information on liquid and gaseous chlorination units were difficult to obtain, whilst cost data for a tablet system was available and hence, chosen for this analysis.

A tablet system is made up of a tablet dispenser unit and easy to use chlorine tablets and requires no power to run. Each tablet is able to purify 26 KL of water a day with multiple units required for higher flows. The chosen filter and UV disinfectant combo is able to treat water up to 76 L/min (4.56 $m^3$/hr). Cost data for these units are provided in Table 27.

**Table 27: Costs for water treatment units.**

| Item | Approx. cost |
|---|---|
| Sand and carbon filter with UV sterilisation | $2,500 |
| Chlorination system | $1,500 |

### 4.2.10. Holding Tank

Holding tanks for the storage of the treated rainwater have been sized to provide residents with approximately three days of supply without the need for top ups. Prices for the different sized rainwater tanks are shown in Table 28.

### 4.2.11. Maintenance Costs

Gutter maintenance costs for the communal system are taken to be half of individual rainwater tanks on a per household basis as maintenance for tanks are handled separately in a communal system. Maintenance costs for tanks differ for each household layout due to the different size of the collection tanks, sump wells and holding tanks used (Table 29).

Additional ongoing costs are also incurred for changing the filters and UV lamps as well as supply of chlorine tablets to the chlorine dispenser. Cost of maintenance for the centrifugal pumps utilised in the communal system is also higher than pumps used in individual rainwater tanks.

**Table 28:** **Cost of holding tanks.**

| No. of Homes | Calculated Size of Holding Tank (kL) | Available Tank Size (kL) | Cost with Installation |
|---|---|---|---|
| 4 | 2.7 | 2.5 – 3.0 | $1,177 |
| 8 | 5.2 | 5.0 | $1,443 |
| 16 | 10.4 | 10.0 | $1,940 |
| 24 | 15.6 | 13.5 – 18.0 | $2,503 |
| 48 | 30.9 | 30.0 – 34.0 | $4,046 |
| 96 | 61.7 | 60* | $12,200 |
| 192 | 123.5 | 120* | $20,100 |
| 288 | 184.9 | 180* | $28,300 |
| 384 | 246.7 | 240* | $35,300 |
| 576 | 369.9 | 370* | $46,800 |

* indicates that concrete tanks were used for these housing layouts.

**Table 29:** **Maintenance costs for a communal rainwater system.**

| Component | Frequency | Average Cost |
|---|---|---|
| Sediment Check/ Cleaning for tank sizes:<br>40kL<br>80kL<br>160kL<br>240kL<br>480kL<br>640kL<br>720kL | Three Years | <br>$370<br>$600<br>$2,000<br>$2,500<br>$4,800<br>$6,000<br>$6,500 |
| Gutter Maintenance, etc | Annually | $40 |
| Maintenance of Pumps (per set) | Annually | $350 |
| Maintenance of filters and UV lamps | Annually | $300 |
| Maintenance of chlorine dispenser | Annually | $200 |

## 4.2.12. Electricity

Grundfos WebCaps application[1] provides mains power consumption values for each of the different pump sets. Estimated annual power usage will be calculated using the factored energy consumption equation outlined in Section 3.2.11. For recirculation pumps (Table 30), yearly energy usage will be based on an assumption that these pumps run an average of 10 hours per day, which was obtained from an analysis on the running times of recirculation pumps in Capo di Monte.

Transfer pumps, which transfer water from the storage to the holding tanks, are assumed to operate and provide a top-up to the holding tank, based on the total daily water consumption of householders, at fixed flow rates of 2.0 $m^3$/hr for up to 96 households and 4.5 $m^3$/hr beyond this size. Estimated running times of the transfer pumps and annual power consumptions are as shown in the Table 31.

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[1] Grundfos WebCaps application located at http://net.grundfos.com/Appl/WebCAPS/custom?&userid=GPA&lang=ENU

**Table 30: Estimated annual power usage of recirculation pumps.**

| No. of Homes | Recirculation Pump Mains Power Usage, (W) | Daily Power Consumption, $E_d$ (kWh) | Daily Power Consumption, $E_{d.f}$ (kWh) | Estimated Yearly Power Consumption (kWh) |
|---|---|---|---|---|
| 4 | 377 | 3.8 | 6.3 | 2,295 |
| 8 | 448 | 4.5 | 7.5 | 2,728 |
| 16 | 564 | 5.6 | 9.4 | 3,434 |
| 24 | 661 | 6.6 | 11.0 | 4,024 |
| 48 | 996 | 10.0 | 16.6 | 6,064 |
| 96 | 1,510 | 15.1 | 25.2 | 9,193 |
| 192 | 2,580 | 25.8 | 43.0 | 15,706 |
| 288 | 3,410 | 34.1 | 56.8 | 20,759 |
| 384 | 4,900 | 49.0 | 81.7 | 29,829 |
| 576 | 6,820 | 68.2 | 113.7 | 41,517 |

**Table 31: Estimated annual power usage of transfer pumps.**

| No. of Homes | Daily Water Demand (kL) | Transfer Pump Running Time (hrs) | Daily Power Consumption, $E_d$ (kWh) | Daily Power Consumption, $E_{d.f}$ (kWh) | Estimated Yearly Power Consumption (kWh) |
|---|---|---|---|---|---|
| 4 | 0.9 | 0.5 | 0.10 | 0.16 | 59 |
| 8 | 1.7 | 0.9 | 0.17 | 0.29 | 106 |
| 16 | 3.5 | 1.8 | 0.35 | 0.58 | 212 |
| 24 | 5.2 | 2.6 | 0.50 | 0.84 | 306 |
| 48 | 10.3 | 5.2 | 1.00 | 1.67 | 611 |
| 96 | 20.6 | 10.3 | 1.99 | 3.31 | 1,211 |
| 192 | 41.2 | 9.2 | 2.95 | 4.92 | 1,798 |
| 288 | 61.6 | 13.7 | 4.40 | 7.33 | 2,678 |
| 384 | 82.2 | 18.3 | 5.87 | 9.79 | 3,576 |
| 576 | 123.3 | 15.5 | 6.01 | 10.02 | 3,662 |

As sump pumps run only on days when there is rainfall, it is important to conduct some estimation on the number of rainfall days per year falling over the area to estimate the yearly power consumption of the sump pumps. Using the climatic data used in the UVQ modelling, the number of days with rainfall was estimated over the analysed years (1991 – 2010), resulting in an average of 133 days of rainfall per year. Table 32 shows the approximate annual power usage of the sump pumps.

The operation of the UV disinfection is assumed to be in relation to the running of the transfer pump; i.e. UV system only operates when the transfer pump is running. Power consumption is relatively low at about 80W for such a system. The estimated annual power usages are listed in Table 33.

**Table 32: Estimated annual power usage of sump pumps.**

| No. of Homes | Daily Water Pumped during rainy days (kL) | Sump Pump Running Time (hrs) | Daily Power Consumption during rainy days, $E_d$ (kWh) | Daily Power Consumption during rainy days, $E_{d.f}$ (kWh) | Estimated Yearly Power Consumption (kWh) |
|---|---|---|---|---|---|
| 48 | 29.3 | 3.7 | 1.44 | 2.39 | 319 |
| 96 | 73.6 | 9.3 | 3.61 | 6.01 | 800 |
| 192 | 177.7 | 8.9 | 9.35 | 15.58 | 2,072 |
| 288 | 284.5 | 14.3 | 15.02 | 25.03 | 3,329 |
| 384 | 394.5 | 18.0 | 19.62 | 32.70 | 4,350 |
| 576 | 606.6 | 13.8 | 30.08 | 50.14 | 6,669 |

**Table 33:** **Estimated annual power usage of UV disinfection system.**

| No. of Homes | UV System Running Time (hrs) | Daily Power Consumption (kWh) | Estimated Yearly Power Consumption (kWh) |
|---|---|---|---|
| 4 | 0.5 | 0.040 | 15 |
| 8 | 0.9 | 0.072 | 26 |
| 16 | 1.8 | 0.144 | 53 |
| 24 | 2.6 | 0.208 | 76 |
| 48 | 5.2 | 0.416 | 152 |
| 96 | 10.3 | 0.824 | 301 |
| 192 | 9.2 | 0.736 | 269 |
| 288 | 13.7 | 1.096 | 400 |
| 384 | 18.3 | 1.464 | 534 |
| 576 | 15.5 | 2.480 | 905 |

# 5. ECONOMIC ASSESSMENT

## 5.1. Basis of the Cost Assessment

The net present value (NPV) analysis has been applied to estimate the life cycle costs of an individual and communal rainwater system for an analysis period of 50 years. Prices of components discussed in the previous chapter have been used, along with a discount rate of 7%, to carry out the economic assessment. The final NPV has been broken down further to obtain an idea on the cost contribution for each of the individual components; i.e., capital, operational, maintenance and replacement for both these systems. As the systems would be used for different purposes, non-potable for individual and potable for communal, NPVs would be compared on a per household basis. Levelised costs from the final NPV is not within the scope of the study.

## 5.2. Individual Rainwater Tanks

### 5.2.1. Cost Contribution of Various Components

An NPV life cycle assessment was carried out on a per household basis for individual rainwater tanks. Costs of the various components used in the analysis have been obtained from their providers and are shown in Appendix B along with the resulting NPV over the analysis period. The NPV of installing an individual rainwater tank for its usage as a non-potable water source has been estimated to be approximately $8,568 over the analysis period. The resulting NPVs and their breakdowns are shown in Table 34.

**Table 34: Cost breakdown for individual rainwater tanks over the 50-year analysis period.**

| Cost Components | NPV | NPV Contribution |
|---|---|---|
| Capital Costs | $4,526 | 52.8% |
| Maintenance Costs | $2,486 | 29.0% |
| Replacement Costs | $1,355 | 15.8% |
| Operation Costs | $201 | 2.3% |
| **Total Costs** | **$8,568** | |

Capital costs of setting up a rainwater tank represent approximately 53% of the overall NPV, signifying a major proportion of total costs. Although capital costs in the study carried out by Stewart (2011) study was much higher at 80% - 82%, this can be attributed to the shorter analysis period (25 years), and also the assumption taken that the home owner would carry out basic maintenance with minimal costs ($20/annum). This would bring down the total NPV, thus skewing the costs contribution to capital costs.

Maintenance costs make up 29% of total NPV, followed by replacement costs of pumps and tanks at the end of their life cycle at almost half this value (16%). Electricity costs for running a standard sized pump is shown to have the least contribution and comprises of only 2% of total life cycle costs, which represents a relatively low cost component on the overall scheme of things. These are also in agreement with Stewart (2011), although figures have not been stated. The study by Marsden Jacobs Associates (2007) indicated that operating and maintenance costs made up only 10% of overall costs. This is due to the study assuming a low operating and maintaining costs of $20/annum which is a much lower figure than that used for this project.

### 5.2.2. Sensitivity Analysis

Sensitivity analysis was carried out on the discount rates, using the Office of Best Practice Regulations (OBPR, 2007) recommendations of 3% and 11%, to examine their influence on the overall NPV and on the different cost components (Table 35). Only ongoing costs will be affected by the sensitivity analysis, as these costs need to be brought to present costs.

**Table 35: Results of sensitivity analysis as a comparison against the original obtained values.**

| Cost Components | 7% Rate | 3% Discount Rate | | 11% Discount Rate | |
|---|---|---|---|---|---|
| | NPV | NPV | %age of original NPV | NPV | %age of original NPV |
| Capital Costs | $4,526 | $4,526 | 100% | $4,526 | 100% |
| Maintenance Costs | $2,486 | $4,669 | 188% | $1,613 | 65% |
| Replacement Costs | $1,355 | $3,007 | 222% | $697 | 51% |
| Operation Costs | $201 | $375 | 187% | $132 | 66% |
| **Total NPV** | **$8,568** | **$12,576** | **147%** | **$6,968** | **81%** |

Sensitivity analysis results show that the final results are more sensitive to lower discount rates than higher rates, with more changes observed for all components. Prices are affected by more than 85% for all ongoing costs using the 3% rate, whilst the higher discount rate of 11% lowers price by a maximum of 49%. This is also shown in the total NPV, with the 3% discount rate increasing it by up to 47%, whilst the 11% rate decreases the value by 19%. Replacement costs are shown to be the most sensitive to discount rates, with prices more than double for the lower discount rate and half for the higher discount rate.

## 5.3. Communal Systems

### 5.3.1. Cost Contribution of Various Components

The NPV method was used to obtain the costing for a communal rainwater system on a per household basis with cost data obtained from various suppliers. Appendix C presents the spreadsheet used to obtain the NPV for the 192 households' layout. This example spreadsheet was applied for all households analysed in this report. Results followed a slightly different trend to individual rainwater tanks, with initial costs representing the largest proportion of total life cycle costs in communal systems, followed by maintenance, operational costs and replacement costs making up the least. Figure 9 shows the proportion of costs for the various cost components of a communal rainwater system.

Overall, the capital cost represents more than 70% of total costs and is significantly high in comparison to individual rainwater tanks. All other costs were lower than for an individual system, with maintenance costs representing 10% to 15% of overall life cycle costs, operation costs coming up to 3% to 5% and replacement costs at 1% to 10% for various cluster sizes of households.

### 5.3.2. Capital Costs

Capital costs in communal rainwater harvesting systems can be divided into two main categories, one for laying the pipes network and another for setting up the storage and treatment units including auxiliary systems (storage and holding tanks and pumps). These two categories are the dominating factors in influencing final net present values for the majority of housing layouts, with cost distribution not below 30% for both categories beyond the 16 households scale.

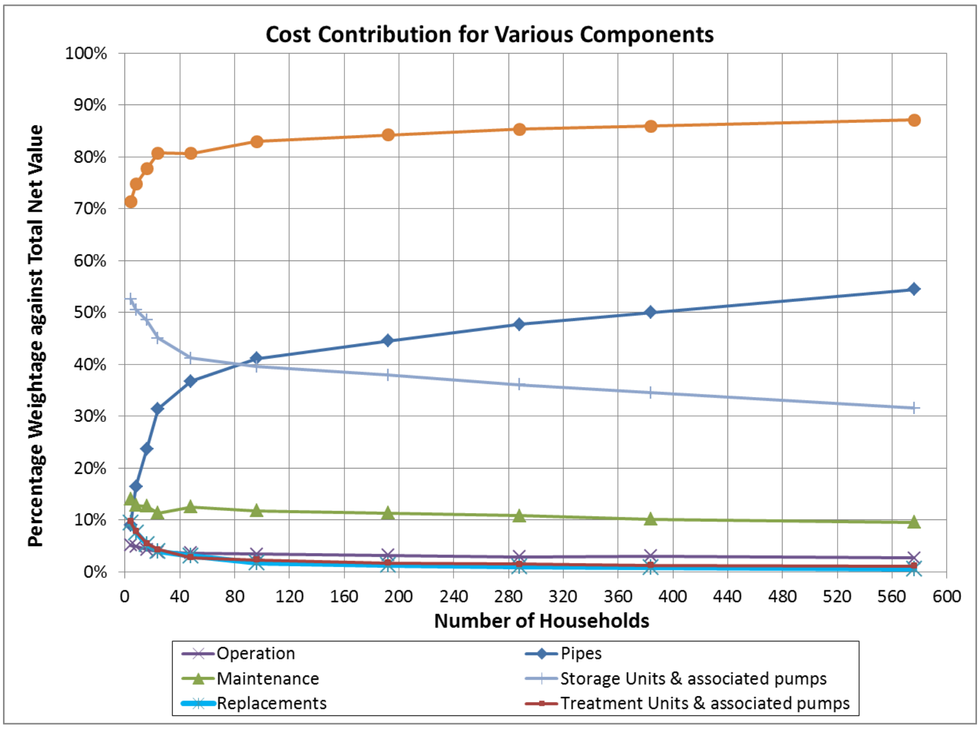


**Figure 9: Cost weightage for various components in communal rainwater systems over a 50-year period.**

From Figure 9, the cost weightage of pipes is observed to rise with increasing number of households, whilst on the other hand, the weightage for both storage and treatment systems decrease. This can be attributed to increases in pipe laying costs on a per household basis with increasing number of households, along with a decrease in the costs of storage and treatment units as shown in Figure 10.

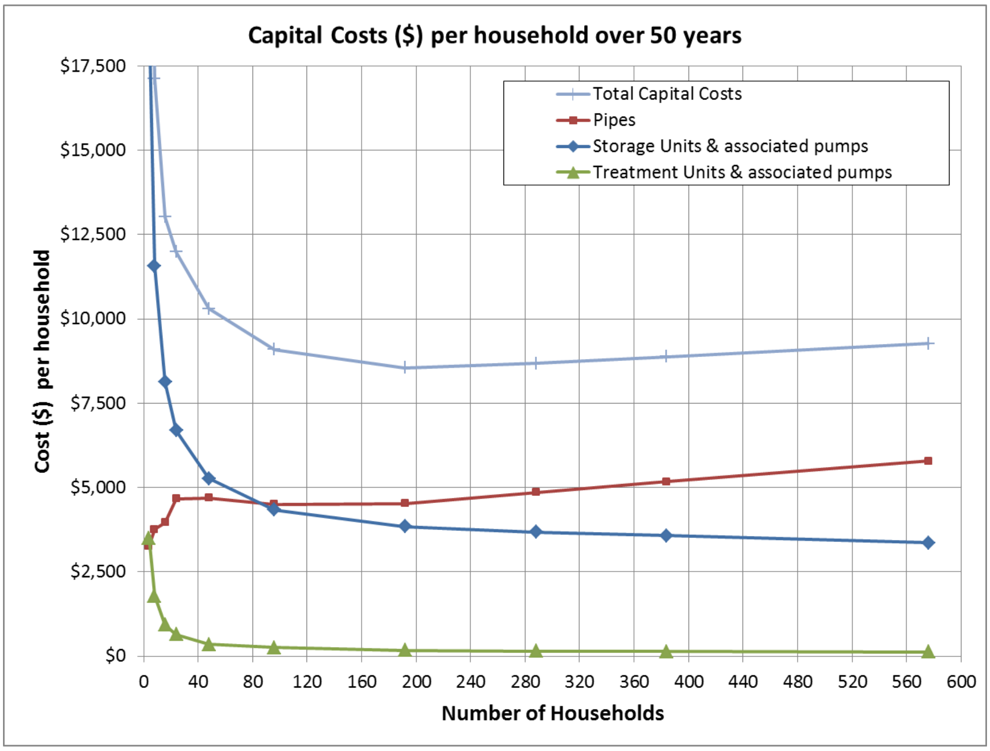


**Figure 10: Capital costs per household for pipes, storage units and treatment units for various housing layouts over a 50-year period.**

The increase in pipe laying costs is a result of a combination of a number of factors. Progressively longer lengths of pipes are required as the number of dwellings increases resulting in the increase of overall costs. As the length of rainwater collection pipes increases, they are buried deeper, making it more expensive to convey rainwater to tanks as a depth factor is triggered for any pipes laid deeper than 1.5 meters (see Section 4.2.1 for the different depth factors). Furthermore, larger diameter pipes are required for each increasing housing layout, as collection pipes receive more rainwater, resulting in increasing costs as bigger pipes are more expensive to lay. All these factors result in a steady rise in pipe laying cost for increasing households creating a dis-economy of scale as highlighted in studies by Clark (1997) and Booker (1999).

On the other hand, storage and treatment unit costs are observed to drop on a per household basis. Increasing the number of households results in larger collection tanks being required, along with more excavation for these tanks. Bigger holding tanks are also required. These components make up the bulk of the construction costs for the storage and treatment units and are the main drivers of these costs. Unlike pipe laying, where the per meter costs increases for bigger sized pipes, costs of tanks on a per kilolitre basis decreases for increasing sizes. Although bigger sized pumps are used for a larger number of dwellings, the increase in costs is minute relative to the overall cost of constructing the storage and treatment facilities. Other costs, such as the water treatment units (filter, UV and chlorination) and the housing for these units are more or less unchanged. These factors result in an overall drop in treatment units' costs compared to pipe laying costs.

Figure 10 also shows that for lower numbers of households, between 4 to 96 dwellings, the individual capital cost is influenced by the cost of the storage and treatment units with the cost curves following a similar trend. Beyond this household number, the rising pipe costs affect the individual capital costs indicating dis-economy of scale. This can be seen in Figure 10, with the capital costs following a similar trend to the pipe costs. This observation is in agreement with studies carried out by Booker (1999) and Clark (1997), both of which state that treatment costs are the dominant factor for lower scale of connections and the dis-economy of pipe networks affecting higher scales of connections.

### 5.3.3. Life Cycle Costing

The life cycle cost analysis was conducted for various housing layouts and plotted in Figure as cost per household against number of households. The costs curve in Figure shows an initial drop in the costs per household, falling sharply from the four homes to 96 homes layout, with the curve flattening off between 192 and 288 homes. Both Booker (1999) and Clark (1997) in their analyses described similar trend on the cost curve. Only small differences in costs occurred over the housing range where the curve flattens off. Due to the nature of the cost curve and a degree of uncertainty involved in the analysis, it is difficult to estimate exactly where the most economically viable layout will occur, although the minimum cost can be observed to lie in the 192 household layout and is found to be \$10,150. Beyond this size, the cost per household starts rising, with the rise clearer after 288 households. This is a result of pipe costs exerting more weightage on the total life cycle cost of the system; making up more than 44% of overall costs for both layouts. Storage and treatment costs, although declining, still contribute up to 39% of overall life cycle costs.

The household costs for each of the housing layouts are influenced by the capital costs and represent the bulk of total net present values, ranging from 71% to 87%. This can be supported by Figure 10, which shows the capital costs graph following a similar trend to the final costs curve; an initial sharp drop till 96 households before smoothening off to reach its lowest at 192 households and rising after this. As described in the previous section, this similar trend to the capital costs, when broken down further, indicates that up to and including 96 houses, the economics of scale of treatment unit costs has a great influence on the individual NPV whilst, beyond this number, pipes are the main cost drivers due to their dis-economy of scale.

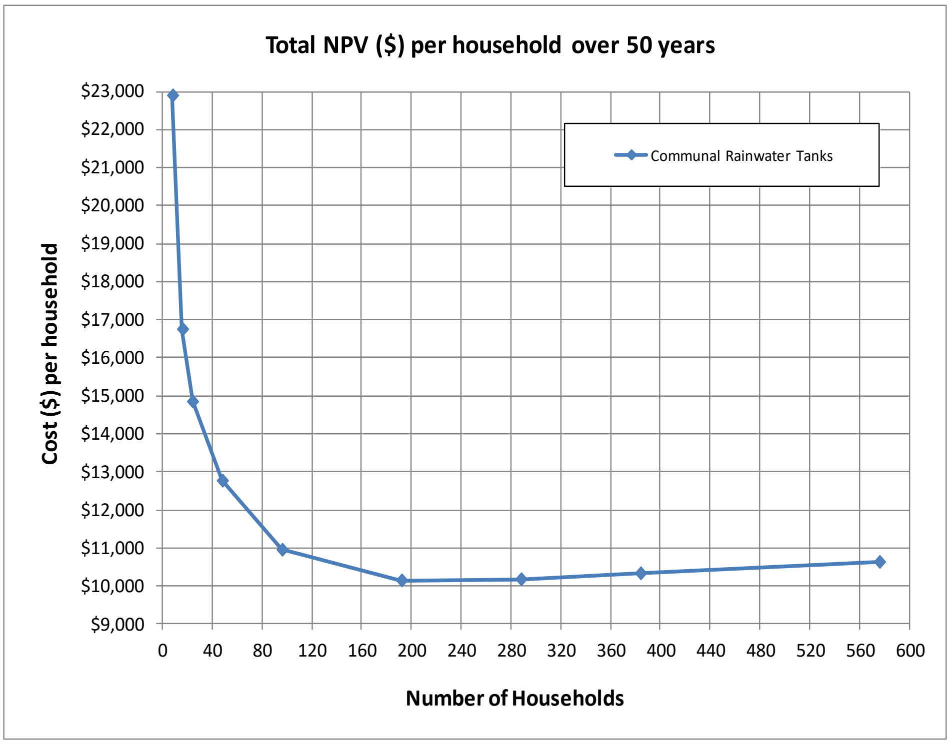


**Figure 11:** **Cost per household of communal systems for various sized housing layouts.**

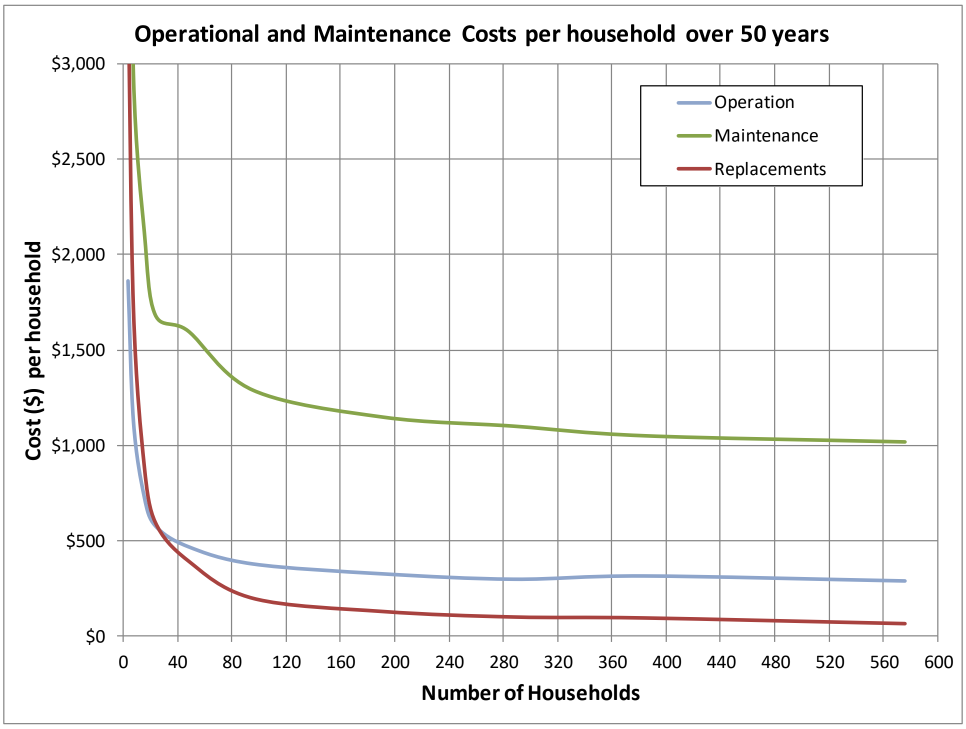


**Figure 12:** **Operational, maintenance and replacement costs for individual households.**

Ongoing costs are made up of the operational, maintenance and replacement costs for the communal rainwater systems. Each of these individual categories of ongoing costs initially has a decrease in their distribution before flattening and providing a more constant weightage. These costs are not as influential to the total NPV as they represent a small percentage, 29% to 13%, of the overall NPV.

Furthermore, the shape of the plot, as shown plotted in Figure for these three types of cost, does not follow that of the total NPV for individual households. After a rapid initial drop, these costs are observed to slowly decline beyond 96 households, indicating an economy of scale present. This also provides further validity to the influence pipe costs have over the overall NPV costs for each dwelling beyond this housing layout, as pipe prices and their laying charges drive up costs to create a dis-economy of scale.

### 5.3.4. Sensitivity Analysis

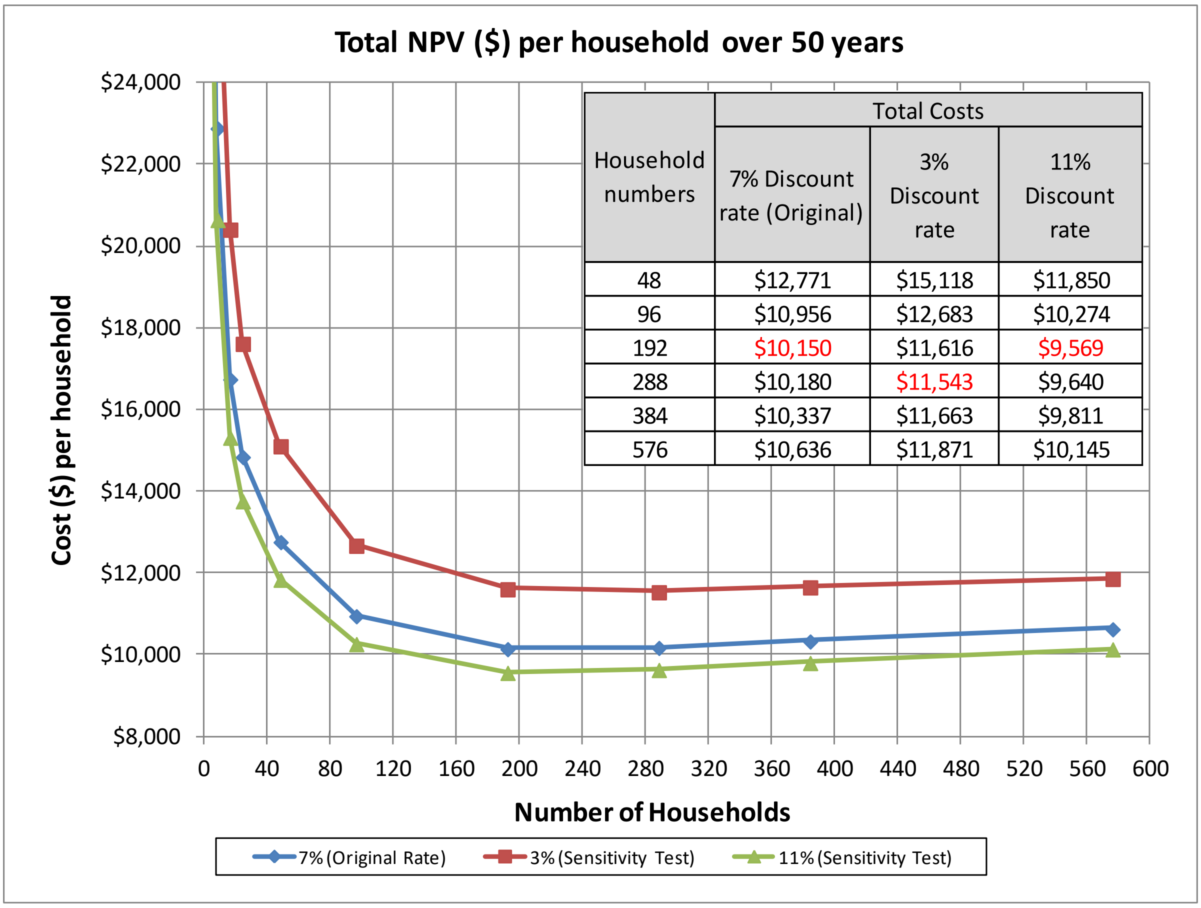
Due to the uncertainty over the discount rate used, the OBPR (2007) recommends discount rates of 3% and 11% for sensitivity analysis purposes. Capital costs will not be affected by changes in the discount rates as these are the costs incurred at the time of commissioning a project. However, ongoing costs (maintenance, operational and replacements) would all be affected and would affect final optimal scale results; i.e., using a lower rate of 3% would increase overall costs, whilst the higher rate would decrease it. Table shows the results of the different discount rates used in the sensitivity testing for the ongoing cost categories when compared against the values obtained from the original rate of 7%.

**Table 36: Results of sensitivity analysis as a comparison against the original obtained values.**

| Household Numbers | 3% Discount Rate | | | | 11% Discount Rate | | | |
|---|---|---|---|---|---|---|---|---|
| | Maintenance | Operation | Replacement | Total Costs | Maintenance | Operation | Replacement | Total Costs |
| 4 | 187% | 186% | 232% | 129% | 65% | 66% | 52% | 89% |
| 8 | 187% | 186% | 232% | 125% | 65% | 66% | 52% | 90% |
| 16 | 188% | 186% | 233% | 122% | 65% | 66% | 51% | 91% |
| 24 | 188% | 186% | 233% | 119% | 65% | 66% | 51% | 93% |
| 48 | 188% | 186% | 233% | 118% | 65% | 66% | 51% | 93% |
| 96 | 188% | 186% | 232% | 116% | 65% | 66% | 52% | 94% |
| 192 | 188% | 186% | 232% | 114% | 65% | 66% | 52% | 94% |
| 288 | 188% | 186% | 232% | 113% | 65% | 66% | 52% | 95% |
| 384 | 188% | 186% | 232% | 113% | 65% | 66% | 52% | 95% |
| 576 | 188% | 186% | 232% | 112% | 65% | 66% | 52% | 95% |

From the results of the sensitivity analysis, it can be seen that applying a lower discount rate of 3% affects final results more, with rises in costs overshadowing the decrease in costs from the higher discount rate of 11%. Prices rose by more than 85% of original costs, as compared to a decrease of 35% for both maintenance and operational costs for lower and higher rates respectively. Furthermore, replacement costs went up by more than 130% of the original value for the 3% discount rate, whilst it dropped by 50% for the 11% discount rate. This also showed that the cost of replacement was the most sensitive component of the ongoing cost categories to the discount rate, as it had the highest change amongst all. Total overall costs were also affected more by the lower discount rate, which rose by more than 12% for all households, whilst the maximum drop for the higher rate was 11%.

Figure shows similar trends in results of both sensitivity analyses when compared to that of the original discount rate used, with an initial sharp drop before flattening off and eventually rising. From previous analysis of the original cost curve, this trend signifies the influence which capital costs still have over overall costs.



| Household numbers | Total Costs | | |
|---|---|---|---|
| | 7% Discount rate (Original) | 3% Discount rate | 11% Discount rate |
| 48 | $12,771 | $15,118 | $11,850 |
| 96 | $10,956 | $12,683 | $10,274 |
| 192 | $10,150 | $11,616 | $9,569 |
| 288 | $10,180 | $11,543 | $9,640 |
| 384 | $10,337 | $11,663 | $9,811 |
| 576 | $10,636 | $11,871 | $10,145 |

**Figure 13:** **Results of Sensitivity Analysis on cost per household of communal systems over 50 years.**

Whilst it is once again difficult to estimate the size of optimal housing from the cost curves obtained in the sensitivity analysis, the inset table in Figure shows minimum costs occurred at 192 and 288 households for the higher and lower discount rates respectively. This indicates that, although discount rates affect final costs, the occurrence of the optimal housing layout remain relatively unchanged, with numbers lying between 192 and 288 households, which is where the trough of the cost curve occurs. It also shows that the discount rate would only slightly affect the household layout numbers at which the household cost of the system would be minimum, as the discount rate mainly changes the ongoing costs; albeit not by a great extent as the minimum cost is restricted between these two household layouts. The outcomes from the sensitivity analysis agree with Clark (1997), who also noted change in overall costs when carrying out sensitivity test in the discount rate, but no difference on the scale of servicing.

The results from the sensitivity testing indicate that the optimal housing scale number is not influenced by the market rates of interest and inflation, which is the source of uncertainty for the discount rates. Total NPV are as expected, either higher or lower than the original obtained value, depending on the discount rate used in the sensitivity analysis.

### 5.3.5. Influence of Land Topography

In the arbitrary household layout, the land is taken to have a flat surface with gravity collection pipes transporting water to the storage tanks. Final NPVs could be reduced if the topography of the land is ideally suited for a communal harvesting system to collect rainwater, i.e., the slope of the ground equal to or greater than 0.5% (the gradient used in the pipe design) so that all main collection pipes can use gravity to transport rainwater. However, the cost of distribution would be slightly higher due to the higher pumping energy required for pumping.

Ground with a gradient greater than 0.5% would lower costs as collection pipes would follow the slope of the ground resulting in no depth factors being applicable for pipe laying costs; i.e., collection pipes would not be laid at more than 1.5 meters depth. Costs would be further reduced if the ground slopes by more than 0.5%, as smaller pipe sizes would be able to be used in the collection pipe network.

A slope of greater than 0.5% would also ensure that there will be no requirement for a sump well to initially collect rainwater, as pipe depths would not be buried below 1.5m; the depth when a sump well would be required, and the inflow pipe would be connected straight to the rainwater storage tank. The omission of a sump well would not only save on the initial capital costs for the construction of a communal system, ongoing costs would be reduced as well, as there would be no need for the maintenance, operation or replacement for the components of a sump system.

Within the analysis carried out on land having a flat surface (less than 0.5% slope), treated rainwater distribution pipes are assumed to be laid parallel to the ground surface, whilst collection pipes have a 0.5% slope, thus resulting in separate trenches being required to lay these pipes. Having a communal system built on a landscape with a gradient would reduce pipe laying costs further as distribution and collection pipes could be placed in the same trench. This is backed up by Booker (1999), who noted a 16% reduction in the cost of the pipe network by combining pipes in the same trench. The potential health risk due to cross contamination between the two pipes is also low as raw rainwater from roofs is comparatively a clean source of water.

### Effects of Topography on Overall Costs

Using the optimal household configuration of 192 homes as a baseline, a desktop analysis was carried out to compare the effects land slope has on the final cost values. This investigation assumed that the slope of the ground for this scenario is the same as that taken for the initial gradient of the pipes in the analysis; i.e. 0.5%. Cost data were omitted for the sump system, depth factors for pipes were not taken into consideration and collection and recirculation pipes were assumed to be laid in parallel, triggering a 16% reduction in overall pipe costs. Results showing the difference between the two systems are shown in Table 37.

**Table 37: Cost differences, on an individual household basis, between a flat and sloped topography for a 192-house layout over 50 years.**

| Layout | Overall NPV | Pipes | Storage Units | Treatment Units | Maintenance | Operation | Replacements | Capital Costs (Pipes, Storage and Treatment) |
|---|---|---|---|---|---|---|---|---|
| Flat Topography (Original) | $10,150 | $4,522 | $3,849 | $177 | $1,149 | $325 | $128 | $8,548 |
| Sloped Topo (Adjusted) | $8,770 | $3,654 | $3,466 | $177 | $1,080 | $291 | $102 | $7,297 |
| % of original cost | 86.4% | 80.8% | 90.1% | 100.0% | 93.9% | 89.5% | 79.9% | 85.4% |

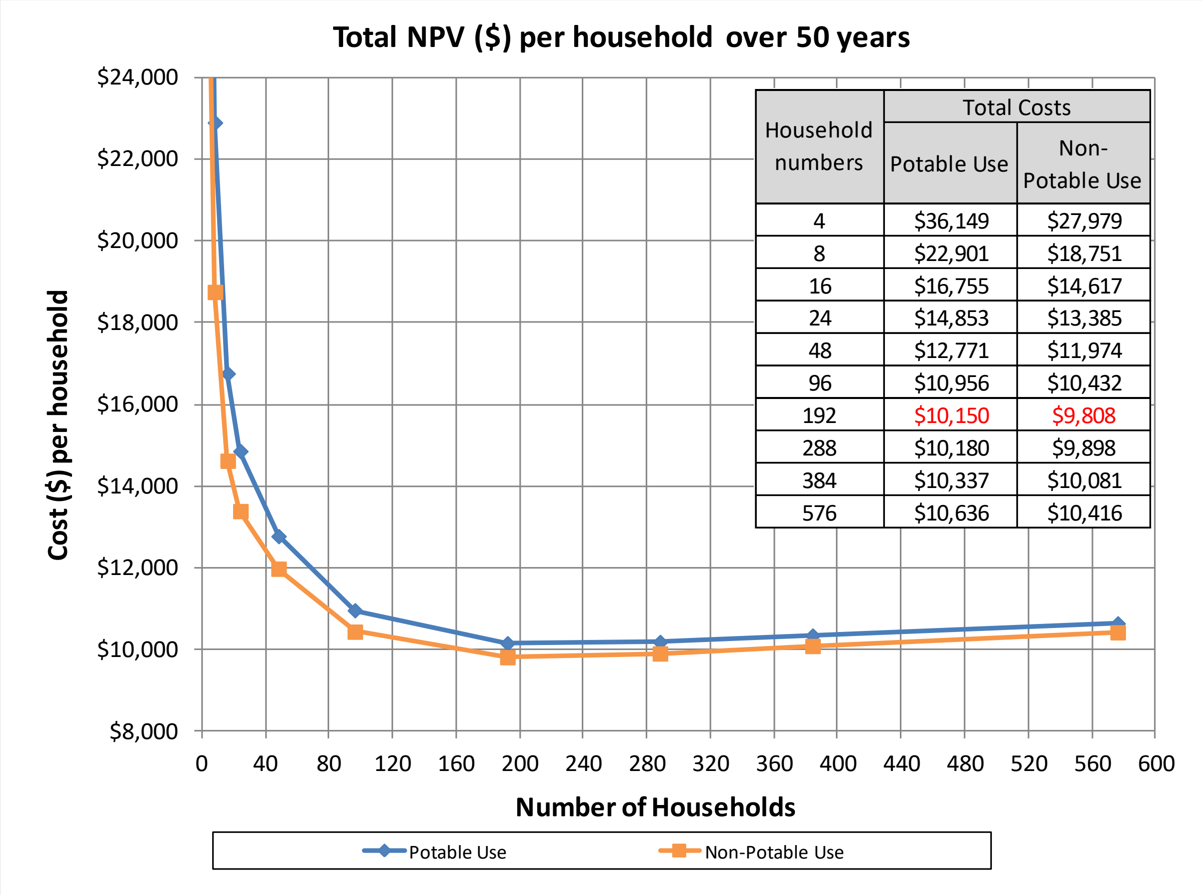
Table 34 shows that, with a sloping site, there are overall reductions in costs in all categories, except in treatment units. Pipe costs have dropped by almost 20% due to the shallow depths at which they are buried (no depth factors) combined with the use of a dual pipe system in a single trench. Treatment unit costs are unaffected as the system still supplies and treat the same amount of water as in a flat topography system, hence requiring no changes to its design. Cost of constructing the storage units reduces by 10% due to the exclusion of the sump system, resulting in a decrease in overall capital costs by 14%. This also results in a reduction to ongoing costs, which dropped to 80% to 94% of original costs. Overall NPV costs are shown to drop by approximately 13%.

Although such a physical layout is probably very rare in reality, this analysis shows that there can be a significant reduction in costs for a communal system built on sloping land. Even with a shallow gradient of 0.5%, as was used in the study, considerable reductions of 14% are observed in start-up costs due to lower pipe laying charges. Further savings would be noted if such rainwater harvesting systems were constructed on a steeper gradient as this would involve using smaller collection pipes.

With initial costs proving to contribute significantly to the overall life cycle of a communal rainwater harvesting system, it is apparent that the topography of the surrounding land would have a significant impact on the costs at an individual household level. Although this short analysis shows that the topography of the land is able to reduce the total NPV, this was based on an ideal situation, when the slope of the land follows that of the pipeline. In a different scenario, there may be requirements for a pumping station to lift the water, which would possibly raise, rather than lower, overall costs. This has been pointed out by Clark (1997), who concluded that local variations will produce costs varying significantly from the obtained average.

### 5.3.6. Impact of Removing Treatment Systems on Costs

An analysis was carried out on the effects of removing the treatment system on overall NPV costs. This analysis excluded the water treatment facility (treatment units), i.e. filtration, UV, chlorination and treated storage tanks, and the associated maintenance and operations costs. This would result in reduced overall costs, due to lower start-up and recurring costs. Figure 14 shows the cost curve obtained, with the treatment facility components removed (for non-potable use) and all others unchanged, plotted alongside the graph for potable use. It can be seen that removal of the water treatment component of the system makes no difference in the trend of the graph, with minimum costs occurring at the same household level, 192 dwellings, as shown in the inset table. As expected, overall costs are reduced for all household layouts, although the difference is small, with a decrease of 4% at the optimal household level.



| Household numbers | Total Costs | |
|---|---|---|
| | Potable Use | Non-Potable Use |
| 4 | $36,149 | $27,979 |
| 8 | $22,901 | $18,751 |
| 16 | $16,755 | $14,617 |
| 24 | $14,853 | $13,385 |
| 48 | $12,771 | $11,974 |
| 96 | $10,956 | $10,432 |
| 192 | $10,150 | $9,808 |
| 288 | $10,180 | $9,898 |
| 384 | $10,337 | $10,081 |
| 576 | $10,636 | $10,416 |

**Figure 14: Cost curves for non-potable and potable systems over 50 years.**

The removal of the treatment system would mean that the rainwater harvested would only be fit for non-potable use. However, a more detailed cost analysis should be carried out to understand the overall NPV for non-potable use as further costs reductions would be observed such as smaller holding and collection tank sizes resulting in lesser excavation. According to the end use breakdown from Beal *et al.* (2010), and omitting leaks, non-potable use would be lower in demand than potable use with consumption coming to 54 L/p/d against 82.3 L/p/d used for this study. Due to the lower demand, recirculation pumps and pipes sizes would be anticipated to be smaller, resulting in lower overall costs; including operating costs. Whilst it is noted that the NPV for a communal non-potable system would be lower than for a potable system, a more detailed cost study should be carried out to have a more direct comparison against a communal system for potable use.

## 5.4. Individual vs Communal Rainwater Systems

### 5.4.1. Results from Analysis

Although the final NPV cost curve for communal systems flattens off making it difficult to visualise the most economical viable housing layout, the minimum cost to operate a communal rainwater harvesting system with flat topography occurred at 192 households (Figure). The cost of pipes (44.6%), storage units (37.9%) and maintenance (11.3%) provided the biggest contribution to the overall cost per household. Cost of on-going operation (3.2%), treatment units (1.7%) and replacements (1.3%) provided a relatively minor contribution to the cost per household. This cost distribution is dissimilar to individual rainwater tanks, with capital (52.8%), maintenance (29%), replacement (15.8%) costs providing the major contribution to cost per household and operation costs providing a relatively small contribution (2.3%).

Total NPV for the optimal communal system over the 50-year duration per household was calculated to be $10,150, translating to an annualised cost of $735 at 7% discount rate. The price per household for a communal rainwater tank, based on its optimal costing, is more expensive than an individual rainwater tank, which has a NPV of $8,568 per household over its useful life, or $621 per annum. However, these costs are compared on an individual housing basis with water consumption being used for different purposes; potable for the communal and non-potable for the individual. Furthermore, the benefits of utilising a communal system for potable supply outweighs the costs associated with it as individual tanks would mainly be used as a non-potable source.

Sensitivity analyses for both individual and communal systems are in agreement with one another, with final costs values more sensitive to lower discount rates than higher rates. Furthermore, replacement costs for both rainwater systems were the most affected by the discount rates, with identical changes being observed; lower discount rates doubling costs and higher rates reducing costs by half.

Costs of communal rainwater systems are affected significantly by the slope of the surrounding land. Individual rainwater tanks, on the other hand, are not influenced by the topography of the land, as the collection system is dependent on roof area and gutters connected to the tank. A preliminary desktop analysis of a sloping site on a communal system resulted in an NPV of $8,770, which is close to the NPV obtained for an individual rainwater tank. However, this is based on the scenario of the treatment system being located in the centre of the development and the slope of the land being ideal for the current pipe layout. In reality, the slope of the land may vary and the costs may be higher or lower.

Removing the treatment system from the communal system, in effect transforming it into a non-potable supply source, would provide a more directly comparable figure against the individual rainwater systems. Costs per household were shown not to decline much, by 4% for the optimal housing level, and a more thorough investigation is required for comparison purposes.

The communal rainwater system for this study has been designed based on 94% volumetric reliability whilst individual rainwater tanks normally provide a much lower reliability. If a 5 kL volume was assigned for each household in the communal system, as per QDC MP 4.2, the reliability would drop to 85% (see Figure ), which would effectively mean lower costs for storage tanks for all household layouts; i.e. reduced cost per household. This implies that as reliability drops, so would the cost, signifying that costs for communal systems could potentially be comparable to individual rainwater tanks if reliability of communal tanks were dropped to equal that for individual systems. Furthermore, the demand for potable water (82.3 L/p/d) is much higher than non-potable water (54 L/p/d). If the NPV for both the rainwater systems end up being similar or comparable, then it would possibly be cheaper to operate a communal system based on levelised costs (per kL). However, with communal systems used mainly as a potable source, a high reliability is required in order to maintain a constant water supply to the householders, thus resulting in higher costs.

Additionally, for individual systems, there are issues with water quality and maintenance which were highlighted by Ahmed *et al.* (2012) and Moglia *et al.* (2012). Ahmed *et al.* (2012) discovered more than one faecal indicator in individual rainwater tanks, suggesting that additional treatment be carried out on collected rainwater for it to be deemed fit for potable purposes. Furthermore, Moglia *et al.* (2012) reported on the management of rainwater tanks and the difficulty in improving maintenance activities for such systems due to the complex nature of the socio-psychological factors relating to householder tank maintenance behaviour. With water quality and maintenance issues being a common problem, this highlights the limitations in using individual rainwater tanks as a potable water source, which is the prime advantage provided by a communal system. Water quality in communal rainwater systems can be more easily maintained for potable purposes as the maintenance would be handled by the body corporate, relieving householders of this responsibility.

Overall, it is shown that the provision of an individual rainwater system is more economical than a communal system, although the former system is mainly restricted to non-potable uses. However, communal systems would mainly be used as a potable source, with the advantages including better reliability and potable quality water, which proves to be a significant benefit outweighing the higher costs involved. Additionally, the topography of the land would also influence the final costs of a communal system and costs could potentially be reduced further depending on the layout of the system.

# 6. CONCLUSION

The aim of the study was to investigate the economics of scale of communal rainwater harvesting system and also to conduct an economic assessment of an individual rainwater tank for comparison. The Net Present Value (NPV) method for life cycle costing (LCC) was adopted for the analysis. Cost data were obtained from suppliers and literature review and applied with a 7% discount rate recommended by the OBPR (2007).

Housing layouts ranging from four to 576 homes were created to assist in obtaining the economics of scale of communal rainwater systems. A flat topography development scenario with a centralised location for storage and treatment was considered in this analysis. As no specified guidelines are available on the design of communal rainwater harvesting systems, information was collected from various literature and sources, including the analysis of the communal system at Capo di Monte (CDM) in SEQ. Basic designs of the different components of the system, including pipes, pumps, tanks and treatment units were required to develop guidance on the scale issues.

The total NPV for an individual 5 kL rainwater tank system was estimated to be $8,568 with capital costs making up the highest proportion followed by maintenance and replacement costs. Operation costs contributed the least to the overall cost per household, which was in agreement with a life cycle cost assessment study carried out by Stewart (2011). Sensitivity analysis on discount rates showed that final results are affected more by lower than higher rates, with the replacement cost component being the most sensitive to this parameter.

Capital costs have a large influence on the overall costs of communal rainwater systems and made up more than 70% of overall NPV. Further break down of the start-up costs showed pipe costs dominating initial capital costs for household layouts with more than 96 homes, whilst treatment unit costs affected household layouts with less than 96 homes. This was due to increasing pipe costs for increasing numbers of households, with the greater length of pipe required and increased pipe size, highlighting the dis-economy of scale outlined by Clark (1997) and Booker (1999). However, the opposite was true for the storage and treatment units for communal systems, with the cost per household decreasing with larger housing developments. Other cost categories, such as operational, maintenance and replacement costs were not as influential to the total NPV for communal systems.

Optimal housing scale for communal rainwater tank provision was observed to be between 192 and 288 households with the minimum for this study occurring around 200 dwellings and total NPV per household at $10,150. Capital cost was the most influential component of total costs, with pipe costs affecting households on larger scale developments. Treatment costs were most influential for developments with less than 96 households. Ongoing costs were not as influential on final cost values.

Sensitivity analysis on discount rates showed that lower rates have more influence on final results than higher rates, with the minimum cost per house shifting to the 288 household layout at low discount rates. However, the range where the most economically viable housing layout occurs is not affected by discount rate; results still lie between 192 and 288 households, indicating the influence of capital costs. Replacement costs are shown to be the most sensitive component to discount rates, with costs of replacement dropping by half for higher rates and more than doubling for lower rates.

At the communal scale, topography influences overall NPV costs at the optimal household level of 192 households, with overall costs reducing by up to 13% where the ground slope was at least 0.5%, due to a reduction in costs of all components. This was based on the assumption that, where the ground was sloping at 0.5%, collection and recirculation pipes were laid in parallel.

Removing the treatment systems, effectively converting the communal system to a non-potable supply, would contribute only 4% reduction of overall NPV at the optimal scale of development. Although this would mean rainwater being used for non-potable use, a more detailed study is warrented to have a more complete cost comparison against a communal system for potable use.

Comparisons of NPV costs between individual and communal systems at the optimal household level showed individual tanks (at $8,568) to be lower in cost per household than the communal systems on flat land (at $10,150). The costs were more comparable for a communal system with a graded slope, which was $8,770. Sensitivity testing on the discount rate came to the same conclusion, with final values being more sensitive to lower discount rates than higher discount rates and replacement costs being the most affected for both systems. However, the use of communal rainwater systems as a potable source provides better reliability and water quality with fewer maintenance issues than individual rainwater tanks, offering an advantage which offsets the higher costs involved.

In conclusion, the results presented in this report can be used as guidance for predicting breakdowns of the costs over the life cycle of both an individual and a communal rainwater system. Although communal systems are more expensive to implement than individual rainwater tanks, the benefits of communal systems to be used as a potable water supply outweigh the higher costs involved. In addition, the study also provides a methodology for carrying out future economics of scale analysis of communal rainwater systems for a variety of scenarios, including topography of the site.

# APPENDIX A - Hydraulic Analysis of Capo di Monte

The following appendix was obtained from the hydraulic analysis for Capo di Monte (CDM) and contains relevant parts useful for this report. It outlines the methodology and results obtained from a hydraulic modelling and validation study conducted for the CDM site which is a water self-sufficient community, utilising a communal rainwater system for its potable source and recycled water for its non-potable needs. This was part of a study by Cook *et al.* (2012) which looked into the performance of rainwater harvesting at the cluster scale level for a residential estate.

## Model Setup

The SWMM hydraulic model package was used to carry out the hydraulic analysis of the rainwater collection system of a greenfield development in Capo di Monte (CDM). Site drawings with pipe lengths, sizes, materials as well as measured slopes provided the main inputs for the hydraulic pipe network, with aerial photos providing an estimate on the roof areas; essentially the catchment size for the communal rainwater harvesting system. The Rational Method, which estimates the potential volume of rainfall that can be captured, was used to further validate the results obtained from the hydraulic model.

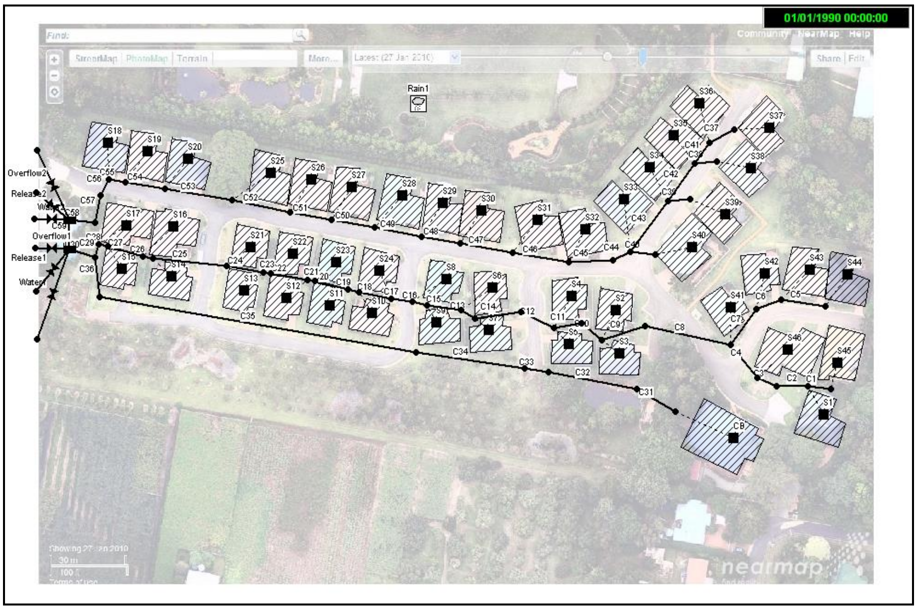


**Figure 1: Hydraulic modelling setup for the CDM rainwater collection system using the SWMM model.**

## Model Calibration

### Calibration and Validation Results

Model calibration and validations are necessary requirements as the quality of the calibration influences all of the analyses and interpretations that follow. Initial model runs with the preliminary model parameters for calibration showed that there was excess flow entering the collection system. Calibrations for the model required these excess flows to be reduced so that tank storage levels in the model runs would better match observed levels.

Parameters which were initially adjusted to try and match the observed data included pipe roughness as well as catchment (roof) widths, roughness and slopes. Altering these parameters showed little or no changes in the flow entering the system, hence, resulting in only minor differences in the rainwater tank storage levels.

For a more significant change to be observed in the modelled storage levels, sufficient modifications to the flow entering into the pipe network were required. Roof area was identified as the prime physical factor which needed to be modified for calibration purposes. A site inspection of the area, concluded that roof connectivity was 100% for the site; i.e. roofs contribute fully to the rainwater collection system. Hence, adjusting the roof areas in the model would essentially provide the runoff coefficient (in percentage terms) for the system. Runoff coefficients are losses associated with evaporation, wind effects, splashing and spillage.

Calibration runs were carried out with 85%, 87.5% and 90% of the roof areas. Modelled results of rainwater storage levels for these calibration runs matched well with observed data for the chosen event. Results from the verification runs further confirmed these values, with the average of these three values, 87.5%, taken for future analytical purposes. The following figures show some results from the calibration and validation runs produced from the hydraulic modelling study.

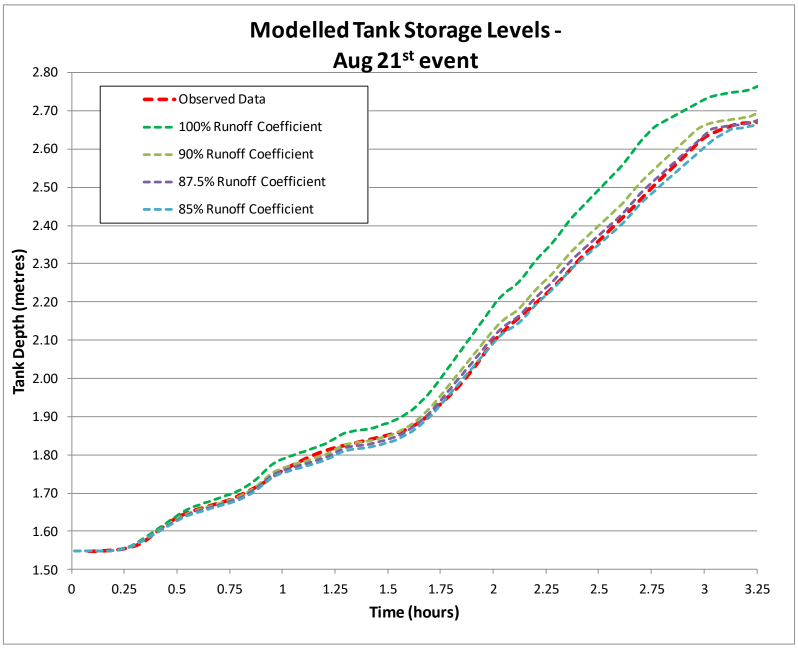


**Figure 2a:** **Calibration results from hydraulic model (Aug 21st).**

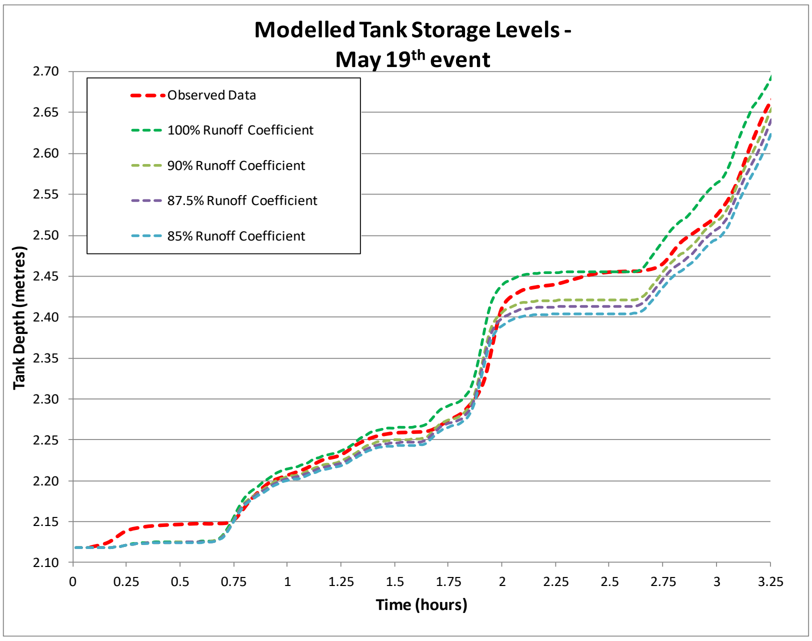


**Figure 2b:** **Validation results from hydraulic model (May 19th).**

### Rational Method

A simple method of estimating the potential volume of rainfall captured by roofs over an event is by using the Rational Method. The roof area is multiplied by the depth of rainfall over the length of the event and a runoff coefficient used to account for any losses in the system. The Rational Method is hence given as follows:

$$\text{Captured Volume (m}^3\text{)} = \text{Roof Area (m}^2\text{)} \times \text{Depth of Rainfall (m)} \times \text{Runoff Coefficient} \qquad (1)$$

All losses through evaporation, splashing, spillage and wind effects are represented by the runoff coefficient and 100% of roof areas are known to contribute to the system.

The above equation was used to analyse the chosen events used in the SWMM model's calibration and verification. Rainwater tank storage volumes were plotted against the rainfall depths for each of the events and are shown in Figure 3.

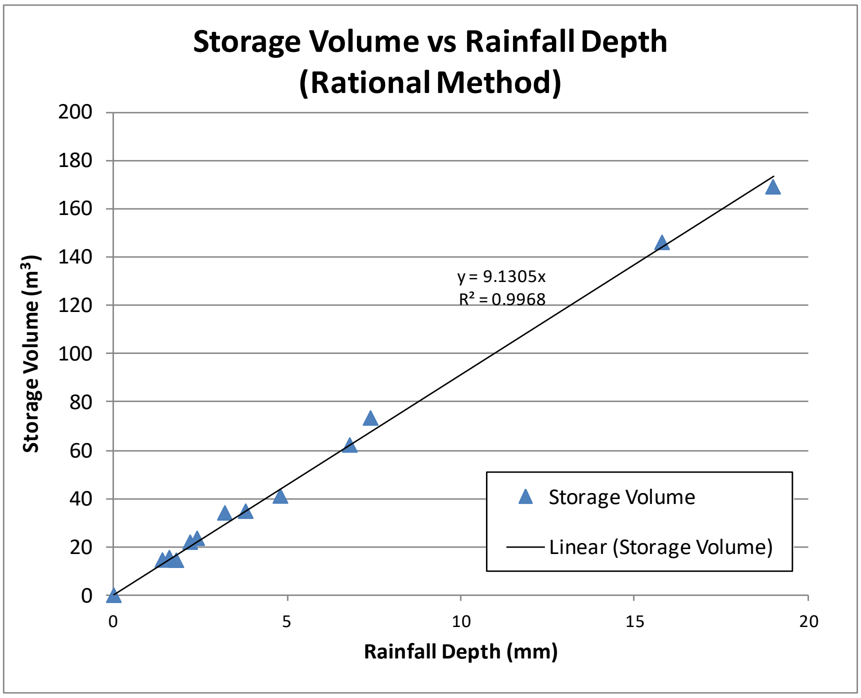


**Figure 3:** **Regression analysis of the Rational Method for CDM.**

The effective contributing roof area (roof area multiplied by runoff coefficient) for the site is estimated to be 9,130 m$^2$ from a regression analysis of the plots; this was defined by the gradient of the line. This value is approximately 85% of the total roof area of 10,689 m$^2$ of the CDM development and represents the approximate runoff coefficient for roofs for this site. The results of the analysis further consolidated the hydraulic model's estimated runoff coefficient of 87.5% ensuring more confidence in its use in further calculations within the report.

### Roof Drainage

The Queensland Urban Design Manual (DERM, 2007) recommends designing gutters and downpipes for roof drainage in accordance with Notes on the Science of Building (NSB) 151, NSB 152 and NSB 153 (CSIRO, 1978) to adequately convey runoff for a design storm with an Average Recurrence Interval (ARI) of 20 years and critical storm duration of 5 minutes. The consultant had confirmed the design of the system was for a 20 year ARI storm, although the contributing flow from each property was limited to assumed design value of 1.0L/s. This part of the hydraulic analysis will investigate the accuracy of this flow limitation into the hydraulic network.

### Design Rainfall Data

Design storms are normally produced as Intensity Frequency Duration (IFD) data; with rainfall intensities produced for a series of ARIs and storm durations, as these parameters are normally interrelated in a given location. The process of estimating IFDs, known as frequency analysis, is an important part of hydrological design procedure. The procedure for obtaining IFD tables is outlined in the Bureau of Meteorology website (BOM, 2011). Using this procedure, the design rainfall intensity for a storm with 20 year ARI and duration of 5 minutes at CDM is calculated to be 233mm/hr.

Although rainfall data is available from the rain gauge at CDM, the BOM website does not recommend carrying out analysis of rainfall data based on a single station as it would be unreliable, not temporally and spatially consistent and hence unfit for design purposes. Instead, consistent and accurate IFD data derived for the whole of Australia and which are readily available from the website, should be used.

### Downpipe Flows

Estimations of flow through a downpipe are important to determine if the roof is capable of draining design storms adequately. DERM (2007) suggests roof drainage to be able to convey design storms of 20 years ARI and 5 minute duration.

On average, there are approximately 6 downpipes connecting the roof to the rainwater pipe network at each property in CDM, with roof runoff assumed to be equally distributed into the downpipes. To estimate peak flows obtained from a roof area, the general form of the Rational Method can be used (DERM, 2007):

$$Q = \frac{C.I.A}{3.6 \times 10^6} \cdots\cdots\cdots\cdots (2)$$

*Where:*

Q = peak flow rate ($m^3$/s), C = coefficient of discharge, I = intensity of rainfall (mm/hr) and A = area of catchment ($m^2$).

Using the Rational Method, the peak flow obtained for each roof for the 20 year ARI, 5 minute duration storm is calculated to be 0.01257$m^3$/s. The coefficient of discharge used in this case was the runoff coefficient obtained from the hydraulic model analysis of the site, i.e. 87.5%. Hence, flow down each downpipe is calculated to be = 0.01257/6 = 0.0021$m^3$/s.

To check if the downpipes are able to cope with such flows, pipe full flows down the downpipes can be estimated using the orifice equation (May, 1997):

$$Q_{dp} = B.C.A\sqrt{2.g.H} \cdots\cdots\cdots\cdots (3)$$

*Where:*

$Q_{dp}$ = flow in the downpipe ($m^3$/s), B = blockage factor (1 if no blockage), C = orifice discharge coefficient (0.6 on average), A = downpipe cross section area ($m^2$), g = gravitational acceleration (9.81 m/$s^2$) and H = depth of water (m) in gutter.

The size of the gutters were measured to be 115mm x 90mm (W x D). NSB 151 recommends a freeboard allowance of 50mm to account for ripples and turbulence; effectively reducing the carrying capacity of the gutters to 40mm. The diameter of the downpipes was measured off to be 90mm.

Assuming that there are no blockages in the gutters (B = 1), from the orifice equation, the maximum effective flow that a downpipe is able to receive is 0.0034$m^3$/s which is higher than the peak flow of 0.0021$m^3$/s produced in a 20 year storm. This shows that the downpipes are able to take the flows of a 20 year ARI, 5 min duration storm and are adequately designed.

**Downpipes Connections to Collection System**

An in-ground pipe installed around the property, connects downpipes at the corners of the building and is connected to the main collection systems through a connector. The following figure shows a schematic of the connection.

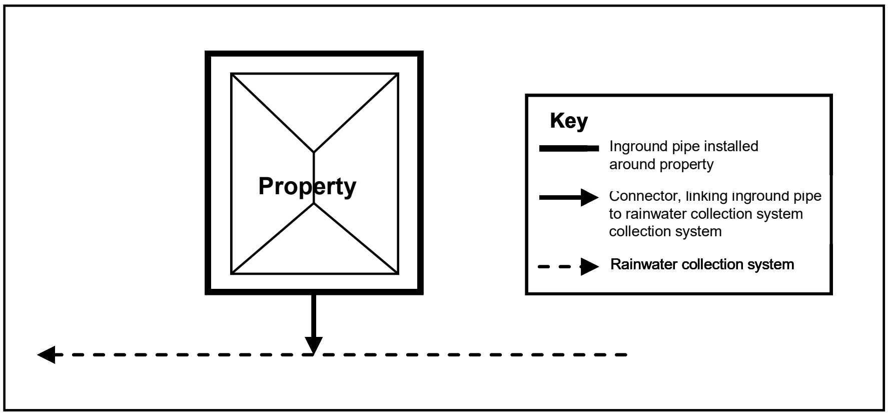


Figure 4a: **Schematic of in-ground pipe and connectors around a typical CDM household.**

For simplicity, the in-ground pipes around properties have not been modelled. Instead, runoff from the roof has been connected directly to the main pipes through the connector. Figure 4b shows a typical layout of how runoff from a roof is connected to the collection system.

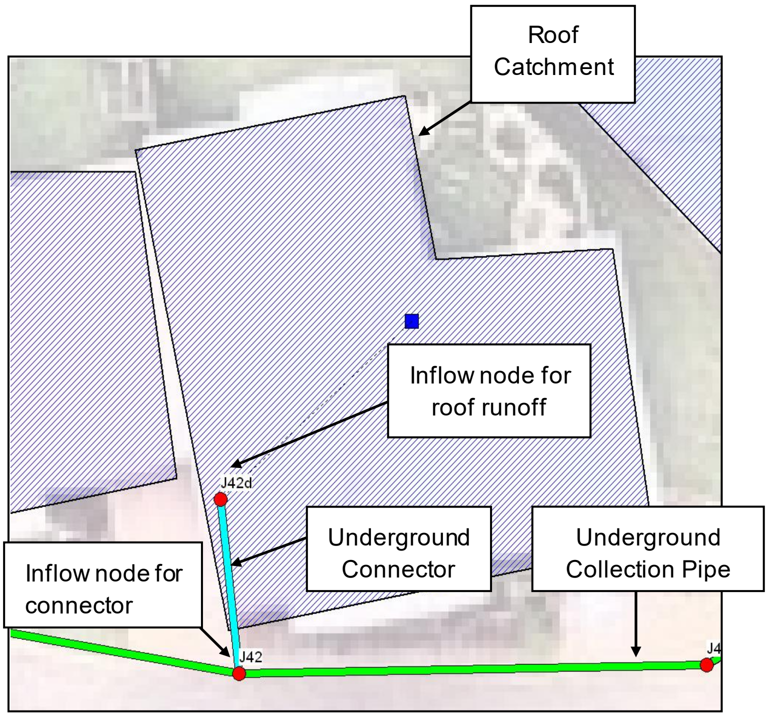


Figure 4b: **Typical layout of roof runoff connection to main pipes.**

The diameters of the connectors were unknown, as there were no drawings and information available, and have initially been assumed to be 100mm, with reference to the PotaRoo project (Fitzgibbon Chase); a similar rainwater harvesting system which will harvest roofwater from approximately 11 hectares of roof catchment in the Fitzgibbon Chase development area. A slope of 1% has been applied to the connectors, which is the minimum grade used for all roofwater pipes in the site.

Model calibration and validation had been carried out for low rainfall events, with rainfall intensities not exceeding 2mm/5min. The results from these model runs showed that flows produced from such rainfall intensities are able to fully enter the system via the connectors, with no losses from overflow into the stormwater systems. Table 3 shows the average peak runoff generated from each roof for the lower events, the corresponding peak flows in the connectors, as well as the peak intensity for the event.

**Table 3: Average rainfall flow harvested per roof (property)**

| Date | Peak runoff generated, ($m^3$/s) | Peak flow in connectors, ($m^3$/s) | Peak rainfall intensity, (mm/ 5 min) |
|---|---|---|---|
| 9-May | 0.00026 | 0.00026 | 0.4 |
| 19-May | 0.00090 | 0.00090 | 1.4 |
| 12-Jun | 0.00026 | 0.00026 | 0.4 |
| 29-Jun | 0.00129 | 0.00129 | 2.0 |
| 21-Aug | 0.00078 | 0.00078 | 1.2 |
| 6-Oct | 0.00025 | 0.00025 | 0.4 |

The simulated results show that flows up to 0.00129$m^3$/s is being contributed into the system from each property through the connectors. Furthermore, runoffs generated on the roofs, after losses are taken into account of, passes through the connectors without being impeded. The highest modelled flow of 0.00129$m^3$/s in the connector shows that each of the property within the collection system contributes more than the consultant's suggested value of 1.0L/s.

Although this gives a general estimate on the contributing flow from each property during low events, it remains to be seen whether this holds true during larger events. This is because, for higher events, the connectors, depending on their sizes, could prove a restriction to the amount of flow which is able to enter the system. Further calibration of the model was carried out to ensure that the results of simulations of higher events were well represented.

### Calibration for Higher Event

Modelled results for a rainfall event recorded on the 24th of November, peaking at 2.6mm/5min, showed more volume than was monitored, collecting at the rainwater storage tanks. This showed that each roof was allowing more flow than necessary into the system, possibly indicating that the connectors' chosen diameter of 100mm was an overestimation. Using this event, the sizes and roughness of the connectors were adjusted to further calibrate the model to ensure accurate simulations of larger events.

Results from the calibration runs indicated that a 75mm diameter connector with a roughness of 0.025 provided a better representation of the connectors in the system than the original of 100mm sized and roughness of 0.01. Although the roughness is on the high side, this can be justified for losses not accounted for within the connectors including losses at bends, length of inground pipe around property linking downpipes to the connector, the frictional losses for this length of pipe and the generally small hydraulic surface area of the connector. The figure shows the tank storage levels with the original sizing of 100mm and with the adjusted sizing of 75mm and roughness of 0.025 for this event.

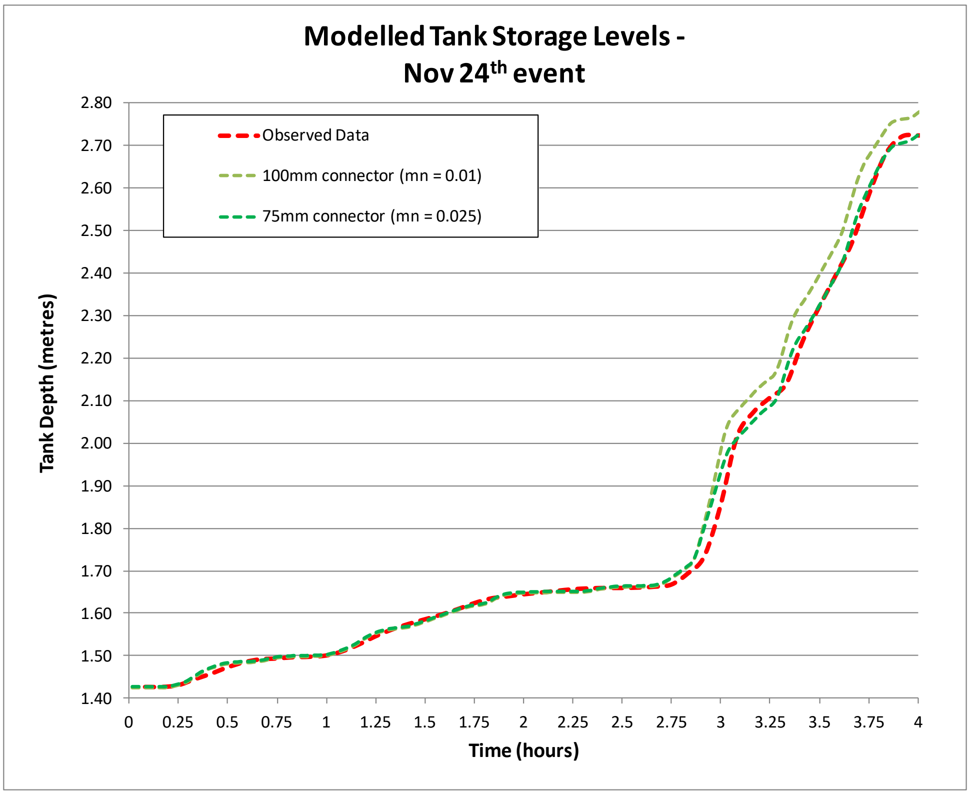


**Figure 5: Calibration result for higher intensity rainfall (24th Nov).**

Flows generated from one of the "higher" smaller event, the June 29th event, had a maximum intensity of 2mm/5 min producing a peak flow of 0.00129m$^3$/s which, with the initial sizing of 100mm, was able to pass fully through the connectors. Using the updated connector size of 75mm, the connector full capacity is also at 0.00129m$^3$/s, indicating that the system is designed to handle a storm of such an intensity. Any flow beyond this, from an event of a higher intensity, would overflow into the stormwater network.

For this reason, earlier calibrations and verification simulations need not require further investigations with the updated connector size, as the models were run with rainfall intensities of equal or lower intensity than 2mm/5min; essentially, the maximum generated runoff from the roofs would be equal or less than the connectors' capacity of 0.00129m$^3$/s.

## Conclusions

The hydraulic modelling study shows that the Rational Method is able to predict with some accuracy, the volume of rainfall potentially collected by roofs from a rainfall event provided the runoff coefficient is of a reasonable assumption, ranging between 0.85 (85%) to 0.90 (90%). An average of 87.5% is thought to be reasonable for use.

Although the gutters and downpipes are able to take design flows of a 20 yr ARI, 5min storm, ultimately the structure controlling the amount of flow entering the collection system is the connector of the downpipes to the pipe network.

Each property within the site is able to contribute a maximum of 0.00129m$^3$/s (1.29L/s), which is higher than the consultant's estimate of 1.0L/s, to the rainwater system. This is due to the size of the connector, which effectively limits the flow entering the main collections pipe.

From the rational method, the system is calculated to harvest approximately a maximum rainfall intensity of 2mm/5min.

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# APPENDIX B - Costs Data (Individual Rainwater Tanks)

The following appendix shows the costs data obtained from suppliers and plumbers.

| *Pump Costs* | | | *Rainwater Tank Costs* | |
|---|---|---|---|---|
| **Dealer** | **Quote** | | **Dealer** | **Quote** |
| 1 | $705 | | 1 | $1,524 |
| 2 | $950 | | 2 | $1,204 |
| 3 | $735 | | 3 | $1,301 |
| 4 | $680 | | 4 | $1,563 |
| 5 | $680 | | 5 | $1,685 |
| 6 | $875 | | 6 | $1,298 |
| 7 | $845 | | 7 | $1,665 |
| 8 | $1,300 | | 8 | $1,505 |
| 9 | $956 | | 9 | $1,695 |
| 10 | $695 | | 10 | $1,849 |
| **Average Cost:** | **$842** | | **Average Cost:** | **$1,529** |

| *Plumbing Costs* | | | *Tank Installation Costs* | |
|---|---|---|---|---|
| **Plumber** | **Quote** | | **Installer** | **Quote** |
| 1 | $1,000 | | 1 | $400 |
| 2 | $1,000 | | 2 | $350 |
| 3 | $750 | | 3 | $350 |
| 4 | $800 | | | |
| **Average Cost:** | **$888** | | **Average Cost:** | **$367** |

| *Laying Concrete Slab (2.1m x 2.1m x 0.1m thick)* | | | *Pump Installation Costs* | |
|---|---|---|---|---|
| **Installer** | **Quote** | | **Installer** | **Quote** |
| 1 | $600 | | 1 | $250 |
| 2 | $600 | | | |
| 3 | $650 | | | |
| 4 | $850 | | | |
| 5 | $800 | | | |
| **Average Cost:** | **$700** | | **Average Cost:** | **$250** |

## Life Cycle Costing Table for Individual Rainwater Tanks

| | |
|---|---|
| **Analysis period (years)** | 50 |
| **Discount rate (%)** | 7.00% |
| **Enery cost ($/kWh)** | 0.22759 |
| | |

| System Details | Description | Quantity No. | Unit | Unit Cost | Capital Cost | Annual Cost | | Life of Component | Net Present Value (including capital costs) |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Maintenance | Energy Cost | | |
| | | | | | | | | | |
| **Rainwater Tanks (Individual)** | | | | | | | | | |
| ***Installation*** | | | | | | | | | |
| | Plumbing | 1 | No. | $888.00 | $888.00 | | | 50 | $888.00 |
| | Concrete Laying (foundation for rainwater tank) | 1 | No. | $700.00 | $700.00 | | | 100 | $700.00 |
| | Tank Installation | 1 | No. | $367.00 | $367.00 | | | 25 | $434.62 |
| | Electrical Installation (pumps) | 1 | No. | $200.00 | $200.00 | | | 10 | $392.98 |
| ***Main Items*** | | | | | | | | | |
| | Tank with Accessories | 1 | No. | $1,529.00 | $1,529.00 | | | 25 | $1,810.72 |
| | Pump (550W -700W) with Auto Mains Switchover | 1 | No. | $842.00 | $842.00 | | | 10 | $1,654.46 |
| ***Maintenance Costs*** | | | | | | | | | |
| | Sediment Check/ Cleaning | 1 | No. | $162.00 | | | | 3 | $691.88 |
| | Gutter Maintenance, etc | 1 | No. | | | $80.00 | | | $1,104.06 |
| | Check Signage, pumps, filters, water quality | 1 | No. | | | $50.00 | | | $690.04 |
| ***Electricity*** | | | | | | | | | |
| - | Electricity | 64 | kWh | $0.2276 | | | $14.57 | | $201.02 |
| | | | | | | | | | |
| **Total Cost** | | | | | **$4,526.00** | **$130.00** | **$14.57** | | **$8,567.78** |

# APPENDIX C - Life Cycle Costing for Communal Rainwater Systems

Based on an analysis for 192 homes.

**Life cycle cost of rainwater tanks**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | **Analysis period (years)** | | | 50 |
| | Homes Analysed = | 192 | | | | **Discount rate (%)** | | | 7.00% |
| | | | | | | **Enery cost ($/kWh)** | | | 0.22759 |

| **System Details** | **Description** | **Quantity No.** | **Unit** | **Unit Cost** | **Capital Cost** | **Annual Cost**<br>**Maintenance** | <br>**Energy Cost** | **Life of Component** | **Net Present Value (including capital costs)** |
|---|---|---|---|---|---|---|---|---|---|
| **Rainwater Tanks (Communal)** | | | | | | | | | |
| **Excavation** | Excavation works (rainwater tanks) | 2870 | m3 | $129.20 | $370,804.00 | | | 100 | $370,804.00 |
| | Excavation works (sump well) | 290 | m3 | $141.70 | $41,093.00 | | | 100 | $41,093.00 |
| | | | | | | | | | |
| | **Pipe Laying** | | | | | | | | |
| **Collection System** | ***Lateral System*** | | | | | | | | |
| | uPVC (75mm) | 592 | m | $78.00 | $46,176.00 | | | 100 | $46,176.00 |
| | uPVC (75mm) - depth:1.5m~3m | 176 | m | $92.00 | $16,192.00 | | | 100 | $16,192.00 |
| | | | | | | | | | |
| | ***Main Pipe*** | | | | | | | | |
| | uPVC (75mm) | 64 | m | $78.00 | $4,992.00 | | | 100 | $4,992.00 |
| | uPVC (100mm) | 224 | m | $111.00 | $24,864.00 | | | 100 | $24,864.00 |
| | uPVC (150mm) | 832 | m | $178.00 | $148,096.00 | | | 100 | $148,096.00 |
| | uPVC (225mm) | 480 | m | $270.00 | $129,600.00 | | | 100 | $129,600.00 |
| | uPVC (225mm) - depth:1.5m~3m | 396 | m | $321.00 | $127,116.00 | | | | $127,116.00 |
| | uPVC (300mm) - depth:1.5m~3m | 116 | m | $444.00 | $51,504.00 | | | 100 | $51,504.00 |
| | | | | | | | | | |
| **Recirculation System** | uPVC pipe (100mm) - Lateral systems | 768 | m | $111.00 | $85,248.00 | | | 100 | $85,248.00 |
| | uPVC pipe (100mm) - Main Pipe | 2112 | m | $111.00 | $234,432.00 | | | 100 | $234,432.00 |
| | | | | | | | | | |
| **Sump System** | Sump Well (100kL) | 1 | No. | $18,000.00 | $18,000.00 | | | 100 | $18,000.00 |
| | Manhole - depth:2.0m~3.0m | 1 | No. | $8,010.00 | $8,010.00 | | | 50 | $8,010.00 |
| | Sump Pump - CRI20-01 | 2 | No. | $2,055.00 | $4,110.00 | | | 12 | $7,264.67 |
| | Sump Pump Installation Cost | 1 | No. | $1,650.00 | $1,650.00 | | | 12 | $2,916.48 |
| | | | | | | | | | |
| **Rainwater Collection** | 4 x 480KL Rainwater Collection Tank | 4 | No. | $58,700.00 | $234,800.00 | | | 50 | $234,800.00 |
| | | | | | | | | | |
| **Rainwater Transfer** | Additional Pump Equipment | 1 | No. | $3,685.00 | $3,685.00 | | | 50 | $3,685.00 |
| | Transfer Pump (370W) - CRI5-02 | 2 | No. | $1,185.00 | $2,370.00 | | | 12 | $4,189.12 |
| | Transfer Pump Installation Cost | 1 | No. | $1,650.00 | $1,650.00 | | | 12 | $2,916.48 |
| | | | | | | | | | |
| **Rainwater Holding** | 120kL Holding Tank (concrete) + Installation | 1 | No. | $20,100.00 | $20,100.00 | | | 100 | $20,100.00 |
| | Concrete Laying (foundation for holding tank) | 0 | No. | $700.00 | $0.00 | | | 100 | $0.00 |
| | | | | | | | | | |
| **Rainwater Recirculation** | Additional Pump Equipment | 1 | No. | $5,380.00 | $5,380.00 | | | 50 | $5,380.00 |
| | Recirculation Pump - (4000W) -- CRE20-03 | 2 | No. | $6,550.00 | $13,100.00 | | | 12 | $23,155.05 |
| | Recirc. Pump Installation Cost | 1 | No. | $1,650.00 | $1,650.00 | | | 12 | $2,916.48 |
| | | | | | | | | | |
| | Pumps Commissioning | 3 | No. | $660.00 | $1,980.00 | | | 12 | $3,499.77 |
| | | | | | | | | | |
| | **Treatment Unit** | | | | | | | | |
| **Treatment Unit** | Shelter for Treatment units and Pumps | 1 | No. | $39,032.00 | $39,032.00 | | | 100 | $39,032.00 |
| | Sand Filters + UV (including installation) | 1 | No. | $2,500.00 | $2,500.00 | | | 12 | $4,418.90 |
| | Chlorination (including installation) | 2 | No. | $1,500.00 | $3,000.00 | | | 12 | $5,302.68 |
| | | | | | | | | | |
| **Annual Costs** | Sediment Check/ Cleaning (Storage Tank) | 4 | No. | $4,800.00 | | $19,200.00 | | 3 | $82,001.03 |
| | Sediment Check/ Cleaning (Sump Well) | 1 | No. | $2,000.00 | | $2,000.00 | | 3 | $8,541.77 |
| | Gutter Maintenance | 192 | No. | $40.00 | | $7,680.00 | | | $105,989.73 |
| | | | | | | | | | |
| | Maintenance of Pumps | 3 | No | $350.00 | | $1,050.00 | | | $14,490.78 |
| | Maintenance of filters+UV | 1 | No. | $300.00 | | $300.00 | | | $4,140.22 |
| | Maintenance for Chloine doser | 2 | No. | $200.00 | | $400.00 | | | $5,520.30 |
| | | | | | | | | | |
| | **Electricity** | | | | | | | | |
| | Pumps (Recirc) | 15700 | kWh | | | | | | |
| | Pumps (Transfer) | 1800 | kWh | | | | | | |
| | Pumps (Sump Pump) | 2080 | kWh | | | | | | |
| | Treatment Units (UV) | 270 | kWh | | | | | | |
| | Total Electricity Usage | 19850 | kWh | $0.2276 | | | $4,517.66 | | $62,347.10 |
| | | | | | | | | | |
| **Total Cost** | | | | | **$1,641,134.00** | **$30,630.00** | **$4,517.66** | | **$1,948,734.56** |
| | | | | | | | | Total NPV per household = | **$10,149.66** |

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