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Communal Rainwater Tank Systems Design and Economies of Scale

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Abstract

Communal rainwater tank systems provide an alternative urban water supply solution for reducing dependence on centralised water supply networks. Rainwater from household roofs is transported through a gravity collection system and stored in a centralised communal tank before being treated and supplied back to homes through a reticulated pumping system. Literature on the design, life cycle costing and economies of scale of communal rainwater tank systems is currently limited. This study intends to develop a methodology for the system design, assess the economies of scale of communal systems and identify the main cost contributors for the total capital and life cycle costs. A methodology developed for this analysis is presented for the benefit of water professionals across the globe to support similar studies in their local regions. Housing layouts were developed, designed and costed for a flat topography and a centralised storage and treatment scenario, ranging from 4 to 576 homes. An economic assessment was then carried out using the net present value method (NPV). The results show that costs of storage and treatment units are more influential for a group of households at lower scale, whilst the diseconomy of scale of pipes is a major cost factor for higher scale of household groups. An optimal scale was observed between 192 and 288 households and sensitivity analysis on the discount rate showed no changes within this range. A basic analysis showed that topography of the land does influence overall NPV. However, the influence factor depends on the nature of the slope, with costs varying for differing scenarios and further work required to have a thorough understanding of its influence in final NPV.

Keywords: Urban water; water demand management; cost contributors; system design; economies of scale; communal rainwater tanks.

1 Introduction

In 2012, Australia emerged from more than a decade of droughts. The experience highlighted the delicate water security predicament facing a population that is already living on an exceptionally dry continent. At one point, dam storages levels in South East Queensland (SEQ) fell to 17% of capacity (SEQWGM, 2010). With increasing pressure on existing water infrastructure from population growth and climate change, the need for alternative water supply solutions to reduce the reliance on potable water from the water grid has been recognised. One option for addressing this need could be through the implementation of decentralised systems, which involves the collection, treatment and use of rainwater, groundwater or wastewater at different spatial scales (Cook et al., 2009; Cook et al., 2013).

Rainwater tanks are already an established feature of individual households in many parts of the world. Mandatory regulations have been implemented in various countries requiring the installation of rainwater tanks for new buildings with certain garden sizes (Catalonia, Spain) and roof area greater than 100m^2 (Belgium) (Domenech and Sauri, 2011). In Australia, 26% of households use a rainwater tank as a source of water (ABS, 2010). Prior to 2013, households in SEQ were required to fulfil water saving targets of 70 kL per annum through the installation of a 5 kL tank connected to a 100 m^2 roof area or half of the available roof area, whichever is the lesser of the two, under the Queensland Development Code (QDC) Mandatory Part (MP) 4.2 (DIP, 2008).

Studies involving single household rainwater tanks within the SEQ region have shown that water savings of 40 to 58 kL per household per year (kL/hh/yr) could be achieved (Beal et al., 2012; Chong et al., 2011; Maheepala et al., 2013; Umapathi et al., 2013). However, system failure and maintenance issues may reduce the positive impacts which rainwater tanks have on mains water savings. Indeed, social research conducted in SEQ has highlighted householders' motivation and skills to adequately maintain a single dwelling rainwater system varies, resulting in issues with ongoing maintenance that may lead to increased failure rates of system components and poor water quality (Mankad et al., 2012; Walton et al., 2012).

An alternative option to counteract the likely maintenance problems encountered in single household rainwater harvesting systems is to implement communal rainwater tank systems, which collect, store and treat rainwater across multiple households within a residential development. The treated water can then be supplied back to homes for either potable or non-potable purposes. Communal tanks are intended to be plumbed for internal household uses and hence, require a continuous supply to avoid disruptions of such use. As the systems are climate dependant, achieving 100% reliability is improbable and hence requires a supplementary source in the form of top-up from mains supply (if accessible) or on-site bore water supply. These systems could resolve recurring maintenance issues and potential health risks, since a maintenance organisation body would usually be employed to take responsibility of operation and maintenance, as opposed to individual rainwater tanks where the home owner is solely responsible. Thus, communal rainwater harvesting systems are being considered as potential potable options in greenfield developments with the aim to reduce dependence on fresh water supplies.

Literature on the design, economies of scale and life cycle costing of communal rainwater harvesting systems is currently limited, as it is a relatively new and emerging approach in the Australian context. A financial assessment of a communal rainwater harvesting (RWH) system in the UK resulted in average annual savings of 756 GBP with a payback period of 23 years (Ward et al., 2010) although this did not delve in the economies of scale of such a system. A comparison of two separate studies in Florianópolis, Brazil, demonstrated the economies of scale of using a rainwater system in multi-dwellings, with reduced payback periods of less than 5 years for 3 blocks of four-storey apartments (Ghisi and Ferreira, 2007) obtained, against more than 20 years for single dwelling households (Ghisi and de Oliveira, 2007). Domenench and Sauri (2011) showed similar results for a study in Sant Cugat del Vallès, Spain, with payback periods significantly lower for a multi-family dwelling (14 flats) against a single family house for a range of rainwater tank supplied end uses.

Booker (1999) investigated into the economies of scale for greywater collection, treatment and reuse systems, and demonstrated a diseconomy of scale in pipe networks affecting system size of above 12,000 connections whilst treatment units were the dominating costs for connections at the lower scale (<1,200). A separate study conducted by Clark (1997) used a simple communal sewer model and historic pipe cost data from South Australia Water to demonstrate the diseconomies of scale prevalent in pipe collection systems. An analysis by Fane et al. (2002) into Clark's (1997) study was in agreement with Booker's (1999) observations and showed an economy of scale existed below 500 connections with treatment costs dominating, whilst a slight diseconomy of scale was present beyond 10,000 connections. Sensitivity testing on the discount rate carried out by Clark (1997) showed slight changes in the life cycle costs, although there was no significant difference at where the optimal household was located. Furthermore, Clark (1997) concluded that local factors will influence costs varying from the averages, although the findings are believed to reflect the average situation.

This paper presents the results of a study investigating the economies of scale of a communal rainwater harvesting system through a desk study that quantifies the whole life cycle costs of the system using the net present value (NPV) method of life cycle costing. A methodology used for this study is also presented for the benefit of water professionals to conduct similar studies in other parts of the world.

2 Methodology

A methodology was developed for designing and understanding the optimal scale of communal rainwater tank systems. Such systems harvest rainwater from the roofs of multiple dwellings which then flow through a gravity collection system to communal rainwater tanks, where it can be stored and treated, then pumped back to homes for fit for purpose applications. The rainwater collection potential, estimated communal rainwater tank capacity, distribution system, treatment units and water capacities were designed and costed based on the housing layout, density of housing and its topography. The process was repeated for various scales of housing layouts and the communal harvesting system cost per household of various layouts was then compared. As each housing layout will be designed under similar specifications, the housing layout that has the minimum cost per household was considered to be the optimal scale for a communal rainwater tank system. Variations in design approaches and cost data for different states and countries will exist, which must be considered when utilising the outlined methodology. The overall methodology is described in the following steps and depicted in Fig. 3:

1. Select a typical housing layout being adopted in new greenfield developments based on information from local state housing development agencies and local developers. Collect information of variables which may influence the system design such as average size of housing land, average roof area, housing density, street width, historical rainfall data and public open spaces.
2. Develop a typical housing layout to be used in the housing developments of various scales with varying number of houses in each planned development.
3. Develop layouts of various housing scale developments (4, 8, 16, 24, 48, etc) as shown in Fig. 1a and 1b for 4 and 24 homes respectively.

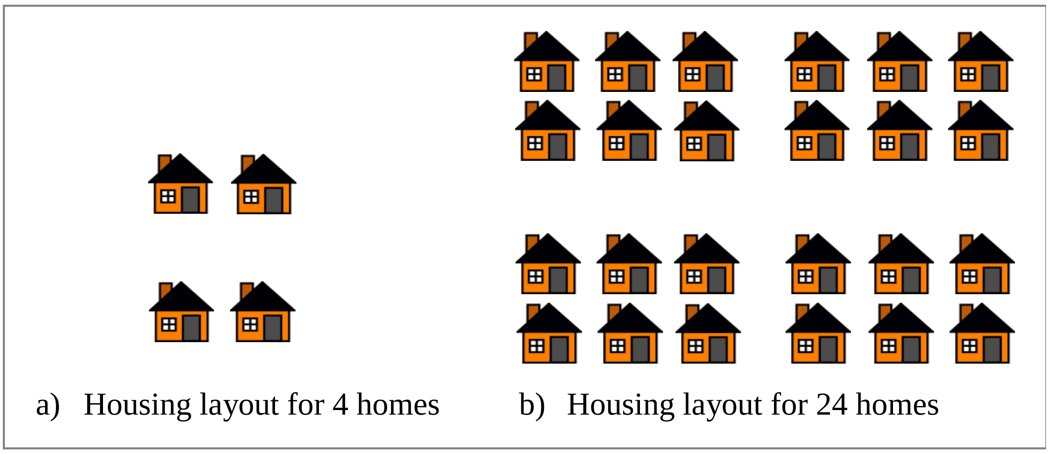


Fig. 1. Examples of layouts of various housing scale developments.

4. Select the location of a communal rainwater tank for each housing layout considering the overall topography of the area. For a development on a flat terrain, the communal rainwater tank should be situated in the centre of the development (Fig 2a) to minimise the depths of pipes which increases the cost of rainwater collection and supply networks as a result of pipe depth factors (Table 2). Alternatively, in the case

of a sloping topography, the communal rainwater tank can be located on the lower side of the housing development to maximise the benefits of the land gradient (Fig. 2b).

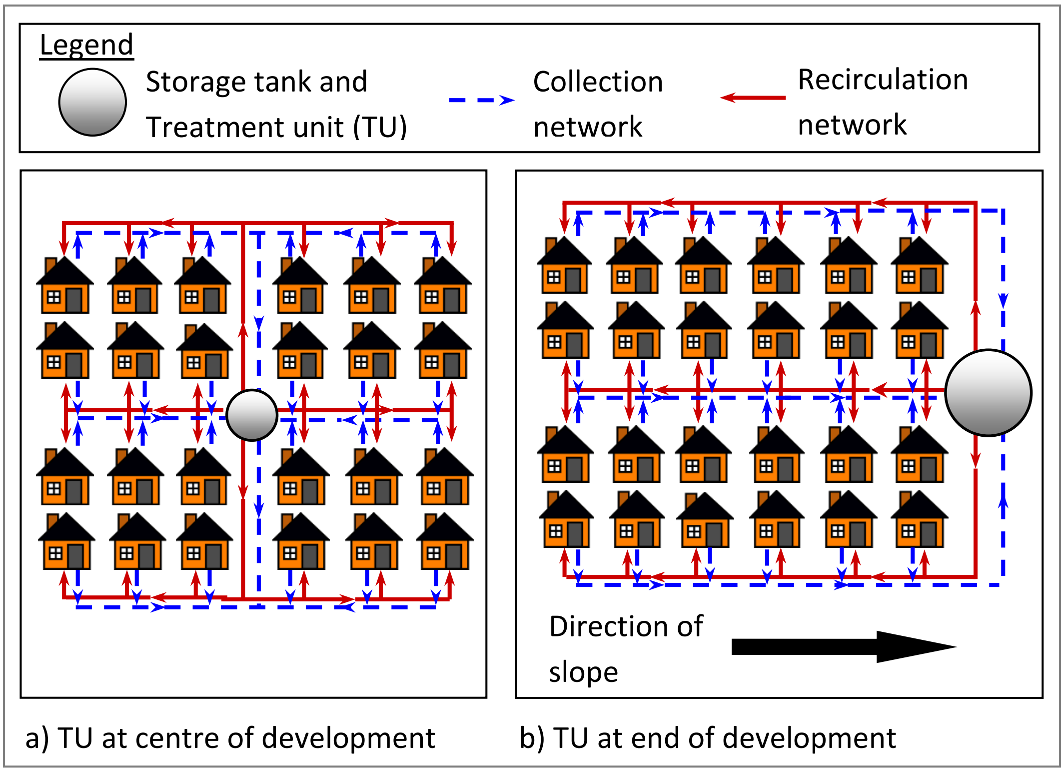


Fig. 2. Examples of layouts for communal rainwater systems.

5. Plan the layout of the rainwater collection and distribution systems for the various housing scale layouts similar to that shown in Fig. 2a and 2b.
6. Collect information on the rate of water supply for various end uses and decide on whether the application of rainwater will be for potable and/or non-potable uses.
7. Collect information on the local water supply system design guidelines and approaches. Estimate peak flows in the rainwater collection and distribution systems for each housing layout using local guidelines (See Sections 3.2.2 and 3.2.3).
8. For each layout, conceptually design the communal rainwater tank system using the following approach:

- Estimate the size of the rainwater tank with optimal volumetric reliability based on water balance approaches considering roof area connectivity, rainwater patterns and end uses (i.e. water demand/consumption estimates). Also explore the availability of an alternative water source to top-up the rainwater tanks as a supplementary source in case the water level in rainwater tank is low.
- Estimate rainwater flows from roof areas into the collection pipes.
- Estimate the flows in collection pipes as per the layout for sizing. Design the gravity flow collection system based on local design guidelines and topography. Consider if sump wells will be required. Sump wells are smaller sized underground tanks which serve as interim storage before the main storage tanks. If sump wells are not considered, the communal rainwater tanks will have to be placed underground which sometimes can be very deep and thus can be uneconomical to construct and operate (See Fig. 5).
- Check the depth of the invert level of the collection system at the communal tank location. If the depth is high, a sump cum pumping system for lifting the rainwater to the storage tank will be required. The designer can check for the trade-off between collection system cost at various gradients and the cost of cluster rainwater tanks if laid underground completely and associated pumping system.
- Design the rainwater distribution system based on local design guidelines.

9. Collect local cost data for pipes, pipe laying, rainwater tanks, sump wells, pumps and treatment units by contacting local rainwater tank suppliers, installers and plumber.

Also collect data on electricity charges and other operation and on-going maintenance costs (labour and chemicals).

10. Conduct a life cycle costing (LCC) for financial analysis of each housing layout for a selected analysis period using standard approaches (see Section 4.1). This should include capital, operation and maintenance and replacement costs over the analysis period. Estimate the LCC per household for each housing layout and the contribution of various infrastructure components.
11. Plot the graph of final costs (on a per household basis) against the total number of houses for the associated housing development layouts to estimate optimal scale of communal rainwater tank system for selected topography and housing density as demonstrated in Fig. 9.

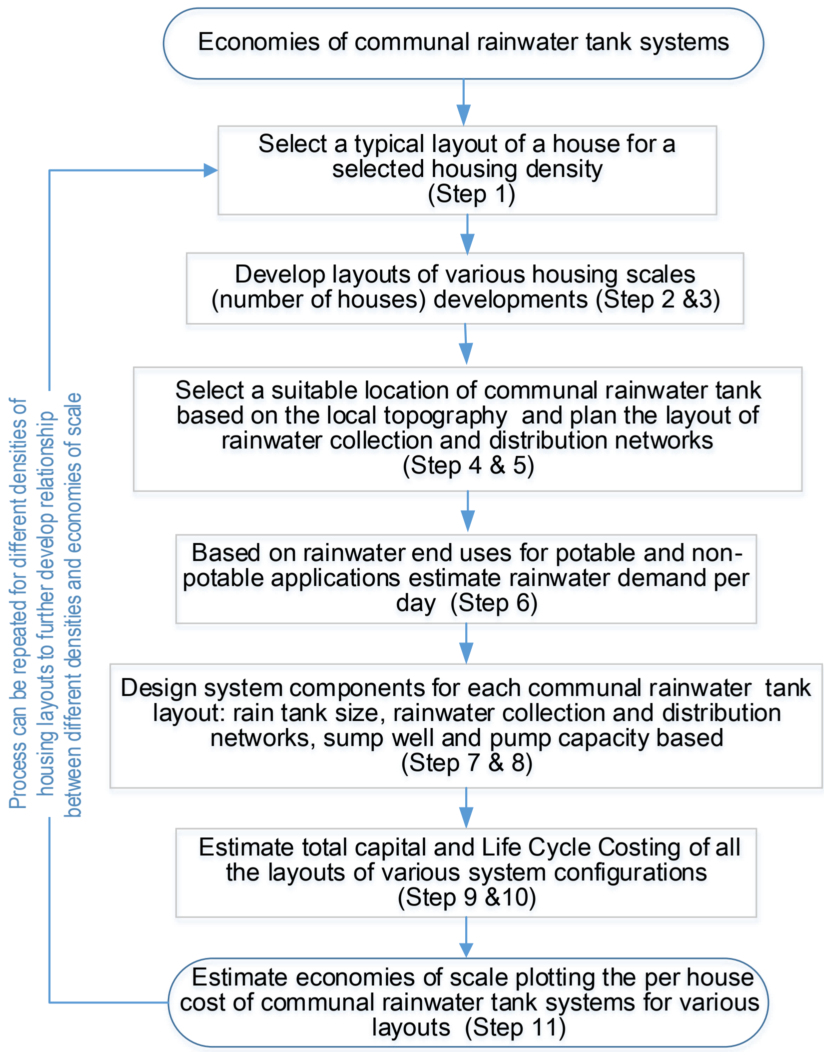


Fig. 3. Schematic of the methodology.

3 Application of proposed methodology

The methodology outlined has been applied to a hypothetical development area in South East Queensland, Australia. Cost values for individual components of the system are for December 2011 rates collected locally, when the study was carried out, and requires indexing using the outlined method in Section 4.1.1 to bring the prices to current rates. The financial analysis carried out in this study was carried out using Australian dollars ($).

3.1 Development of hypothetical housing layouts

The study considered a flat topography scenario, with the storage tank and treatment unit located in the centre of the development. Housing layouts for 4, 8, 16, 24, 48, 96, 192, 288, 384 and 576 homes were developed to assist in the financial analysis, with densities of these developments maintained at around 20 dwellings per hectare. Plotting final costs values, on a per household basis, against their respective housing layouts would in theory provide the optimal number of houses which the system would be most economical at. To develop housing layouts, Google Earth was used to measure new housing plots within new greenfield developments in SEQ. A representative lot size with dimensions of 16 m by 25 m (See Fig. 4) and road widths measuring 8 m was adopted in developing the various housing layouts. Fig. 4 shows an example of a typical housing layout for 48 homes utilised in this study. The rainwater tank and treatment plant were located in the centre to minimise the cost of collection and distribution networks.

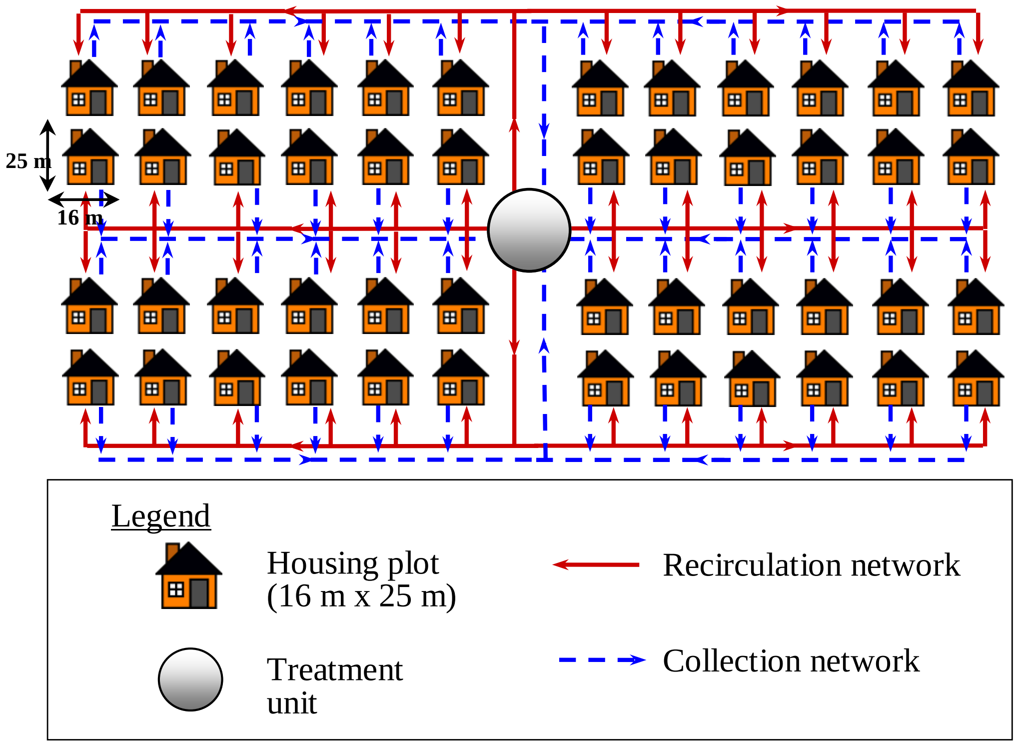


Fig. 4. Illustrative diagram of a layout for 48 households.

3.2 Design of communal system

There are no specified guidelines on the design of a communal RWH system for potable use in Australia. Guidance has been taken from various literature and sources, including an existing communal rainwater site in Capo di Monte (CDM) in SEQ (Cook et al., 2012). The primary function of the communal RWH system used in this study is to collect, transport, treat and recirculate rainwater back to the residential households for potable use (Fig. 5). A housing occupancy rate of 2.6 persons per household (OESR, 2012) was used to determine daily potable consumption and peak flow calculations. Basic design of pipes, tanks, pumps, and treatment units for the 10 different layouts of communal rainwater systems were conducted to obtain specific cost data.

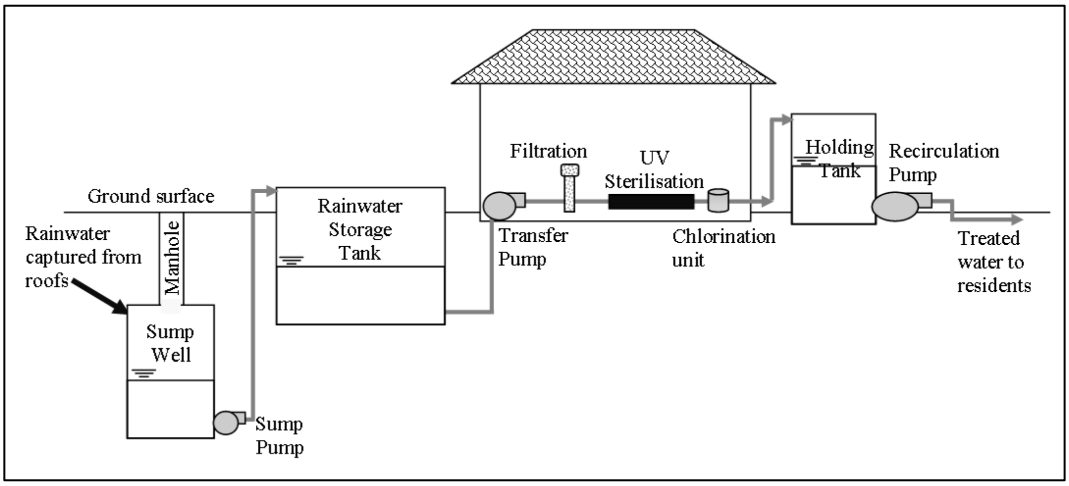
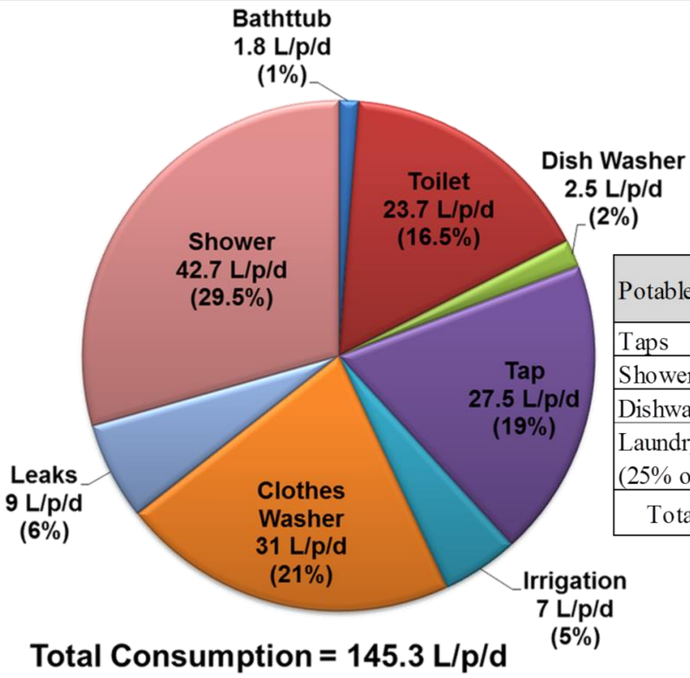


Fig. 5. Basic layout of rainwater treatment system – sump wells were incorporated where the depth of rainwater collection system is greater than 1.5 m.

3.2.1 Potable water usage

The communal rainwater tank utilized in this study will provide for appliances specifically requiring a potable source including shower, taps, dishwasher, laundry (hot water). Appliances such as laundry (cold water), toilet and irrigation which do not require potable water will be provided for by other sources (e.g. on site water recycling facility).

The volume of potable water end usage varies substantially between regions (Beal et al., 2010) and as such, potable consumption details are required at the local level for the design of the communal rainwater system. Estimates of average daily water consumption were based on the South East Queensland Resident End Use Study (SEQREUS) (Beal et al., 2010) and is summarised in Fig. 6.



Potable Appliance	Water Use (L/p/d)
Taps	27.5
Shower	44.5
Dishwasher	2.5
Laundry - hot water (25% of total laundry use)	7.8
Total potable demand =	82.3

Fig. 6. Nature of average daily per capita water consumption (L/p/d) for SEQ (Beal et al., 2010) and potable water consumption used in design of communal system.

The table in Fig. 6 also shows the potable water appliances used in the design of the communal system. Combined water consumption of shower and bath appliances was taken for the shower usage and hot water for laundry system taken as 25% of total water used for laundry (EBMUD, 2008).

3.2.2 *Peak flows in distribution system*

Estimation of peak water demand is important in the design of pumps and pipes for a water distribution network. Although design guidelines outlining peak flow factors are available, Swamee and Sharma (2008) stated that local information on flow requirements will provide a much better indication of peak flow selection. Hence, peak flow demands have been estimated from two studies of diurnal pattern analysis in SEQ. Smart water meter data from Umapathi et al. (2013) and Beal et al. (2010) containing peak flow rates per person were obtained and plotted against the respective 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 occupancies of 1 to 55 and $y = 2.1424x^{-0.203}$ ($R^2 = 0.9957$) for occupant numbers between 56 and 610, were used to calculate peak flows for the different household layouts. Beyond 610 people, the average of Beal et al. (2012) findings of 0.58 L/min/person was used as this was the maximum population analysed under this study and there is uncertainty as to what the trend may be beyond this population size.

3.2.3 *Collection pipes*

As is standard practice, the street layout of the model housing development was used in the design of the pipe network layout. Hydraulic analysis of the site at CDM showed that household connectors from the down pipes to the rainwater collection system limited flows into the system to 2 mm/5 min (Cook et al., 2012) and the remaining flows diverted to the stormwater collection system. Applying the Rational

Method (DERM, 2007), with roof area estimates of 220 m^2 (obtained from Google Earth) and 100% connectivity to the system, and a recommended runoff coefficient of 0.875 for a pitched roof (Cook et al., 2012), resulted in a maximum flow harvest of 0.00128 m^3/s. Gravity rainwater collection pipes, which convey roof waters to the storage tank, were sized using Manning's equation with polyvinyl chloride (PVC) as the chosen pipe material. Manning's roughness loss for the PVC material has a recommended value of 0.01 (Rossman, 2010). A slope of 0.5% was used so as to obtain a minimum pipe full velocity of >0.7 m/s according to the Queensland Urban Drainage Manual (QUDM) (DERM, 2007). A minimum cover of 0.6 m was used at the start to determine pipe depths for estimating costs. Swamee et al. (1987); and Sharma and Swamee (2008) provide direct methods for sizing collection systems.

3.2.4 Rainwater tanks

Water balance simulation, probabilistic methods and hydrological approach are some of the methods which have been adopted in the design and optimisation of a RWH system (Hashim et al., 2013; Imteaz et al., 2011; Imteaz et al., 2013). The water balance approach has been used to develop a range of computational software, such as Rainwater TANK (Jenkins, 2007), UVQ (Mitchell and Diaper, 2010) and RainTank (Vieritz et al., 2007), to model rainwater tank scenarios.

For the study, sizes of rainwater tanks were estimated using UVQ (Mitchell and Diaper, 2006), an urban water balance and contaminant balance analysis tool, and included input parameters such as rainfall data, roof areas, tank sizes and daily water demand. Rainfall data recorded over a 20 year period (1991 to 2010), at time intervals of 6 minutes were obtained from the Bureau of Meteorology (BOM) Brisbane Airport (Station ID: 040842) and censored to limit peak intensities of rainfall to 2.4 mm/6 min (equivalent to 2 mm/5 min). The restriction of the peak intensity was considered due to connectors, linking the down pipes to the collection pipes, which restrict the flow (Section 3.2.3) into the tanks and as such, would affect the sizing of the rainwater tanks.

Volumetric reliability is the ratio of the rainwater that the communal rainwater system is able to provide, against the total household water demand placed on the rainwater tank. A volumetric reliability of 94% was used to estimate tank sizes for the different household layouts. This value was chosen as, in addition to providing a high reliability to maintain constant water supply to householders, it also represented the approximate point at which the gain in the tank reliability becomes marginal against the increase in tank storage size. Hence, the size of a rainwater tank required by a household was computed to be 10 kL; this was multiplied by the total number of houses in a layout to estimate the size of a communal rainwater tank (e.g. 160 kL for 16 households layout).

3.2.5 Sump well and sump pump

Sump wells are smaller sized underground tanks which serve as a temporary storage for rainwater, before it is transferred to the main storage tanks. They are required to avoid deep excavations for installing the rainwater storage tanks, which would add a significant amount to the initial costs. In this study, sump wells were required for pipe outlets exceeding 1.5 m in depth. Sump well and pumps were sized using UVQ, with sump well volumes replacing storage volumes and pump rates replacing daily demand in the UVQ parameter fields. A trial and error method was used to ensure annual extracted volume from the sump well would match the required annual demand from the rainwater tank sizing. This method will allow spillage from the sump well to occur, as the extraction rate could at times be lower than the inflow rate to the sump well. In the event that the storage tank is filled up, water could also be stored in the sump well after the rainfall event.

3.2.6 Treatment unit

The treatment unit consisted of a transfer pump, which will pump rainwater from the storage tank to the holding tank through a sand and carbon filter, ultraviolet (UV) sterilisation and chlorination processes to ensure that the water was fit for potable use. The chlorine system will be made up of a tablet dispenser unit and easy to use chlorine tablets which requires no power to run.

Holding tanks were sized to store 3 days of potable water supply without the need for top up. Recirculation pipes of 100 mm were used according to guidelines for potable water mains from Gold Coast City Council Planning Scheme Policies (GCCC, 2008).

3.2.7 Rainwater distribution system

The water distribution network and pumping systems were designed using local water supply guidelines (GCCC, 2008) and hydraulic methods for distribution network layouts of various scales (Swamee and Sharma, 2008).

3.2.8 Pumps

Pumps were selected using the online Grundfos WebCaps application[1], which allows users to input flow and head requirements and recommends a range of pumps to choose from. Head losses for pumps were calculated using the Bernoulli's equation:

$$H_p = Z_L + H_L + H_T - Z_p \qquad (1)$$

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

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).

The head loss in pipes was calculated using the Darcy-Weisbach formula and the minimum pressure, or terminal head, at the property boundary for potable water was taken to be 22 m in accordance with Gold Coast City Council Planning Scheme Policies (GCCC, 2008).

3.2.9 Power consumption

Estimates of power consumptions were carried out to estimate the operating costs for the RWH system. Energy consuming equipment within the communal RWH system includes the various pumps and the UV system, with the former being the main consumer. The UV system operates when the transfer pump runs and its power was estimated from suppliers' guidance. A simple method of obtaining daily power consumption is simply multiplying the mains power consumption (P_1) with the daily operating hours (O_d). However, a study by Ward et al. (2011) notes that this tends to underestimate total power usage by 60% in pumps, due to the high energy consumed during the start-up phase. Hence, the factored pump energy consumption can be taken as:

$$E_{d.f} = \frac{P_1 \times O_d}{0.6} \tag{2}$$

Where: $E_{d.f}$ = factored daily energy consumption (Wh) per day, P_1 = mains power consumption (W), O_d = daily operating duration (hours).

4 Cost Estimation

4.1 Life cycle costing

Life cycle costing (LCC) is used as a financial 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). This approach involves all costs incurred over the analysis period, i.e. the initial outlay to commission the project (capital costs), the ongoing costs for the smooth operation and maintenance of a system and replacement costs for purchasing components at the end of their useful life.

The analysis is based on the net present value (NPV) method, a life cycle costing tool which is commonly used to determine current values of future investments. It is common practice to assume present and future costs of a component are the same due to uncertainties in predicting future costs. The capital cost NPV, P_{NC} of a component having a life cycle of n years can be expressed (Newnan et al., 2002) as:

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

Where P = Capital cost, i = discount rate and n = life of component.

In the case of a component having a life span less than the analysis period, it would require periodic replacing after its useful life and hence a modification of Equation 3. For instance, if a component has a life span of a quarter of the analysis period, the P_{NC} of such a component can be estimated as:

$$P_{NC} = P + P \cdot (1+i)^{-n/4} + P \cdot (1+i)^{-2n/4} + P \cdot (1+i)^{-3n/4} \tag{4}$$

For annual costs of expenditure (A), such as maintenance and operation costs,the net present value (P_{NA}) over n years can be estimated using the following equation (Newnan et al., 2002):

$$P_{NA} = A \cdot \left[\frac{(1+i)^n - 1}{i \cdot (1+i)^n} \right] \qquad (5)$$

An analysis period of 50 years, taken to be the estimated useful life cycle for a typical household, was used along with a discount rate of 7% recommended by the Australian Government's Best Practice Regulation Handbook (OBPR, 2007). Prices for the various components were obtained mainly through direct communication with service providers and other sources, detailed in Table 1.

Table 1. Main components and costs for the RWH system in this study.

Component	**Cost**	**Life (years)**	**Source**
<u>Capital Costs</u>			
Pipes (PVC) - indexed			
75 mm	$77.23/m	80	GCW (2008)
100 mm	$110.24/m	80	GCW (2008)
150 mm	$178.08/m	80	GCW (2008)
225 mm	$269.24/m	80	GCW (2008)
300 mm	$355.10/m	80	GCW (2008)
375 mm	$472.75/m	80	GCW (2008)
450 mm	$871.31/m	80	GCW (2008)
Pumps			
Recirculation Pumps	$3,380 - $8,420/unit	12	Supplier
Transfer Pumps	$1,070 - $1,185/unit	12	Supplier
Sump Pumps	$1,575 - $2,055/unit	12	Supplier
Additional Pump Equipment (e.g. lockable main isolator, circuit breakers, folded base, isolation valves)	$3,685 - $5,945/unit	50	Supplier
Installation	$1,650/set	-	Supplier
Commissioning	$660/set	-	Supplier

Tanks			
Concrete tanks; storage and larger sized holding tanks, sump wells	$717/m^3$	50	Supplier
Plastic tanks; smaller sized holding tanks	$1,177 - $4,046/unit	25	Supplier
Other Capital Costs			
Excavation (indexed)	$96.35/m^3$	-	Rawlinson Group (2011)
Manholes (indexed)	$5,550 - $14,020/unit	50	GCW (2008)
Treatment system housing (indexed)	$39,032/unit	50	CDM manager
Sand, carbon filter and UV	$2,500/unit	12	Supplier
Chlorine doser (tablet system)	$1,500/unit	12	Supplier
Ongoing Costs			
Maintenance			
Sediment checks and cleaning (every 3 years)	$370 - $6,500/3 years	-	Plumber
Gutters	$40/house/annum	-	Plumber
Pumps	$350/pump/annum	-	Plumber
Filters + UV lamps	$300/annum	-	Supplier
Chlorine doser	$200/annum	-	Supplier
Operation			
Electricity for pumps (transfer, recirculation and sump), UV system	$0.2276/kWh	-	Energy suppliers

4.1.1 Indexed Rates

Costs for pipes, excavations, manholes and treatment units housing were required to be scaled up to current levels (December 2011). The Producer Price Index, from the Australian Bureau of Statistics (ABS), was applied, with Index Number 3011, *"House construction Queensland"*, under *"Table 15"*, *"Selected output of division E construction, subdivision and class index numbers"*, (ABS, 2012) chosen as appropriate for this purpose. The formula used to update prices is as follows:

$$\%\ change\ in\ price = \frac{Current\ Year\ Index - Base\ Year\ Index}{Base\ Year\ Index} \times 100\% \qquad (6)$$

4.1.2 Pipe Cost Multiplication Factors

Pipe costs differ with varying depths and length. Gold Coast Water (2008) recommended pipe cost multiplication factors for short length pipes (<200 m) and pipes at varying depths as shown in the following table.

Table 2. Cost factors for varying depths of pipes (GCW, 2008).

Pipe Length	**Length Factor**	**Depths**	**Pipe diameter <300 mm**	**Pipe diameter ≥300 mm**
0 – 50 m	2.0	Up to 1.50 m	1.00	1.00
51 – 100 m	1.7	1.5m to <3.0 m	1.19	1.25
101 – 200 m	1.5	3.0m to <4.5 m	1.34	1.40
> 200 m	1.0	> 4.5 m	1.47	1.54

5 Results and Discussion

5.1 Cost contributions

The cost contributions were estimated for a communal rainwater tank system for various development layouts, starting from a group of 4 homes and up to 576 homes. The layout for 48 homes is shown in Fig. 4. The NPV method of cost analysis was used to obtain the individual household costing of a communal rainwater system. The life cycle cost was estimated considering capital, operation, maintenance and replacement costs.

Capital costs were divided into two main categories; one for laying the pipe network and another for setting up the storage and treatment units including auxiliary systems (storage tanks, holding tanks and pumps). Operation costs mainly consisted of the costs of power for running the pumps and the UV units. Maintenance costs were required to maintain the system to operate at optimal capacity and included sediment checking and cleaning of the tanks and sump wells, gutter clearance, changing the filters and UV lamps, supply of chlorine dosage and maintenance of the pumps. Replacement cost is the cost of replacing a component at the end of their useful life span and is estimated separately to original capital costs. Costs were included for components with useful life shorter than the analysis period such as holding tanks, pumps, sand and carbon filter with UV including chlorine doser.

Capital costs represent the largest proportion of total life cycle costs (>70%) in a communal system followed by maintenance (10% - 15%), operation (3% - 5%) and replacement costs (1% - 10%) as shown in Fig. 7.

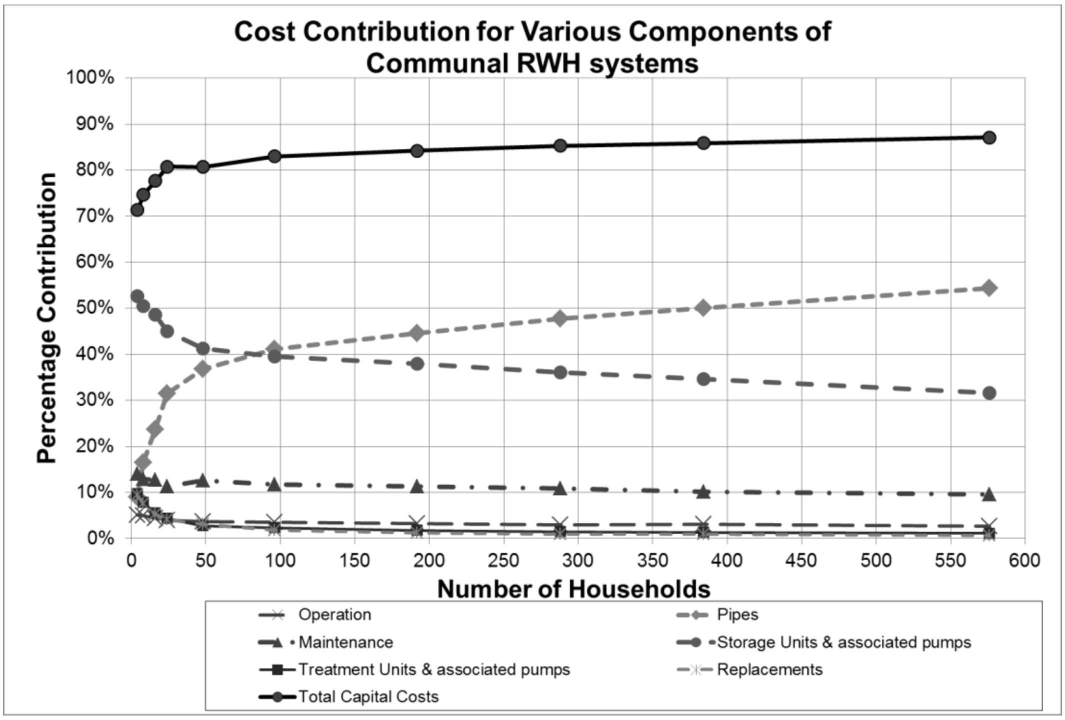


Fig. 7. Cost contributions for various components of communal RWH systems per household.

5.2 Capital Costs

Fig. 7 showed the two categories of capital cost components (pipes plus storage and treatment units) being the dominating factors in influencing final NPV for the majority of housing layouts, with cost distribution for both components not dropping below 30% beyond the 16 households scale. The cost weightages for both storage and treatment systems are observed to decrease with increasing household layout, whilst contributions from pipes are noted to rise. This can be attributed to increasing pipe laying costs, along with a decrease in the costs of storage and treatment units as shown in Fig. 8.

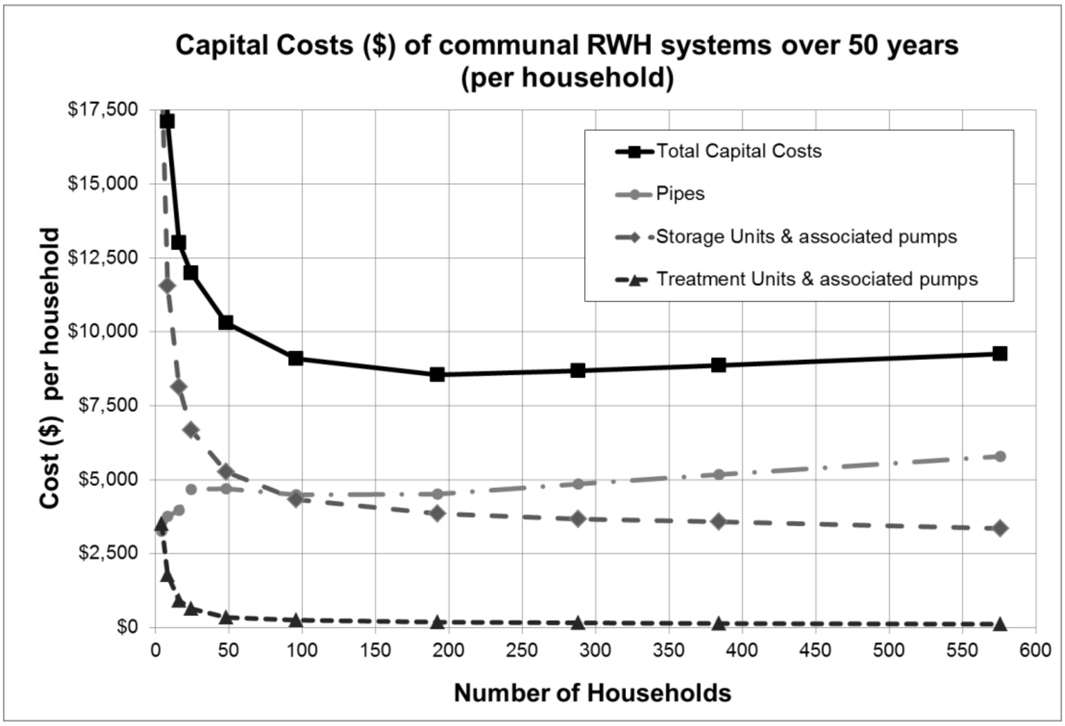


Fig. 8. Breakdown of capital costs per household for a communal RWH system.

Progressively longer, deeper and larger pipes for increasingly larger developments results in a diseconomy of scale, shown in Fig. 8 and highlighted by Clark (1997). Storage tanks and treatment units on the other hand are shown to have an economy of scale, with bigger sized tanks decreasing in costs on a per kL basis that is further compounded by minimal price changes in other costs (filter, UV, chlorination and pumps). Fig. 8 also shows that for lower number of households, below 96 dwellings, capital costs are influenced by the costs of the storage units and treatment units, whilst beyond this household number, the pipe costs dominate. This observation is in agreement with Booker (1999) and Clark (1997), both of whom state that treatment costs are the dominant factor for lower scale of connections with the diseconomy of pipes affecting higher scales. Although both studies by Booker (1999) and Clark (1997) were carried out for greywater and wastewater respectively, the results from this study showed similar observations for communal rainwater systems.

5.3 Life cycle cost per household

Life cycle costing of a communal rainwater harvesting system on an individual household basis is plotted in Fig. 9 and shows an initial drop in the costs, falling sharply from the 4 homes to 96 homes layout before flattening off between 192 and 288 homes. Booker (1999) and Clark's (1997) studies also noted similar trends where the curve flattens off and only small cost differences occur in this household range. The minimum cost of $10,150 per household is observed to be in the 192 households' layout. However, the difference in per household cost between layouts of 192 and 288 households is very low (Fig. 9). Beyond the 288 homes layout, the costs start increasing, with the rise clearer after 288 households as a result of the diseconomy of scale of pipe costs exerting a larger influence on the total life cycle cost of the system.

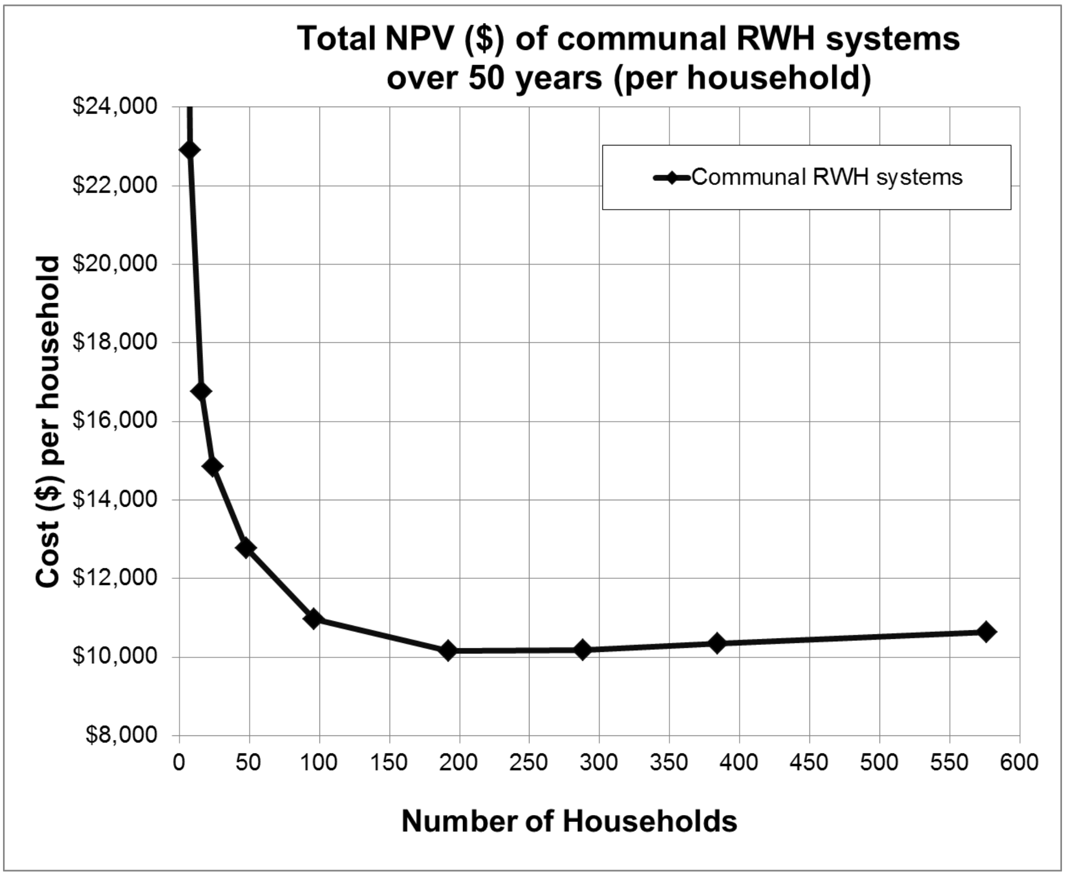


Fig. 9. Total NPV costs per household for communal RWH systems.

Fig. 9 further shows the influence capital costs have over the final individual household NPV with both graphs (Fig. 8 and Fig. 9) following a similar trend; an initial sharp drop until 96 households before smoothing off to reach its lowest level at 192 households and rising after this. This capital costs trend, when broken down further, indicates that including and below 96 households, the costs of storage and treatment units dominate, whilst beyond this number, pipes are the main costs drivers. Although ongoing costs are included in the final NPV, their low contribution of 13% - 29% and continuing economy of scale trend proves the minimal influence they have over final cost values.

5.4 Sensitivity analysis

The Office of Best Practice Regulations, Australia (OBPR, 2007) recommends carrying out a sensitivity analysis on discount rates due to the uncertainty over the rates used. Rates of 3% and 11% are recommended altering maintenance, operational and replacement costs in the process. Results of the sensitivity analysis conducted in this study for all households analysed are shown in Table 3 in comparison to the analysis presented previously, which was based on a 7% discount rate.

The results of the sensitivity testing showed that the lower discount rate of 3% affects final results more than the higher rate of 11%. Applying a lower discount rate of 3% resulted in maintenance and operational costs rising by more than 85% while the higher rate resulted in a decrease in costs of 35%. Furthermore, replacement costs rose by more than 130% of origin values for the lower discount rate, whilst dropping by 50% at the 11% discount rate. This is due to the inverse relationship between NPV and discount rate, whereby when the discount rate is raised, the NPV falls and vice versa.

Table 3. Results of sensitivity analysis for discount rates based on OBPR (2007) recommendations (Units = percentage change in costs)

House Nos.	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%

The impact of discount rates was also assessed on the optimal scale of households. Fig. 10 shows the optimal scale from the sensitivity analysis occurring between 192 and 288 households. This indicates that although discount rates affect final costs, the occurrence of the optimal housing layout remain relatively unchanged. Clark (1997) also noted a change in overall costs but no difference on the scale of servicing when carrying out sensitivity tests on the discount rate. The results from the sensitivity testing indicate that overall NPV are bound to be higher or lower than obtained prices depending on the discount rate used. The optimal number of households per development is not influenced by the market rates of interest and inflation, which is the source of uncertainty for the discount rates. This also highlights the importance of initial costs when considering the optimal housing range, as the sensitivity analysis has shown that discount rates have minimal effect on this housing number.

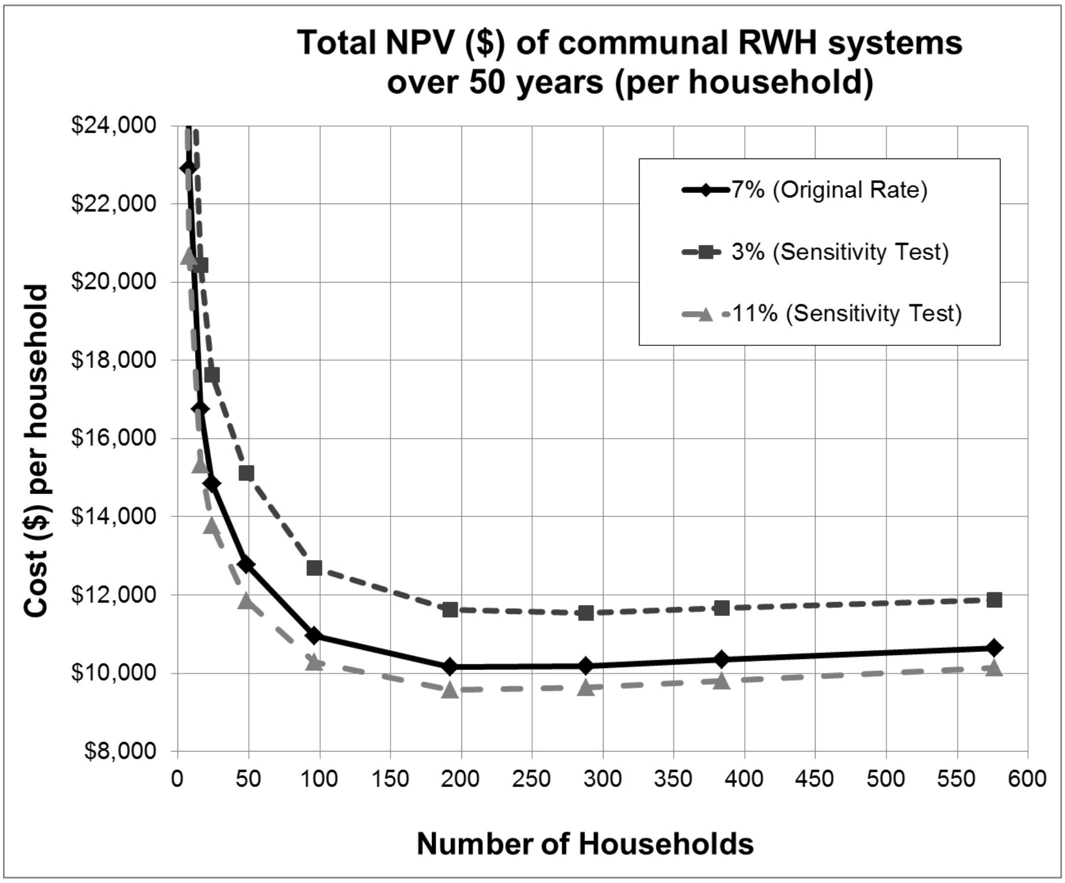


Fig. 10. Results for sensitivity analysis on discount rates.

5.5 Influence of land topography

A desktop analysis was carried out to determine the effects of a sloped topography on the final NPV using the optimal household configuration as a baseline. This investigation assumes that the slope of the ground for this scenario is the same as that used for the initial gradient of the pipes in the analysis; i.e. 0.5%. Cost data for the sump system and depth factors for pipes were not taken into consideration as the excavation depths would be at economical levels due to the sloped topography. However, there would be a slight increase in the cost of distribution due to the increased pumping energy required. In this analysis, collection and recirculation pipes was assumed to be laid in parallel in a single trench which would result in a 16% reduction in the cost of the pipe network (Booker, 1999).

Table 4. Cost differences between flat and sloped topography.

	Flat Topo. (Original)	Sloped Topo. (Adjusted)	% of original cost
Capital Costs			
Pipes	$4,522	$3,654	81%
Storage Units	$3,849	$3,466	90%
Treatment Units	$177	$177	100%
Total Capital Costs	$8,548	$7,297	85%
Ongoing Costs			
Maintenance	$1,149	$1,080	94%
Operation	$325	$291	90%
Replacements	$128	$102	80%
Overall NPV	$10,150	$8,770	86%

Results are presented in Table 4 and showed overall reductions in all cost categories, with pipe costs dropping by almost 20% due to shallower depths and the use of a dual pipe system. Removal of the sump well and sump pump resulted in a 10%

drop in the storage units' costs, as well as a reduction in ongoing costs of 80% to 94% of the original values. Treatment unit costs are unaffected as the system still supplies and treat the same amount of water as a flat topography system, hence no changes to its design are required. Overall costs (NPV) are shown to drop by approximately 13%.

This simple analysis has shown that the topography of the land can reduce the NPV by significantly reducing initial costs, which have proven to be a major contributor to the overall life cycle costs of communal rainwater systems. However, this would be based on an ideal situation, where the slope of the land follows that of the pipeline. In a different topographic layout, there may be a need for additional equipment, for example a pumping station to raise water to economical trenching depths, which would result in increased, rather than decreased costs.

6 Conclusions

The economies of scale for a communal rainwater harvesting system have been assessed, as well as an assessment on the influence of cost components. Housing layouts containing 4 to 576 homes were designed to assist in obtaining cost data for the associated components of the system. Results showed that capital costs made up the largest contribution (>70%) to the system followed by maintenance, operation and replacement costs.

Households of more than 96 homes highlighted the diseconomy of scale prevalent in most pipe networks, whilst lower scale dwellings were shown to be more affected by storage and treatment unit costs. Optimal development sizes occurred between 192 and 288 households, with minimum NPV costs calculated to be $10,150 per household. Sensitivity analysis of discount rates demonstrated that the optimal numbers of households is not necessarily influenced by this parameter, and hence market rates of interest and inflation, which is the source of uncertainty for the discount rates.

Topography of the surrounding land was shown to exert a significant influence on the NPV for communal systems. The study showed total costs dropped by 13% for a layout with a slope of 0.5%. However, in varying scenarios, the final NPV may rise as there may be a need for further system components.

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