Performance of Cluster Scale Rainwater Harvesting Systems: Analysis of Residential and Commercial Development Case Studies

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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

In South East Queensland (SEQ), pressures on the urban water system due to an extended period of drought and increasing demand from population growth have prompted the development of strategies to reduce demand and diversify water supply sources. Rainwater tanks are part of integrated urban water management approach that considers the whole water cycle to provide water services on a fit for purpose basis that minimises the impact on the local environment and receiving waters. Rainwater tanks in urban areas are typically applied at the individual household scale for non-potable water source uses such as toilet flushing and garden irrigation. However, this report has explored alternative configurations of rainwater harvesting, in particular: communal rainwater harvesting for potable supply in a multi unit residential development; and rainwater harvesting for non-potable uses in a high-rise commercial development.

Rainwater tanks were identified in the South East Queensland Water Strategy as a key element in securing sustainable levels of urban water services for SEQ (Queensland Government, 2010). All new detached dwellings are now mandated to achieve 70 kL/hh/year reduction in demand from grid water supplies; for townhouses, this self sufficiency requirement is 42 kL/hh/year. Mandatory Part (MP) 4.2 of the Queensland Development Code (QDC) stipulates that internally plumbed rainwater tanks are one option to achieve these water saving targets. Other options include communal rainwater tanks, stormwater harvesting, dual reticulation recycled water schemes and the reuse of suitably treated greywater.

QDC MP 4.3 also specifies performance criteria and acceptable solutions for alternative water sources in commercial buildings. MP 4.3 specifies that commercial buildings supplied with mains drinking water must provide an alternative water source for suitable uses. The suitable measures identified are a rainwater tank, water storage tank, common tank, or a greywater treatment system. To fulfil the requirements of MP 4.3, rainwater systems need to be connected to any swimming pool on the lot, any external use, each pedestal (toilet), washing machine cold water taps, and any other fixtures specified by the local government planning provisions. The QDC requires that rainwater systems must have a minimum storage of 1500 L per connected toilet, and a connected roof area of 50 m^2 for each connected toilet, or the available roof area, whichever is the lesser. This research project was initiated to understand the feasibility of achieving QDC water savings targets using communal rainwater tanks and identifying any energy penalties associated with such systems. It also sought to understand the practicality of reducing mains water demand in high-rise commercial buildings through rainwater harvesting.

Two case study developments with communal rainwater systems were selected for monitoring and validation. Capo di Monte (CDM) is a 46 dwelling residential development at Mt Tamborine in the Gold Coast hinterland. CDM is self sufficient for water and wastewater services. Potable water demand (drinking, cooking, bathroom and laundry) is met by rainwater harvested from roof runoff, which is treated and stored communally, and then reticulated to residences. Top-up is available from a bore. Recycled wastewater is used to meet non-potable demand (toilet flushing and irrigation).

The other case study is Green Square North Tower (GSNT), which is a 12 storey commercial building in Brisbane's central business district that was certified under the (voluntary) Green Building Council of Australia's environmental rating system as 6 Green Stars. Roof runoff is directed to an 80 kL basement storage tank, before being pumped to two smaller roof-top tanks (40 kL and 27 kL) which, in turn, are gravity fed for toilet flushing and landscape irrigation respectively. Roof runoff is supplemented by cooling tower blowdown water, fire-fighting test water and air conditioner condensate.

For each case study, an extensive monitoring system was set up to capture energy and flow data from the rainwater systems. Manual meters were used to validate electronic data-logging. We also undertook water balance and hydraulic modelling to complement field measurements that explored the performance of communal rainwater systems under different operating assumptions.

The report also compared runoff results from a calibrated hydraulic model with estimates using a rational runoff equation. This showed that, whilst the calibrated hydraulic model was more accurate, the computationally simpler rational runoff equation gave a reasonable approximation of roof runoff and capture and hence may be used for feasibility assessment of communal rainwater systems.

The results for CDM showed the communal rainwater system provided a reliable potable water source, but with an energy penalty that could be minimised by more appropriate pump sizing. Analysis demonstrated that a smaller pump size (kW) would reduce both electricity demand and greenhouse gas emissions. Communal rainwater systems offer a number of advantages over other alternative water sources at the development scale (e.g. stormwater, or recycled water) as roof runoff is a relatively high quality water source, which can be used directly for non-potable uses, or used for potable purposes after appropriate filtration and disinfection. The results reported have demonstrated that a communal rainwater system can reliably provide an alternative water source, with minimal reliance on back-up supply, but that there is the need to optimise the energy efficiency. In addition, a communal approach means that individual householders do not have to maintain and operate their own tank and treatment system. The communal system at CDM is managed by the body corporate. This means that the management burden is not imposed on each household, which may lack the skills or motivation to maintain its rainwater system correctly. Also, communal rainwater systems may be more appropriate in medium density developments where there is a high building footprint to allotment area ratio, which limits space available for storage tanks.

The GSNT monitoring results demonstrated that the system has provided moderate reliability with minimal energy requirements, but with the system yield constrained by system faults. The modelling showed that effective roof area was the constraining factor in improving the reliability of the rainwater supply for toilet flushing. High-rise commercial buildings have a low ratio of available roof area for each potential connected toilet. This illustrates the need to consider other potential sources of non-portable water that can augment rainwater supply. Our investigation of GSNT found that in hot, humid environments, condensate from air conditioner cooling systems could contribute a significant proportion of the non-potable demand. Groundwater infiltration was identified as another potential water source in GSNT, where car parks are underground and dewatering systems are installed to reduce groundwater pressure on the walls and floor. The feasibility of this option needs to be considered on a site specific basis, as the quality and potential yield is dependent upon the depth to the groundwater table, aquifer characteristics, and the interaction with building footings. It is suggested that ongoing monitoring of decentralised water systems as part of the building management system in commercial buildings would enable the early identification and fixing of faults, such as the problem with the pump pressure switch was found, through monitoring, to be limiting rainwater yield at GSNT. Increased monitoring could improve reliability and yield from rainwater systems by informing an appropriate schedule of maintenance tasks.

1. INTRODUCTION

1.1. Overview and Research Questions

The purpose of this research was to determine the potential for cluster scale rainwater harvesting to augment water supply and reduce demand for mains water in South East Queensland (SEQ). Rainwater tanks have been identified in the South East Queensland Water Strategy as a key element in securing sustainable levels of urban water services for SEQ (Queensland Government, 2010). All new detached dwellings are now mandated to achieve 70 kL/hh/year reduction in demand for grid water supplies; for townhouses the requirement is 42 kL/hh/year (Queensland Department of Infrastructure and Planning, 2008). Mandatory Part (MP) 4.2 of the Queensland Development Code (QDC) stipulates that internally plumbed rainwater tanks are one option to achieve the water saving target. Other options include: communal rainwater tanks, stormwater harvesting, dual reticulation recycled water schemes, and the treatment and reuse of greywater.

The QDC MP 4.3 also specifies performance criteria and acceptable solutions for alternative water sources in commercial buildings. MP 4.3 specifies that commercial buildings supplied with mains drinking water must provide an alternative water source for suitable uses. The suitable measures identified are: a rainwater tank, water storage tank, common tank, or a greywater treatment system. This research project was initiated to understand the feasibility of achieving water savings targets specified under the QDC through the use of communal rainwater tanks and identify any energy penalties associated with these systems.

Two case study developments with communal rainwater systems were selected for monitoring and validation: Capo di Monte and Green Square North Tower. Capo di Monte (CDM) is a 46 dwelling residential development at Mt Tamborine in the Gold Coast hinterland, south of Brisbane, which utilises a communal rainwater system to meet potable demand. The other case study was Green Square North Tower (GSNT), which is a 12 storey commercial building in Brisbane's central business district that was certified as a 6 Green Star building under the voluntary Green Building Council of Australia's environmental rating system. Sustainability initiatives at GSNT include a rainwater system that harvested roof runoff for toilet flushing.

The research approach was designed to answer the following questions:

1. Do communal rainwater tanks deliver the mains water savings as mandated in QDC MP4.2 and 4.3?
2. Is there an energy penalty for using a communal rainwater tank?
3. Is the measured performance of communal rainwater systems comparable to their design performance?
4. How can cluster scale rainwater systems be configured to maximise water yield (and hence mains water savings) and minimise energy demand?

1.2. Background

In many developed countries, such as Australia, the sustainability of the traditional method for the supply and use of water for urban applications has become a critical issue for governments. In particular, the capacity of surface and groundwater sources to reliably meet future demand in Australian cities has been questioned. There has been an extended period of drought for much of the last decade that meant severe restrictions were placed on water uses to ensure essential water use demands could be satisfied. Uncertainty in future reliability of traditional water supply sources is compounded not only by inherent climate variability, but also by the rapid rate of urbanisation that is driving growth in demand, and the projected impacts of climate change on inflows to drinking water reservoirs (Marlow *et al.*, 2010, Moglia *et al.*, 2011, Ruth *et al.*, 2007, Sharma *et al.*, 2012). This situation has required decision makers and planners to reassess the paradigm where urban water demand has been largely supplied by high quality drinking water regardless of the quality requirements of the intended end uses such as toilet flushing. The response to these pressures on traditional urban water sources has included demand side measures, such as water conservation and efficiency, and supply side measures through the diversification of water supply sources.

These pressures on the urban water system have been experienced acutely in SEQ in recent years, with a severe drought and growing demand reflected by an additional 745,000 new households projected for the region by 2031 (SEQ Regional Plan). The Queensland Government responded to the looming water supply deficit with a comprehensive regional water strategy - *South East Queensland Water Strategy* (Queensland Water Commission, 2010). Reduced mains water demand through local rainwater harvesting was one of the approaches promoted under this strategy to provide long term water security.

Rainwater harvesting for urban water supply at a local scale has been around since the beginning of urbanised society (AbdelKhaleq and Alhaj Ahmed, 2007). However, the integration of rainwater tanks as a mainstream practice in modern cities to complement conventional reticulated water supply sources is novel. There are a range of simulation studies that have explored the likely performance of rainwater tanks as an alternative supply source (Coombes and Kuczera, 2003; Imteaz *et al*., 2011; Jones and Hunt, 2010; Khastagir and Jayasuriya, 2010; Mitchell, 2008; Vialle *et al*., 2011). However, there is the need for experimental monitoring studies to quantify rainwater harvesting in a range of development settings. Rainwater harvesting can also offer benefits to other parts of the water cycle, including: moderating peak stormwater runoff; reducing discharge of nutrients to receiving waters; and improving stream ecological health (Hall *et al*., 2011, Khastagir and Jayasuriya, 2010; Kim and Furumai, 2012; Villarreal and Dixon, 2005; Farreny *et al*., 2011).

However, in understanding the potential role of rainwater harvesting in complementing conventional centralised water services, it is important to separate sustainability aspirations from biophysical fact if we are to move forward with confidence in arguing for a re-engineering of the water cycle of future urban developments. This includes investigating and evaluating the actual performance of rainwater systems at different spatial scales and different development contexts.

The monitoring and modelling results presented in this report provide a knowledge-base for planners, developers and other users when considering the feasibility of cluster scale rainwater harvesting for residential developments, and rainwater harvesting for non-potable uses in high-rise commercial buildings.

2. CAPO DI MONTE – RESIDENTIAL CLUSTER SCALE RAINWATER HARVESTING FOR POTABLE USE

2.1. Capo di Monte Overview

Capo di Monte (CDM) is a 46 home development at Mount Tambourine located in the Gold Coast hinterland. The development site lies outside of the area serviced by municipal water and wastewater services, so for the development to proceed it had to commission decentralised water and wastewater systems. The communal rainwater system was designed to meet household uses that require potable water quality: kitchen, bathroom and laundry. A wastewater recycling scheme was used to satisfy non-potable demands: toilet flushing and garden irrigation. A local bore is used to supplement both systems in times when demand is higher than supply. The following sections describe: the communal rainwater harvesting scheme in more detail; the monitoring program that was designed to validate the performance of this system in terms of reliability of rainwater yield in meeting potable demand; and the energy required to provide this service. The section concludes by examining the performance of the systems against mains water savings targets specified under the QDC MP 4.2. It also discusses the broader applicability of communal rainwater harvesting in SEQ for reducing mains water demand.

The CDM development can be considered as townhouses; therefore, the applicable target for mains water reduction, under the QDC MP 4.2, is 42 kL per household a year. The use of communal rainwater tanks in meeting MP 4.2 is assessed by the relevant local government on a case by case basis. The QDC specifies that the communal rainwater system must be configured to provide sufficient storage capacity to meet water saving targets considering: local rainfall pattern, roof catchment area, and area available to accommodate rainwater tank. An acceptable solution under the QDC for attached Class 1 dwellings is to have at least 3,000 litres of rainwater storage connected to at least half the roof area, or 100 m^2, whichever is the lesser. Where a communal rainwater system is connected to meet internal household uses, there must be provisions for continuous supply to those uses, such as a trickle top-up supply from mains water. Note that CDM is not liable to meet these QDC regulations as it resides outside the mains water supply area. However, it is a very good bio physical analogy of an equivalent development that occurs within an urban area. Hence, the communal rainwater results from CDM should be directly transferable to townhouses in a traditional urban setting.

Local government approval of communal rainwater tanks also considers the roles and responsibilities of each property owner regarding water quality, system maintenance and ownership of the system. In most cases, the body corporate is assigned responsibility for the communal tank(s), water treatment, pipe work and intended use of supplied water. The body corporate at CDM has similar responsibilities.

2.1.1. Demographics

CDM was planned as a retirement village, therefore the majority of residents are over 60 years old, no longer working, and living in one or two person households. The overall population of CDM is 75 people, with an average household size of 1.65 persons compared to the average household size in the Gold Coast Statistical Division of 2.9 persons per dwelling. The smaller household size and older age of occupants at CDM is expected to impact on the water demand profiles we observed.

2.1.2. Climate

CDM is located in the Gold Coast hinterland, at an elevation of around 500 metres, and is subject to a subtropical climate. The elevated location influences a climate that is cooler than the eastern lowlands, with average maximum temperature during the winter of around 18°C and a summer average maximum temperature of around 26°C. Analysis of historical climate data shows that the annual rainfall over a 24-year period at Mount Tambourine was 1,318 mm. There was a pattern of relatively wet years, around 2,000 mm, interspersed with drier years, with rainfall of around 1,000 mm (Figure 1).

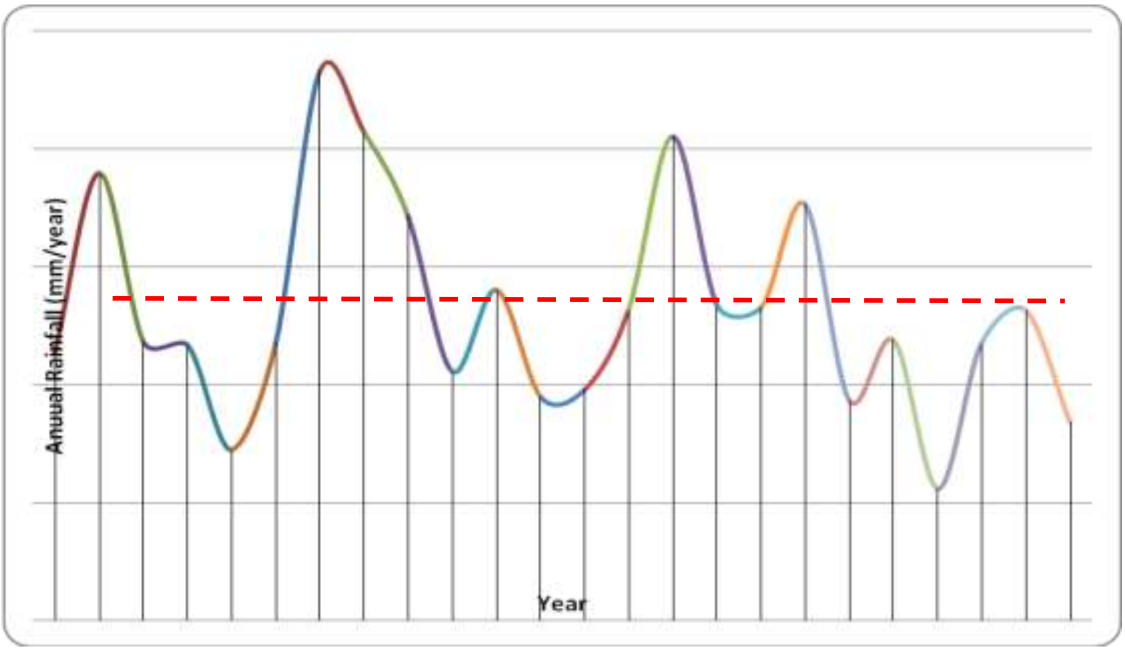

Figure 1: Annual rainfall - Mount Tambourine.

2.1.3. Communal Rainwater System

The communal rainwater system collects rainfall from the roofs of the 46 residential units through a network of household downpipes that feed into collector pipes which transfer the water by gravity to two 200 kL storage tanks. The total connected roof area is around 10,700 m^2, with houses having an average roof area of 222 m^2, and a community centre providing another 488 m^2 of connected roof area. CDM is situated on a slope, with an elevation change of around 25 metres, which represents a slope of 4%. The storage tanks and treatment system are located on the lower slopes of the site, which means gravity can be effectively used to transfer roof run-off. Figure 2 shows the layout of lots at CDM, with connected roof areas numbered. The location of the storage tanks and treatment system is marked by the red circle.

Figure 2: Layout of Capo di Monte.

A water treatment plant, comprising sand filtration, UV sterilisation and chlorination, pumps water to a 20 kL balance tank for subsequent distribution of *potable water* to each house and the community centre. A local bore supplements supplies in times of insufficient rainfall, or excess demand. This top-up water also passes through the water treatment plant. The communal 200 kL tanks at CDM are operated to retain at least 50% capacity to allow for an adequate fire-fighting supply, as required by state regulations. The system is managed by an appropriately trained person who is directly responsible to the body corporate entity. Figure 2 shows the CDM potable water hydraulic circuit.

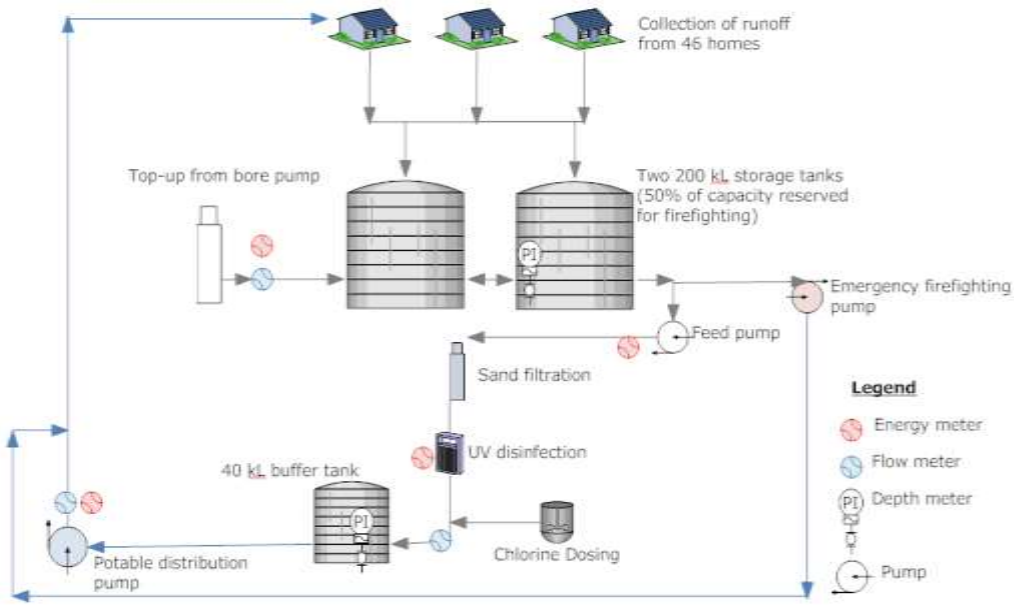

Figure 3: The potable water hydraulic circuit at Capo di Monte and water and energy meters installed for monitoring for this project.

2.1.4. Monitoring System

A monitoring system was set up to validate the performance of the CDM communal rainwater system. Monitoring of energy and water flows through the communal rainwater system was undertaken using a high-frequency logging device that recorded flows or energy pulses at a 5-minute time interval. The data logging system (Campbell Scientific CR 1000) stored the data in 5-minute, hourly and daily data files. Manual recordings taken monthly from the water and energy meters and height sensors were used to cross check the electronically logged data. This cross checking showed that the electronic logging in some cases was not corresponding sufficiently closely with manual readings, so these issues were investigated and resolved. Problems were associated with calibration of the pressure transducer, which was replaced. Appendix A has the technical specifications of the meters used.

All of the meters were connected to a data capture system that consisted of a multichannel hub that had suitable in-channel data connectors to accept all of the inputs and adapt them to RJ45 connectors. The hub was routed to a PC terminal which was connected to a broadband modem that allowed remote access to the data via a hosted web page using a file transfer protocol service. A schematic of the data collection system is shown in Figure 4.

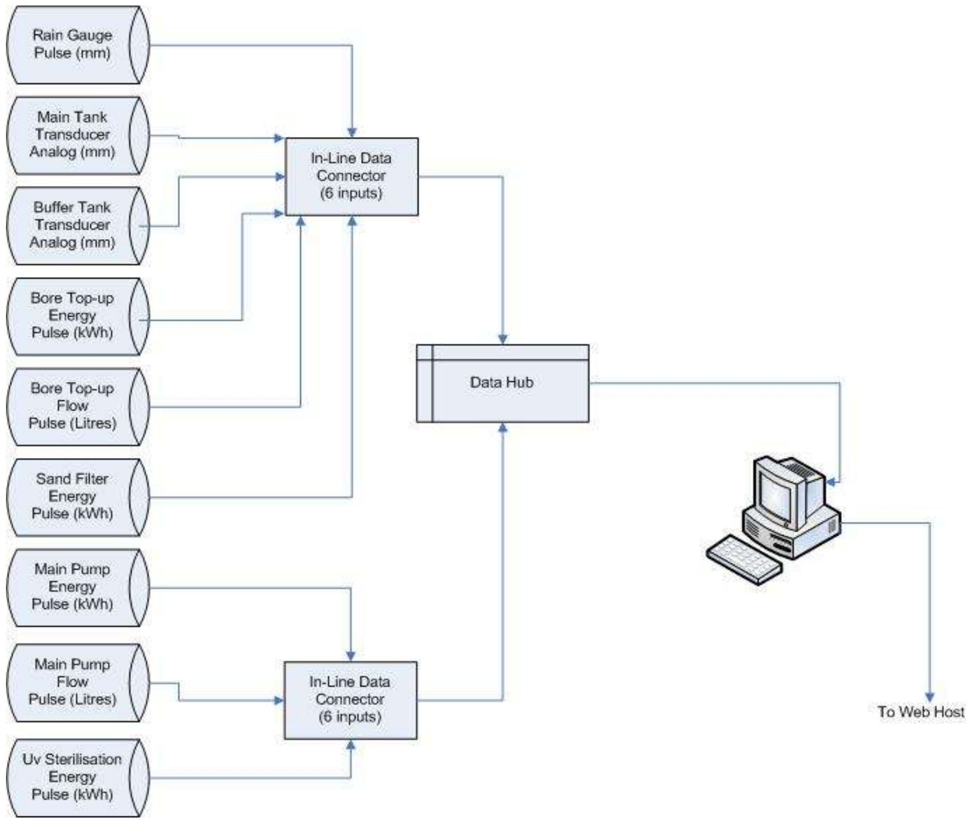

Figure 4: Schematic of electronic data collection system at Capo di Monte.

2.1.5. Methodology Overview

Figure 5 provides a depiction of the flow logic that was used to understand the performance of communal rainwater tanks in the SEQ context. The metering data of energy and flows were the basis for the analysis, which were complemented by modelling to understand system performance under different configurations and climate conditions.

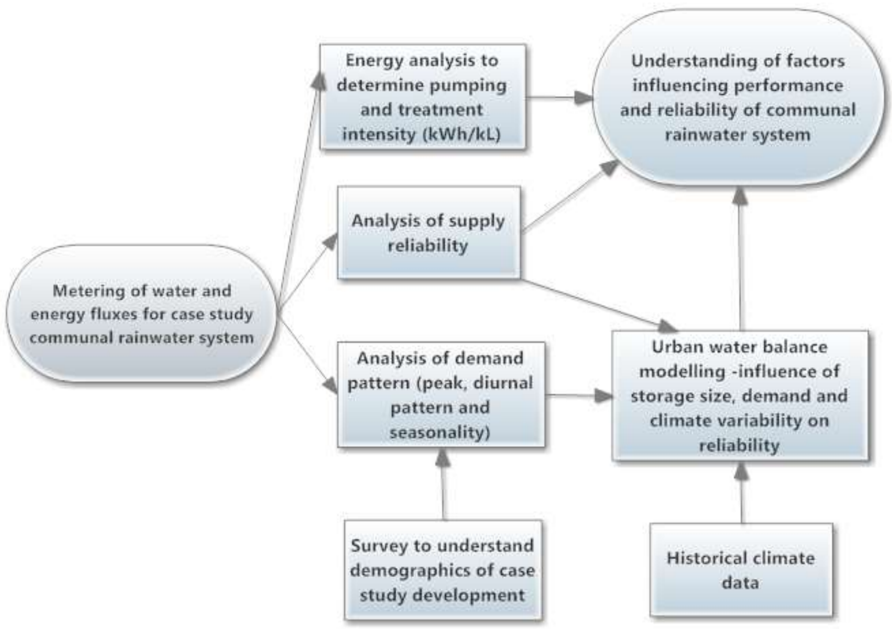

Figure 5: CDM methodology overview.

2.2. Results

The monitoring of CDM commenced in December 2009, and continued until May 2012. Table 1 shows a summary of the manually metered data over the whole monitoring period. This manual data was important in cross checking electronically logged data, as well as identifying any faults in the data logging systems. Table 1 shows that the average per capita water use at CDM, at 156 litres per person per day (L/p/d), was marginally higher than the average water demand reported by the Queensland Government for this region at 150 L/p/d (Queensland Government, 2011). This disparity may in part be due to the older demographic profile of CDM, as Beal *et al.* (2010) showed that older households in SEQ on a per capita basis marginally used more water. This was attributed to older households being home more than working households leading to increased in the home toilet flushing and other uses (Willis *et al.*, 2009b).

The rainwater system, which is used for all internal uses in the home with exception of toilet flushing, was usually able to meet potable demand. Over the 30-month monitoring period, more than 80% of the demand for potable water (35 kL/hh/year) could be met from the harvested roof runoff, with the remainder supplied from treated bore top-up water.

Table 1: Summary of water supply and demand at CDM.

End Uses	Water Supply Source	Average Daily per capita Demand (L/person/day)	Average Yearly Household Demand (kL/household/year)
Potable (communal rainwater system)	Rainwater	58	35
	Bore top-up	12	7
	Total potable	**70**	**42**
Non-potable	Recycled and bore top-up	86	52
Total water use	**All sources**	**156**	**94**

The following sections analyse demand and supply results for the CDM communal rainwater system in more detail, based on electronically logged data. However, in considering the performance of the communal rainwater system at CDM, it is useful to compare recorded rainfall over the monitoring period with long–term averages to determine the influence of climate variability on the reliability of the communal rainwater system. Figure 6 compares monthly rainfall recorded over the monitoring period with long-term averages (1888 – 2012) from a nearby BOM weather station (#040197). This shows that there were some months where monthly rainfall was more than twice the long-term average, particularly during the wetter summer months. The monitoring period also coincided with the breaking of the millennium drought in SEQ, with 2010 in particular being a wet year, with an annual rainfall 800 mm more than the long-term average. The reliability of the communal rainwater system in meeting demand over an extended period, including dry years, is explored in subsequent sections.

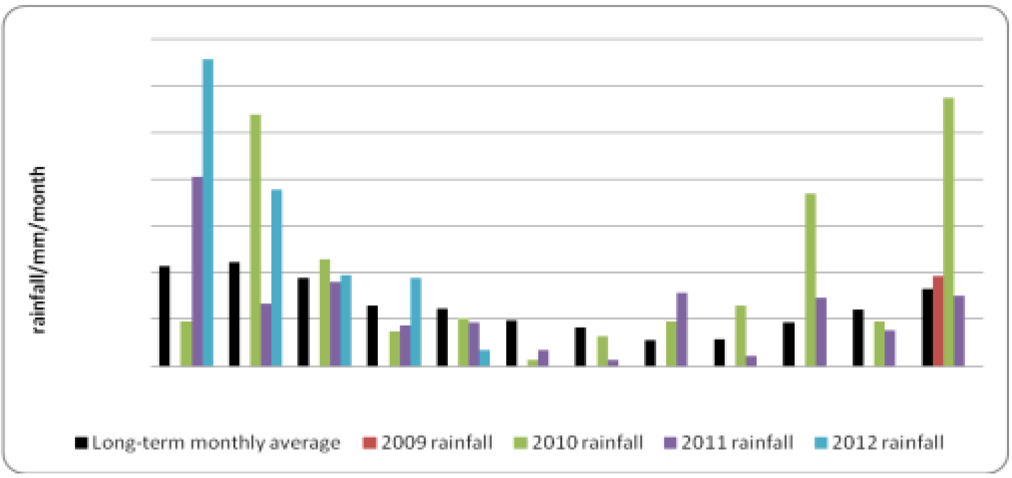

Figure 6: Rainfall over monitoring period vs. long-term averages (rainfall/mm/month).
Source: Averages from Bureau of Meteorology- Station no. 040197: Data period 1888 to 2012.

2.2.1. Monitored Supply at CDM

The monitoring data showed that over the period January 2010 to March 2012, an average 5 kL/day was supplied from the communal rainwater system. This met around 80% of the demand, with the remainder supplied from the onsite bore. Bore supply was greatest during June 2010, the driest month over this monitoring period with only 13 mm of rainfall. During this exceptionally dry month, around 160 kL of water needed to be supplied through bore pumping, which represented 80% of the total water supplied from the potable system for the month.

The electronic data logging system was not fully operational for the same length of time as the manual meters. Therefore, the detailed analysis of the CDM supply system was undertaken for a nine-month period between March 2011 and November 2011 as this period represented a continuous monitoring record. Figure 7 depicts the daily input and output flows for the communal rainwater system, and storage levels, over this monitoring period. This figure shows that daily demand for the rainwater system did not fluctuate significantly as it was used to satisfy indoor potable demand that is not particularly sensitive to climate. Regular rainfall over the monitoring period meant that top-up from bore pumping was only required infrequently. The bore top-up for the potable water system at CDM was not automated, but occurred manually at the discretion of the operations manager. Discussions with the operations manager indicated that bore pumping was activated when remaining supply fell to around 20% of the 200 kL capacity. Thus the effective storage volume for rainwater collection was 160 kL (200 kL - 40 kL), with a further 200 kL quarantined for fire-fighting. This meant that the reserve storage was at least a week's supply based on the average daily potable demand of 5 kL (i.e. 40 kL / 5 kL/day). Figure 7 shows that the lowest point of the effective storage capacity (0 kL of the 160 kL of effective storage) was reached only once, which triggered top-up bore pumping. However, other significant bore pumping events occurred when there was still 20 kL or more of effective storage available. These results indicate that bore top up volumes could be further reduced if the effective storage was allowed to reach the reserve trigger level before top-up was manually activated.

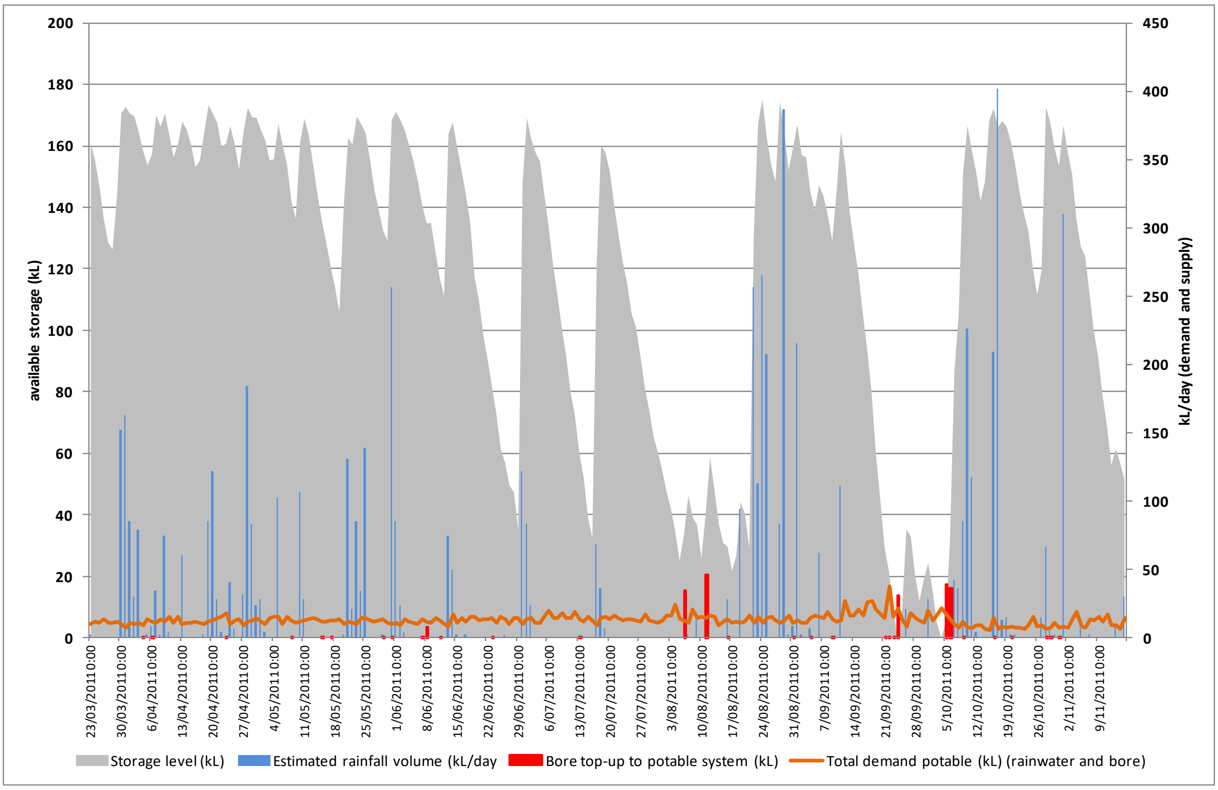

Figure 7: Daily flows and storage level for CDM communal rainwater system.

2.2.2. Monitored Demand at CDM

The demand for potable water at CDM can be categorised into the following end uses: kitchen, laundry and bathroom. The inherent heterogeneity of household water use is worth noting in disaggregating overall CDM water demand to individual households and placing the results in the context of end use studies. Beal *et al.* (2010) made the point that water demand patterns vary significantly between households on the basis of socio-demographics, house size, climate, and cultural practices.

In Australia, there are two comprehensive end use studies that can be used as points of comparisons with the CDM monitoring, and to estimate the breakdown of potable water use in the home: Roberts (2004) and Willis *et al.* (2009a). The study by Willis *et al.* (2009a) was particularly relevant for CDM given that the study was undertaken in a nearby Gold Coast development. The Gold Coast end use study reported on water demand in a 151 detached dwelling development that was serviced by dual reticulated supplies (recycled water and mains water). Table 2 compares these end use studies. In CDM, potable water use of 70 L/p/d (Table 2) was substantially lower than the Gold Coast end use study of 115 L/p/d. Also, contrary to both end use studies, non-potable water demand at CDM of 86 L/p/d was higher than the potable demand. The supply of recycled wastewater for irrigation at CDM probably mean there is less emphasis on water conservation for outdoor uses.

Table 2: Comparison of Capo di Monte of Australian end use studies.

	Melbourne (Roberts, 2004)	**Gold Coast** (Willis *et al.* 2009a)	**Capo di Monte**
Total Potable (litres/capita/day)	122	115	**70**
Total non-potable (garden irrigation and toilet flushing) (litres/capita/day)	88	40	**86**

Figure 8 depicts the average diurnal demand pattern for potable water use in CDM. This depicts a fairly typical bimodal distribution for household water use where there was very little water use before 6 am, a morning peak associated with showering, and another peak in the early evening that can be attributed to meal preparation, dishwashing and showering. Residual water use during the middle of the day can be associated with tap use, and use of washing machines and dishwashers. There was a smaller peak in the late evening, which is likely to be associated with tap use as people get ready for bed. This pattern differs slightly from others generated from dwellings with large families or full-time working inhabitants, which show less residual water use during the afternoon and a more pronounced morning peak, normally created by inhabitants waking and leaving for work before 9 am. There was no significant seasonal variation in potable water demand for CDM, which was to be expected as indoor water demand is not significantly influenced by climate. Daily household potable water demand was also not found to vary significantly among different days of the week over the monitoring period.

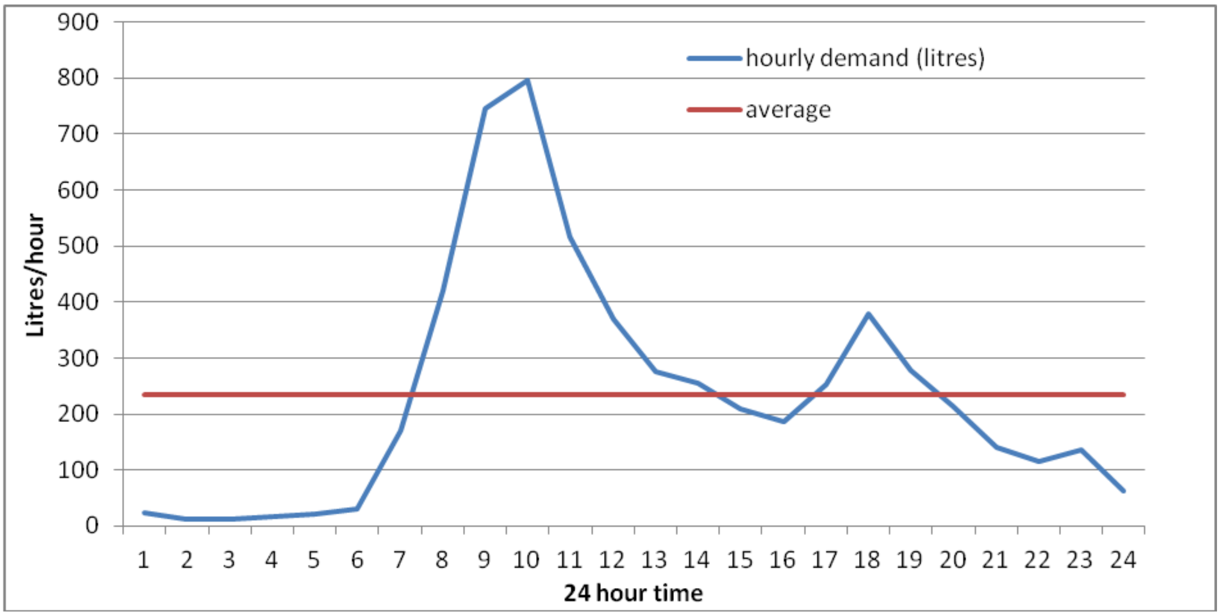

Figure 8: CDM potable system hourly diurnal flow pattern.

Table 3 shows the peaking factors for the CDM communal rainwater system in comparison to government guidelines for water supply planning. A peaking factor represents the ratio between the maximum flow and the average flow over that time period. This shows that mean day maximum month peaking factor was slightly lower than the guideline value, which could be explained by the fact that the communal rainwater system was used for potable demand, which was not particularly responsive to climate conditions compared to outdoor water demand.

Table 3: Peaking factors for CDM communal rainwater system (potable use).

	Capo di Monte Peaking Results	Suggested Water Supply Planning Ranges for Peaking Factors[1]
Mean Day Maximum Month	1.33	1.5 – 1.7
Peak Day Demand	2.91	1.9 – 2.3
Peak Hour Demand	3.33	3.6 – 4.5

[1] Queensland Department of Environment and Planning (2005) Planning Guidelines for Water Supply and Sewerage – Chapter 5: Demand/Flow and Projections – Table 5.4, p 6.

2.2.3. Monitored Energy Demand for Communal Rainwater System at CDM

Other studies of existing decentralised systems conducted by Gardner and co-workers at Payne Rd, Brisbane, showed that the specific energy required for rainwater supply can be up to 5 kWh/kL, which is ten times higher than energy required by centralised systems for pumping water supply in SEQ (Gardner *et al.*, 2006; Beal *et al.*, 2008). Recent monitoring of individual rainwater tanks reported a typical energy intensity of 1.5 kWh/kL, with a range from 0.9 to 1.7 kWh/kL, for systems using rainwater for toilet flushing, laundry and outdoor uses (Retamal *et al.*, 2009). Energy intensity is, however, largely determined by the specific characteristics of each site, which includes system configuration, equipment selection, water use and topography (Retamal *et al.,* 2009; Beal *et al.,* 2008).

A summary of energy use by the CDM potable communal rainwater system is shown in Table 4. This shows that energy consumption at CDM was dominated by pumping of treated rainwater to households, which consumed around three quarters of energy demand attributable to pumping.

Table 4: **Breakdown of specific energy for CDM communal rainwater system - kWh per KL supplied.**

	Specific Energy (kWh/kL)
Potable pressure pump	3.02
Bore top-up pump	0.08
Feed pump for Sand filter	0.65
UV disinfection	0.26
Total specific energy (kWh/kL)	**4.01**

The pump system was an on-demand system with a small pressure vessel, so any water use in the system triggered pumping events. The pump had a 4 kW motor and flow capacity of 0.3 m^3/hour, at 44 m of hydraulic head. Analysis for this system (Sullivan *et al.*, 2011) indicated that the pump was oversized for the system requirements. They showed that a 750 W pump could still meet the head and flow requirements but would reduce the specific energy for pumping by around 50%. The findings by Sullivan (2011) are consistent with those detailed in the next section on a theoretical analysis of pumping efficiency at CDM.

2.3. Pump Efficiency Analysis for CDM

Analysis of the monitoring data revealed that the communal rainwater system at CDM had relatively high specific energy, compared to centralised systems, with most of this energy due demand for pumping. The following investigation was to determine if the pump at CDM could be reconfigured to improve efficiency. Gardner *et al.* (2006) found that start-up energy for rainwater pumping can be a significant contributor to overall energy demand. It was not possible to analyse start-up energy due to the coarse time step for data capture, where flow was monitored at 99.2 Litres/pulse. To analyse start-up energy would require a much finer data resolution, such as 0.5 Litres/pulse. Therefore, the analysis undertakes a theoretical approach to exploring potential efficiency improvements. The friction loss calcultions are shown in Table 5.

Table 5: **Estimated friction loss.**

Parameters	Values	Equation Used
f, Friction Coefficient	0.036	$h_f = \dfrac{8fLQ^2}{\pi^2 g D^5}$
L, Pipe Length (m)	310m	
Q, Peak Flow (m^3/s)	0.0005m^3/s (150L/5min)	
D, diameter of pipe (m)	0.1m	
g, gravitational acceleration (m/s^2)	9.81m/s^2	

The calculated Friction Loss (h_f) is 0.0231 m, which is considered minimal. In order to calculate a pump efficiency curve, total head loss was estimated as per the parameters in Table 6.

Table 6: **Total Head Loss.**

Head Losses	Values
Friction Losses, m	0.0231m
Elevation difference between pump and pipe outlet (max., approx.)	22m
Terminal Head required at each property (from Gold Coast Planning Policies)	22m
Total Head Loss	**44.023m**

The monitoring data showed that peak flow for the communal rainwater systems was 1.8 m^3/hr. The required head used for pump efficiency curves was 45 metres. The pump efficiency curves were estimated using the online WebCaps application (available at: http://net.grundfos.com/Appl/WebCAPS/custom?&userid=GPA&lang=ENU). Table 7 shows the estimated efficiency for the current pump at CDM is 20 to 23%, whilst Table 8 and 9 show that the efficiency for alternative, smaller pumps has increased to about 40%. These results indicate that the current pump at CDM is oversized and that a smaller pump could be more appropriate. Table 10 estimates that a shift to smaller pump could provide an electricity saving of 40 to 66%. Monitoring showed that at CDM around 2,000 kL are pumped from the communal rainwater system per year which required more than 6,200 kWh of power. Therefore, a mains power reduction of 43% for pumping would save around 2,700 kWh. This represents a potential saving of $675 assuming an electricity price of 25 cents/kWh (http://www.originenergy.com.au/2087/Electricity-tariffs-QLD). This would also reduce greenhouse gas emissions associated with the operation of the system. The communal rainwater system generates around 5,600 kilogram of CO_2-e[1] per year. A more efficient pump could reduce greenhouse gas emissions by around 2,500 kilograms CO_2-e per year.

Table 7: Estimated efficiency for current pump (4000 W).

	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., η_p	Shaft Eff., η_s	Overall Eff., η_o
Peak Flow	1.8	45	CRE15-5	4000	221	832	1090	26.5%	76.3%	20.3%
Peak Flow (+20% allowance)	2.2	45	CRE15-5	4000	265	871	1130	30.4%	77.1%	23.4%

Table 8: Alternative Pump 1 (750 W).

	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., η_p	Shaft Eff., η_s	Overall Eff., η_o
Peak Flow	1.8	45	CRE3-10	750	221	409	600	54.0%	68.2%	36.8%
Peak Flow (+20% allowance)	2.2	45	CRE3-10	750	265	468	670	56.6%	69.9%	39.5%

Table 9: Alternative Pump 2 (1100 W).

	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., η_p	Shaft Eff., η_s	Overall Eff., η_o
Peak Flow	1.8	45	CRE3-15	1100	221	395	583	55.9%	67.8%	37.9%
Peak Flow (+20%)	2.2	45	CRE3-15	1100	265	465	672	57.0%	69.2%	39.4%

[1] Estimated based on Queensland Scope 2 emissions for consumption of purchased electricity from the grid – Department of Climate Change (2007).

Table 10: Estimated theoretical power savings from alternative pumps.

L/5min	m^3/hr	Current Pump's Mains Power, Wh	Alternative Pump's Mains Power (CRE3-10), Wh	% Difference (Mains Power)	Alternative Pump's Mains Power (CRE3-15), Wh	% Difference (Mains Power)
10	0.12	909	336	63%	306	66%
20	0.24	914	354	61%	321	65%
30	0.36	930	370	60%	338	64%
40	0.48	941	387	59%	353	62%
50	0.60	954	404	58%	371	61%
75	0.90	980	448	54%	416	58%
100	1.20	1020	495	51%	464	55%
125	1.50	1050	546	48%	521	50%
150	1.80	1090	600	45%	583	47%
180	2.16	1130	670	41%	672	41%
			Average:	54%	Average:	57%

Figure 9 presents a pump efficiency curve for CDM based on recorded pump power data. The pump power values used were recorded at 5-minute intervals, and taken from records between 8:00 am and 10:00 am during the peak demand period. While this period represents the most continuous use of the pump, the CDM pump does not have continuous flow as it is a variable flow pump that runs on demand. However, for the purpose of this exercise, it was assumed that within the 2-hour period of analysis, the pump was running at a constant speed. The curve shows that at peak flow of 1.8 m^3/hr, specific energy was approximately 2.14 kWh/kL. The monitoring data over a 9-month period showed that average specific energy was 3.02 kWh/kL (Table 4), which indicated the pump was much less efficient at lower flower rates.

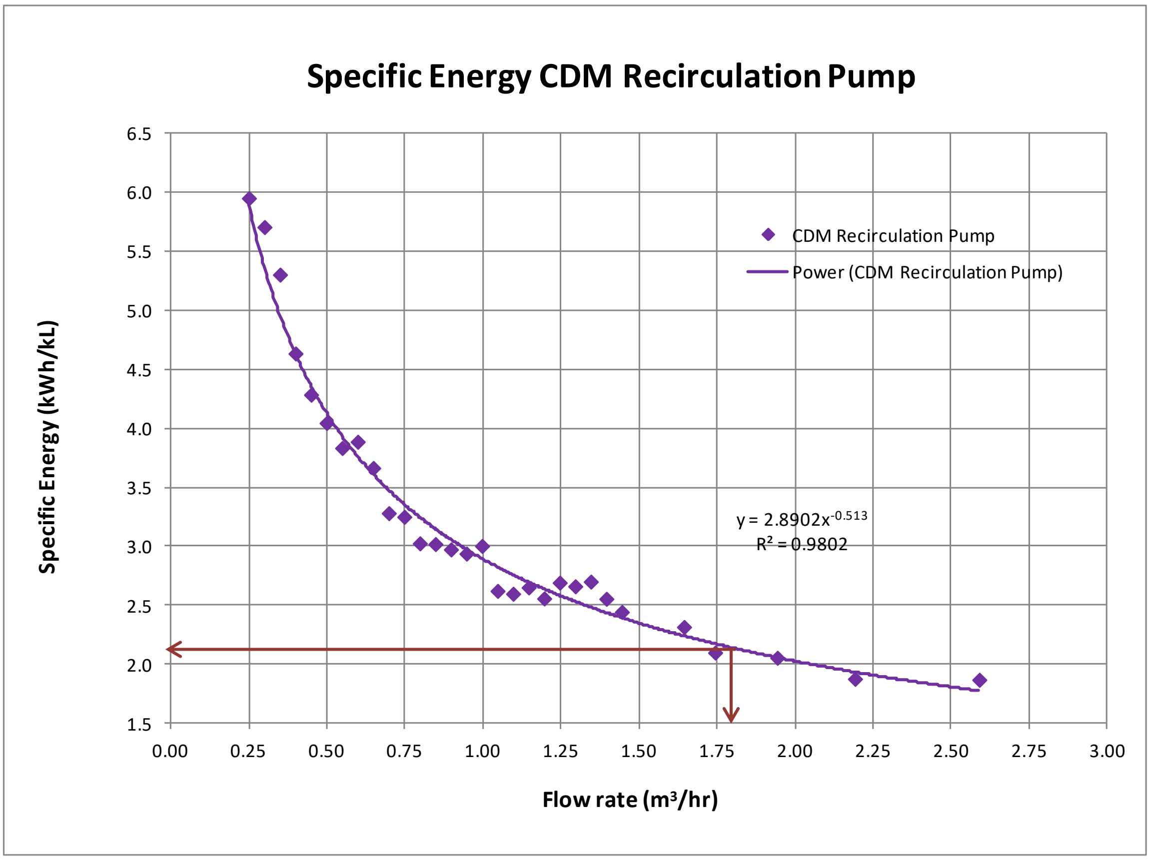

Figure 9: CDM pump efficiency curve.

2.4. Hydraulic Analysis of CDM

Hydraulic performance and efficiency of communal rainwater collection systems was of particular interest for understanding how effective the configuration of the CDM system was in harvested rainwater meeting potable demand, as well as providing guidance on the design of other communal rainwater systems. A distributed rainfall runoff model was setup using the US EPA Storm Water Management Model (SWMM) (Rossman, 2010). The SWMM hydraulic model package was used to carry out the hydraulic analysis of the rainwater collection system of CDM. Important parameters for hydraulic modelling are: (a) climate data (rainfall and evaporation); (b) roof catchment properties (effective roof area, depression storage, slope, imperviousness); (c) conveyance pipe properties (length, diameter, invert levels, roughness coefficient); (d) storage tank/reservoir properties (height, diameter, invert levels at bottom and various pipe connections, height vs. storage relation); and (e) water demand or pump properties (pump rate). The model will be calibrated and validated based on on-site rainfall and water storage level (pressure transducer) relationship. Figure 10 represents the modelling setup.

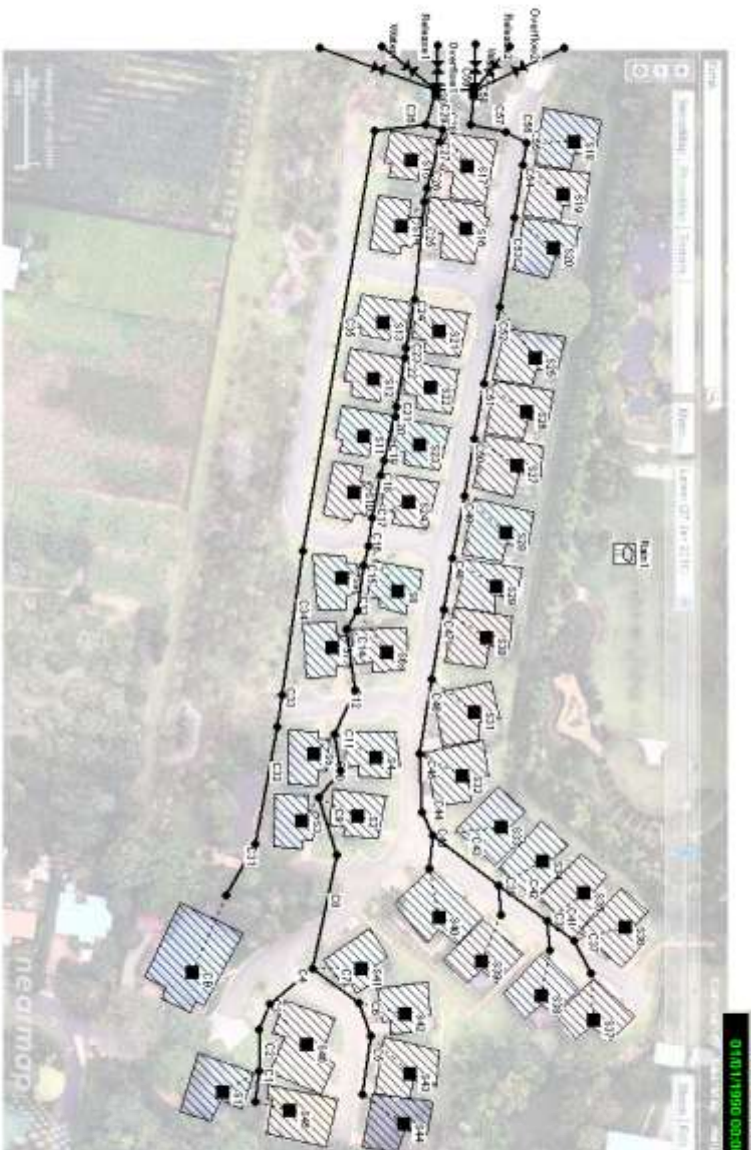

Figure 10: Representation of connected roof area and conveyance system for modelling the CDM system using the SWMM model.

2.4.1. Model Set-Up

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 of the roof areas, i.e. the catchment size for the communal rainwater harvesting system. The Rational Method, which estimates the potential volume of rainfall that can be captured, was also used to check the results obtained from the SWMM hydraulic model. A detailed description of the method used to set-up and calibrate the hydraulic model is contained in Appendix B.

2.4.2. Validation Results

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 by adjusting the values of the pipe roughness or roof parameters 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, with only minor differences in the rainwater tank storage levels.

For a more significant change to be observed in the modelled storage levels, a sufficient modification to the flow entering the pipe network was required. Roof area was identified as the prime physical factor which needed to be modified for calibration purposes. Site inspection of CDM and discussions with the design consultants confirmed that all gutters were connected to the downpipes leading to the conveyance pipe network. Therefore, it was assumed that roof connectivity was 100% for the site, with the entire roof area contributing 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 initial wetting of the surface, 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. Figures 11, 12 and 13 show some results from the calibration and validation runs produced from the hydraulic modelling component of the study.

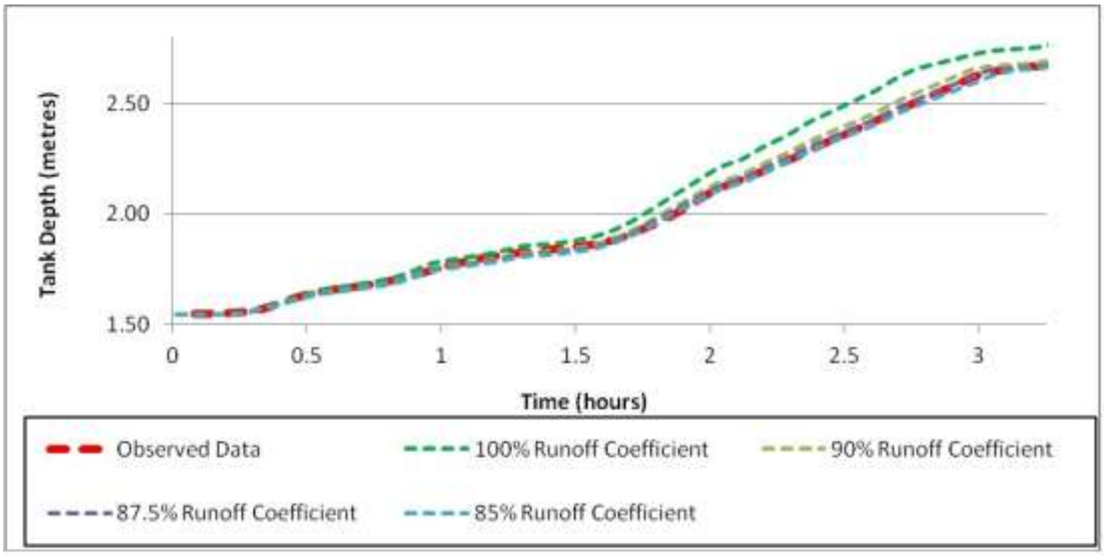

Figure 11: Calibration results from hydraulic model (Aug 21st, 2011).

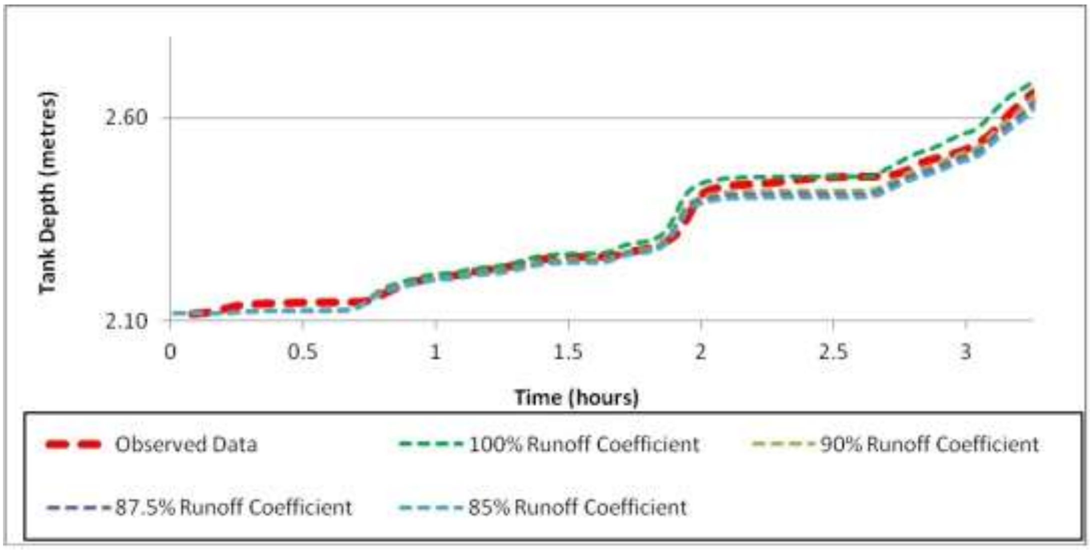

Figure 12: Validation results from hydraulic model (May 19th, 2011).

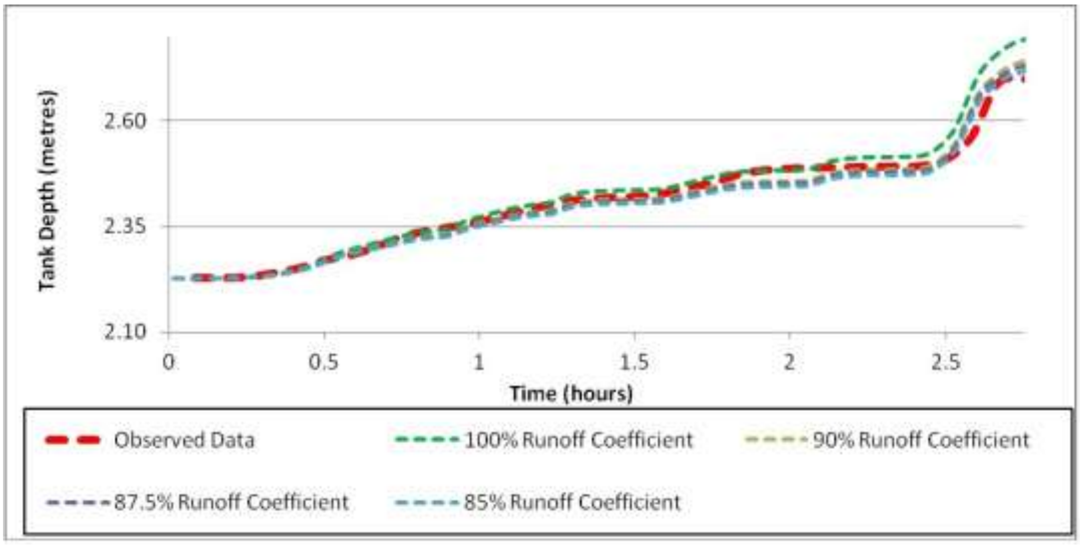

Figure 13: Validation results from hydraulic model (June 29th, 2011).

2.4.3. 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 given as follows:

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

All losses through initial wetting, 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 14. A regression analysis of the plots was used to define the effective contributing roof area (roof area multiplied by runoff coefficient) for the site, defined by the gradient of the line. This was estimated to be 9,130 m^2 and is approximately 85% of the total roof area of 10,689 m^2 of the CDM development and represents the runoff coefficient for roofs for this site. The results of the analysis further support the runoff coefficient of 87.5% estimated by the (SWMM) hydraulic model.

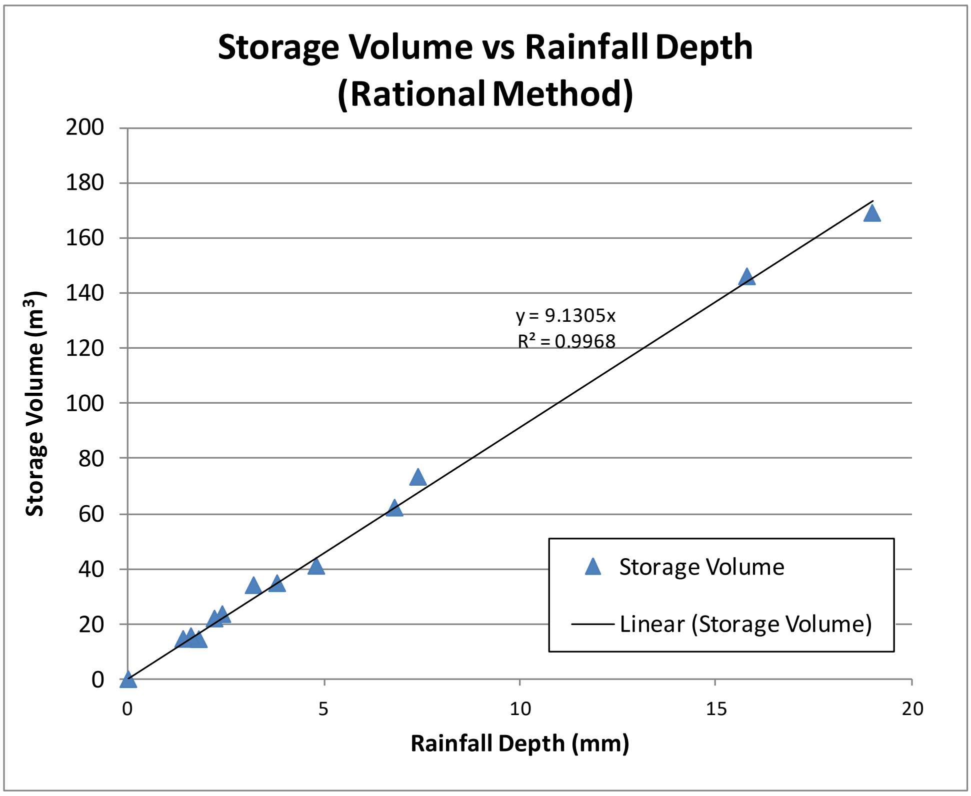

Figure 14: Regression analysis of the Rational Method for CDM.

Roof Drainage

The Queensland Urban Drainage Manual (QUDM) recommends designing gutters and downpipes for roof drainage in accordance with Notes on the Science of Building (NSB) Recurrence Interval (ARI) of 20 years and a critical storm duration of 5 minutes (Department of Natural Resources and Environment, 2007). Bligh Tanner[2] confirmed the design of 151, NSB 152 and NSB 153 (CSIRO) to adequately convey runoff for a design storm with an Average the system was for a 20-year ARI storm, although the contributing flows from each property was limited to a design value of 1.0 L/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. Table 11 shows the IFD table generated for CDM using this method based on data available in Australian Rainfall and Runoff (AR&R) etc.

[2] Consultant engineers responsible for the design of CDM water supply system.

Table 11: IFD table for Capo di Monte.

Duration (hrs)	Rainfall Intensity (mm/hr)							
	Duration	**1 Year**	**2 years**	**5 years**	**10 years**	**20 years**	**50 years**	**100 years**
0.083	5 mins	120	152	186	206	**233**	269	296
0.100	6 mins	112	142	175	193	219	253	278
0.167	10 mins	91.8	117	144	159	181	208	230
0.333	20 mins	67.2	85.4	106	117	133	154	170
0.500	30 mins	54.7	69.7	86.5	96.1	109	127	140
1	1 hr	37.1	47.3	59.1	65.8	75.1	87.2	96.5
2	2 hrs	24.1	30.7	38.5	43.0	49.2	57.2	63.4
3	3 hrs	18.4	23.6	29.6	33.1	37.8	44.1	48.8
6	6 hrs	11.6	14.9	18.8	21.0	24.0	28.1	31.1
12	12 hrs	7.51	9.63	12.2	13.7	15.7	18.4	20.4
24	24 hrs	5.02	6.47	8.25	9.31	10.72	12.6	14.0
48	48 hrs	3.38	4.37	5.64	6.41	7.42	8.78	9.83
72	72 hrs	2.58	3.35	4.34	4.96	5.76	6.83	7.67

Although rainfall data was 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.

From Table 11, the design rainfall intensity for a storm with 20-year ARI and duration of 5 minutes at CDM is 233 mm/hr.

Temporal Rainfall Pattern

Conversions of rainfall intensities to design storms require the use of regional temporal patterns which can be sourced from AR&R Vol 2 (1987) for Zone 3 in which the CDM site is located. Temporal rainfall patterns approximate how rainfall is distributed percentage-wise over the duration of the storm in equal time steps.

As storms with five minutes duration are relatively rare, its temporal pattern is unavailable in the AR&R guides. For the purpose of carrying out a hydraulic modelling analysis for the 20-year ARI storm with five minutes duration, it is assumed that 100% of the rainfall is distributed over this duration; in this case, 19.5 mm of rainfall is modelled over a period of five minutes equivalent to an intensity of 233 mm/hr. The hydraulic modelling analysis for the design storm is carried out in later sections of the report.

Downpipe Flows

Estimations of flow through a downpipe are important to determine if the roof is capable of draining design storms adequately. QUDM suggests roof drainage to be able to convey design storms of 20-years ARI and 5-minute duration without overflowing.

On average, there are approximately six 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 (QUDM, 2007):

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

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 of the six downpipes is equal to 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):

$$Q_{dp} = B.C.A\sqrt{2.g.H}$$

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

Assuming that there are no blockages in the gutters (B = 1), then from the orifice equation, the maximum effective flow that a downpipe is able to convey 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 sized.

Downpipes Connections to Collection System

An in-ground pipe installed around the property connects downpipes at the corners of the building, and is joined to the main collection system through a connector. Figure 15 shows a schematic of the connection pipework.

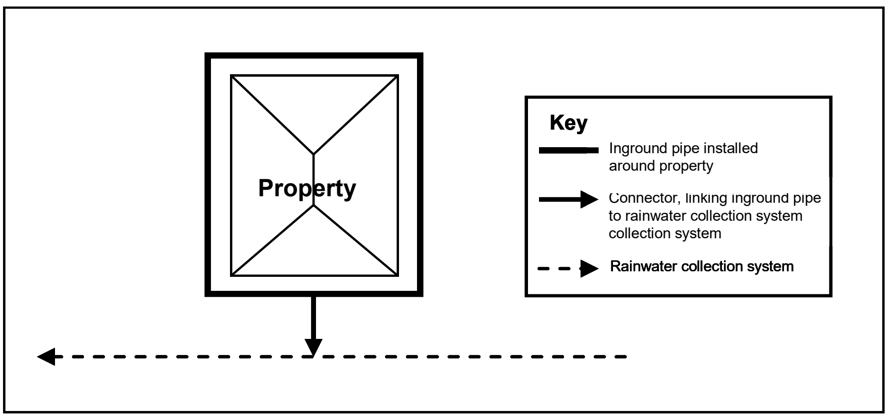

Figure 15: Schematic of in-ground pipe and connectors around a typical CDM household.

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 16 shows a typical layout of how runoff from a roof is connected to the collection system.

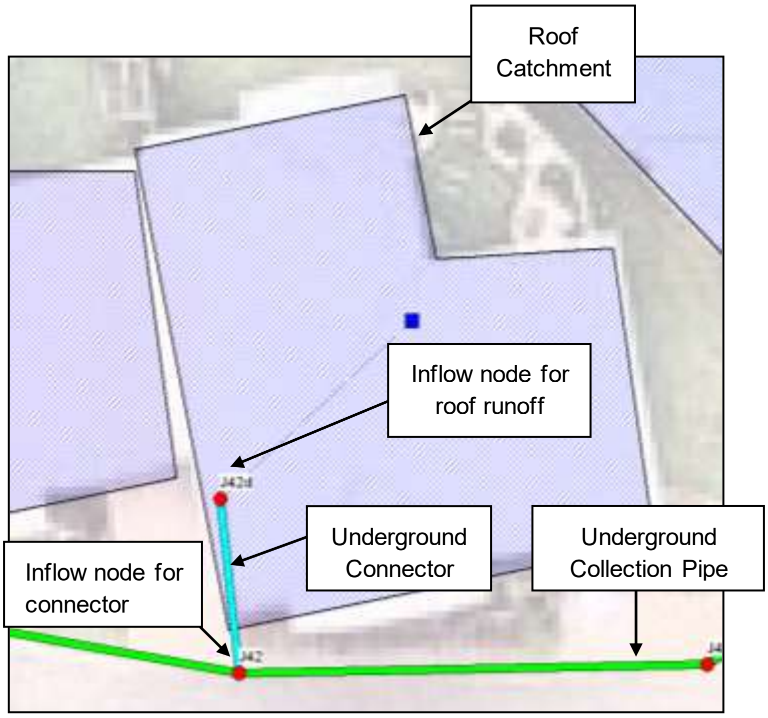

Figure 16: Typical layout of roof runoff connection to main pipes.

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

Model calibration and validation had been carried out for low rainfall events, with rainfall intensities not exceeding 2 mm/5 min (24 mm/hr). 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 12 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 12: 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 peak flows up to 0.00129 m^3/s are being contributed into the system from each property through the connectors. Furthermore, runoff generated on the roofs, after losses are taken into account, passes through the connectors without being impeded. The highest modelled flow of 0.00129 m^3/s (1.29 L/s) in the connector shows that each property within the collection system contributes more than the consultant's suggested value of 1.0 L/s.

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

Calibration for Higher Rainfall Event

Modelled results for a rainfall event recorded on the 24th November 2011, peaking at 2.6 mm/5 min, showed the predicted volume collecting in rainwater storage tanks greater than was measured. This showed that each roof was allowing more flow than necessary into the system, possibly indicating that the connectors' chosen diameter of 100 mm 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 75 mm diameter connector with a roughness of 0.025 provided a marginally better representation of the connectors in the system than the original choice of 100 mm 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 in-ground 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. Figure 17 shows the tank storage levels with the original sizing of 100 mm and with the adjusted sizing of 75 mm and roughness of 0.025 for this event, which shows that the adjusted sizing had a minor influence on calibrating more closely to observed data.

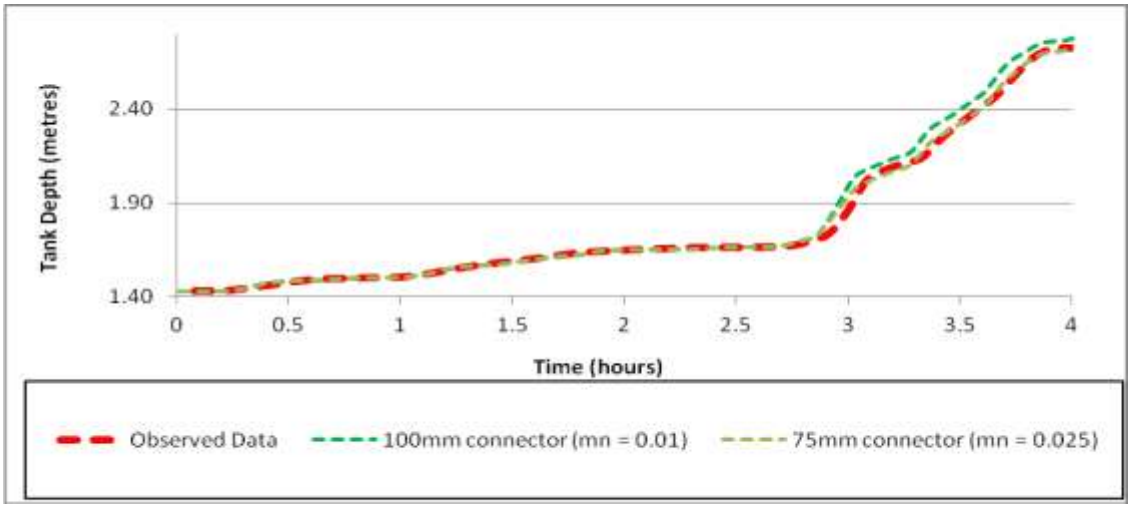

Figure 17: Calibration result for higher intensity rainfall (24th Nov 2011).

Flows generated from one of the "higher rainfall" smaller event, the June 29th event, had a maximum intensity of 2 mm/5 min, producing a peak flow of 0.00129 m^3/s which, with the initial downpipe sizing of 100 mm, was able to pass fully through the connectors. Using the updated connector size of 75 mm, the connector full capacity was also 0.00129 m^3/s, indicating that the system was capable of handling a storm of an equivalent intensity. Any flow beyond this, associated with an event of higher rainfall 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 2 mm/5 min; essentially, the maximum generated runoff from the roofs would be equal to or less than the connectors' capacity of 0.00129 m^3/s.

Q20, 5 Minute Duration Model Run

With the hydraulic model fully calibrated, a simulation was carried out for a 20-year ARI, 5-minute duration storm, to test the effectiveness of the collection system in handling the recommended design storm. As stated in previous sections, no temporal pattern was available for 5-minute duration, hence 100% of the rainfall would be distributed over the design duration; i.e. 19.5 mm over 5 minutes.

Although it has been shown that the gutters and downpipes are able to handle the flows produced from the design storm, the modelling results demonstrate that the connector is the ultimate controller of the rate of rainwater entering into the system. As connectors can only handle rainfall intensities of 2 mm/5 min, the 20-year ARI design storm, approximately 17.5 mm of rainfall over 5 minutes, would be unable to enter the system and hence, overflow into the stormwater network.

As a result of these flow limitations into the system, the majority of the rainfall is lost to the stormwater system and the collection pipe network is able to cope with the reduced flow from the design rainfall event. Additionally, flows entering the system via the connectors were not observed to surcharge the rainwater collection pipes.

2.4.4. Hydraulic Modelling Conclusions

The hydraulic modelling study showed that the Rational Method was able to predict with similar accuracy, the volume of rainfall potentially collected from roofs from a rainfall event provided the runoff coefficient is of a reasonable value, ranging between 0.85 (85%) to 0.90 (90%).

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

Each property within the site is able to contribute a maximum of 0.00129 m^3/s (1.29 L/s) to the rainwater system, which is higher than the consultant's estimate of 1.0 L/s. This is due to the size of the connector, which effectively limits the flow entering the main collection pipe. The conveyance pipe leading to the storage tanks is sized such that it can convey the cumulative runoff from all dwellings provided each dwelling contributes no more than 1.29 L/sec. This capacity is due to conveyance pipe slope and its diameter.

The modelling shows that for rainfall events higher than 2 mm/5 min, connectors control the amount of flow entering the conveyance pipe network. Due to these restrictions, the pipe network is able to cope with the design storm of 20-year ARI and 5-minute duration and no pipe surcharge is observed.

The long-term (30 year) water balance modelling has taken into account the maximum flow that can enter the conveyance pipes, by removing rainfall greater than 1.3 L/sec from the 6 minute climate data used. This is described in the following section.

2.5. Water Balance Modelling for CDM

Water balance modelling was undertaken to explore the reliability of the communal rainwater system at CDM under different operational configurations and its resilience to drier years.

2.5.1. Urban Volume and Quality Model

The approach to modelling the water balance applied a straightforward 'bucket' approach that simulated all the main inputs and outputs from the communal rainwater tank. This mass balance approach was based on the law of conservation of mass: any change in rainwater tank volume during a specified period must equal the difference between the water added and the water withdrawn (adapted from: Zhang *et al.* 2002).

The water balance was implemented using the Urban and Volume Quality (UVQ) model[3]. UVQ is a daily time-step model that simulates water flow paths and contaminant balances through the urban water cycle (see Mitchell and Diaper, 2006). The functionality of UVQ has been designed to allow the user to define both conventional and non-conventional urban water supply and wastewater services, and explore the impact of different scenarios on water flows and contaminant loads and distribution. UVQ can be run from a minimum period of one year up to one hundred years. In order to account for climate variability at different temporal scales it is best for the simulation period to run over a period of decades.

In the UVQ model, imported water supplies and rainwater are the major inflows to the urban water cycle, while wastewater, stormwater and evaporation are the main outflows. Water sources can be used for indoor and outdoor end-uses. Specific end-uses are: kitchen, bathroom, laundry, toilet, garden irrigation and public open space irrigation.

In UVQ the rules for satisfying household demand are as follows:

- Lowest quality water source available for the end use is drawn on first (for example, harvested rainwater is used before potable water for approved indoor uses).
- Indoor demand is satisfied before outdoor demands (for example, if harvested rainwater is available for toilet flushing and garden irrigation then toilet demand is satisfied first).

2.5.2. Model Inputs and Assumptions

Demand for the water balance modelling was based on data recorded from the monitoring of the CDM development, and disaggregated to indoor uses based on proportions specified in Beal *et al.* (2012). The spatial dimensions, in particular roof catchment, was based on analysis of aerial photography using a GIS, which was consistent with the approach detailed in the preceding section on hydraulic modelling of CDM.

Historical rainfall recorded at a six-minute time step was used from the nearby Mount Tambourine weather station for the period 1982 to 2005. The water balance model aggregated six-minute rainfall intensity data to daily records, but the values were adjusted to reflect the effective rainfall. Effective rainfall is the runoff from the connected roof area that can be captured by the downpipe system at CDM, where roof runoff greater than 1.3 L/sec was discounted from the daily rainfall aggregation based on the design hydraulic capacity of collection pipes. The calculation of effective rainfall was based on the rational runoff equation (detailed in preceding section).

The average annual rainfall over the 24-year record, was 1,318 mm. This occurred in a pattern of relatively wet years, with annual rainfall around 2,000 mm, interspersed with drier years, with rainfall of around 1,000 mm (see Figure 1). The monitoring period for this study coincided with a relatively wet period with rainfall for 2010 of 2,300 mm, which is in the 95th percentile range. Therefore, it is worth exploring through water balance modelling how the CDM communal rainwater system is likely to perform in drier years and average years.

For the water balance modelling, it was assumed that the available rainwater storage of 200 kL was zero at the start of the simulation, ie the tanks were full. It was also assumed that top-up with bore water occurred when the available storage reached 20% of effective capacity (40 kL). Therefore, an active available storage of 160 kL was assumed for the analysis.

[3] Available at: http://www.csiro.au/en/Outcomes/Water/Water-for-cities-and-towns/UVQ.aspx

2.5.3. Water Balance Modelling Results

The results in Figure 18 are presented as annual averages over the simulation period of 24 years to account for both seasonal and yearly fluctuations in rainfall. The summarised results demonstrate that bore top-up is only needed to satisfy a small proportion of the demand. This also highlights that, on average, around 86% of the roof runoff (14,096 kL/yr) ends up as stormwater overflow (12,068 kL/yr).

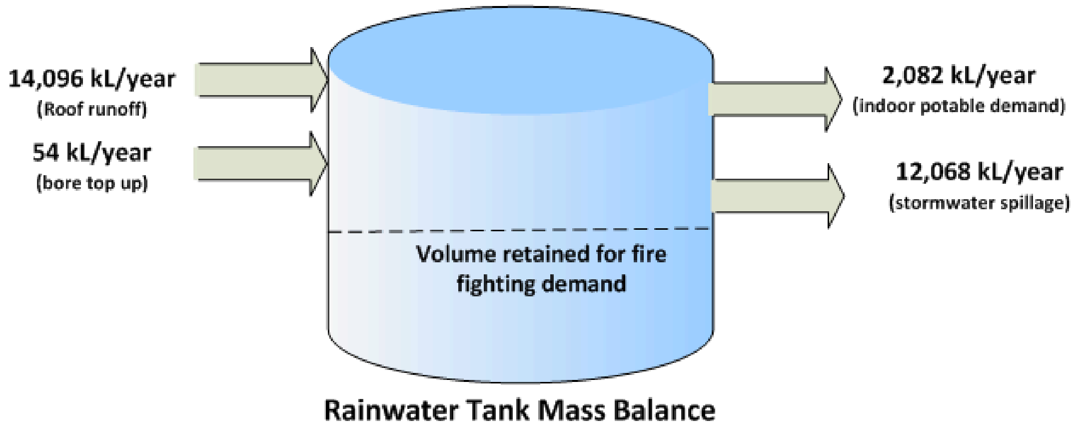

Figure 18: Average annual mass balance CDM rainwater system (1982 to 2005).

We have defined volumetric reliability of the CDM rainwater systems as the ability to satisfy demand from the active storage, without top-up from bore pumping. First we explore the influence of storage size on reliability. Figure 19 depicts the simulated annual average volumetric rainwater yield for different active storage volumes. This shows that for the active storage at CDM (i.e. 160 kL), on average, demand could be satisfied from rainwater inputs on 98% of the days over this 24-year period. To increase the reliability to 99.5% would have required an active storage size of 280 kL. However, the average values can obscure reliability performance during periods of uncommonly low rainfall. The simulation of the 280 kL storage still showed that the rainfall system would have failed for 21 days in 1991, 13 days in 2002, and 10 days in 2004. To avoid supply failure in 1991, 420 kL of storage would have been required. This analysis demonstrates the difficulty of providing 100% supply reliability based only on captured rainfall, given the intermittency of rainfall events.

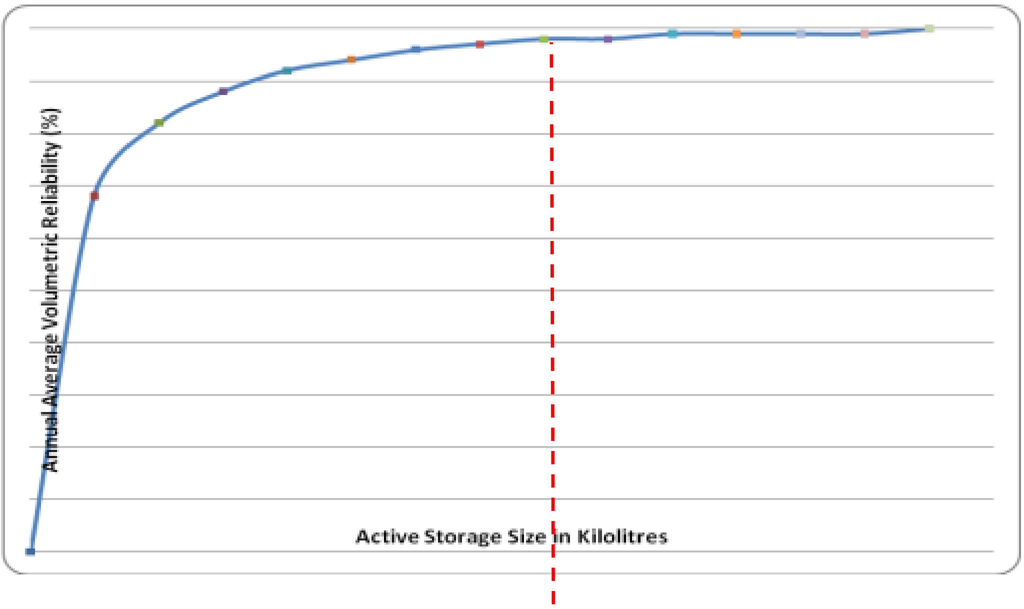

Figure 19: Volumetric reliability of CDM communal rainwater system for different storage sizes.

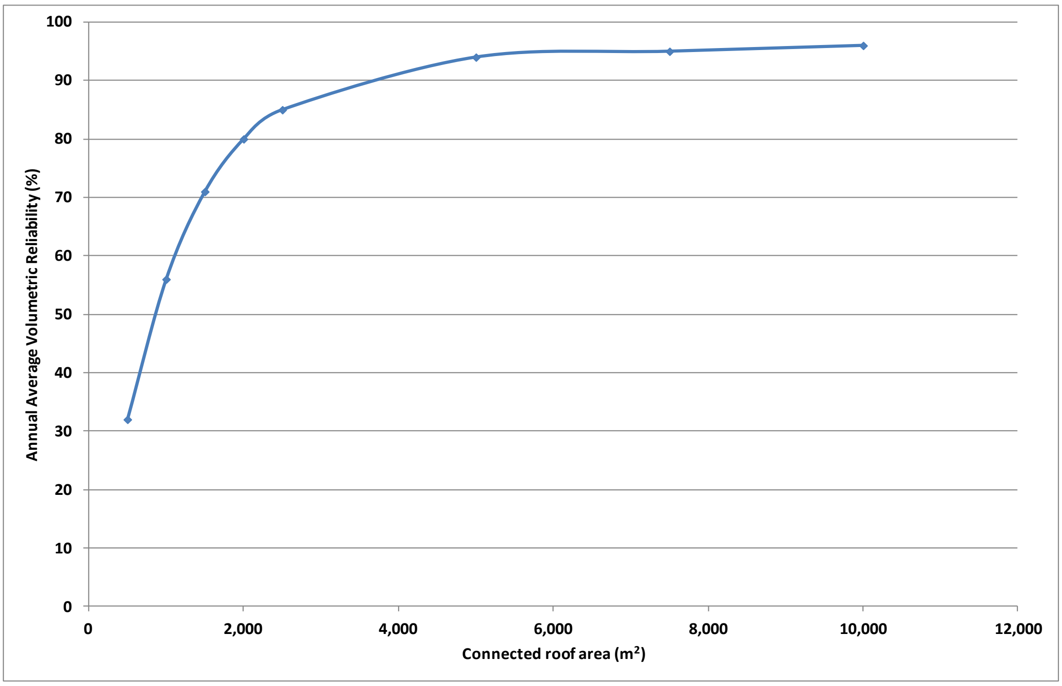

Figure 20: Annual average volumetric reliability and connected roof area.

Figure 20 depicts the relationship between connected roof area and annual average volumetric reliability. This was modelled using the effective storage size of 160 kL and a constant demand of 5.4 kL per day. This analysis showed that halving the connected roof area at CDM from the actual 10,000 m^2 would only reduce the harvested rainwater on average from 2,006 kL/yr to 1,949 k/yr. This correspondingly reduced volumetric reliability from 98% to 94%.

Our analysis showed the CDM system was configured to provide about 98% of the potable demand from harvested rainwater when averaged over the rainfall history. The connected roof area could be reduced with only marginal reductions in the yield, whilst further increases in the storage capacity would only realise small increases in volumetric reliability. This simulated reliability value from the UVQ model is higher than that found from the monitoring of CDM, where around 80% of potable demand was satisfied by harvested roof runoff.

This may indicate that the manual operation of the CDM bore top-up was not optimised to make best use of the available storage. The top-up occurs based on operator judgement taking into account the short-term rainfall projections, storage levels and the need to maintain at least a week's demand (40 kL) in storage to secure supply. The higher yields simulated from the UVQ model may also indicate that losses, such as storage spillage and roof losses, are underestimated due to the daily time step in the model. However, an urban water balance model, such as UVQ, presents a useful approach to broadly understand the expected yield and reliability of a communal rainwater system, accounting for variability in rainfall, during the planning stage of a system. This assessment may be used to initially assess the feasibility of meeting demand with a communal rainwater system, which could then be used to justify a more detailed analysis of the system design.

2.6. Assessment of Performance against QDC MP 4.2

In comparing the performance of CDM against the requirements specified in QDC MP 4.2, it needs to be recognised that CDM does not present the typical example of using a communal rainwater tank to reduce mains water demand. CDM has been designed to be self-sufficient from the centralised water and wastewater system, therefore the regulatory requirement to reduce demand on mains drinking water does not apply. Also, the rainwater system at CDM has been designed with a treatment process so that it is fit for potable demand. In most cases, alternative water sources are applied to meet non-potable water demand on a fit-for-purpose basis. However, the monitoring of the CDM communal rainwater harvesting configuration, particularly the ability of the supply to meet demand, can be used to assess the feasibility of using a similar approach to meet the mains water saving target in QDC MP 4.2.

The mains water saving targets in QDC MP 4.2 were based on modelling undertaken and reported by WBM Oceanics Australia where they investigated the potential yield from rainwater tanks in five climate zones of SEQ (see WBM Oceanics (2006) *South East Queensland Regional Supply Strategy: Rainwater Tank Modelling Report*). Average daily demands for the different climate zones were used in estimating potential yield from rainwater harvesting.

The assumptions used by WBM Oceanics (2006) in modelling rainwater yields for the Brisbane climate zone were:

- Total average daily household demand: **925 litres/day,** which was broken down into *439* litres of external water use and *486* litres of internal use;
- Potential indoor uses for rainwater were assumed to be the following percentages of total indoor demand: laundry (20%), toilet (25%) and bathroom (30%);
- Rainwater was used for the following in different development types simulated:
  - Brownfield – laundry, toilet and outdoor
  - Greenfield one – laundry, toilet, outdoor and shower
  - Greenfield two- laundry and shower (other non-potable demand satisfied by recycled water);
- Trickle top-up characteristics: rate of 39 L/hr and a trigger storage volume of 1,110 L.

The analysis of CDM has shown that 35 kL of rainwater was supplied annually for each household. The configuration of connected roof area (230 m^2 per connected household) and available storage (4,350 litres per connected household) exceed the requirements suggested under the QDC, i.e. each attached Class 1 dwelling has 3,000 litres of storage connected to at least half the roof area, or 100 m^2, whichever is the lesser. However, the mains water substitution for attached dwellings of 42 kL/household/year defined in the QDC was **not** achieved using the communal rainwater system at CDM. This was strongly influenced by the much lower household demand for potable water at CDM when compared to the assumptions used to develop QDC MP 4.2, where the demand to be satisfied by rainwater supply was more than three times the average household demand at CDM (see Table 1).

2.7. Conclusions and Recommendations

Communal rainwater systems are an alternative method for developers to provide potable water services to developments that are not connected to municipal supplies i.e. decentralised developments. These systems may also be suitable for urban infill developments where existing infrastructure is already at capacity and upgrading may prove to be economically or logistically non-viable. However the performance of communal systems to deliver safe, reliable supplies of water has yet to be largely tested as these systems are relatively new.

Results of monitoring at CDM have shown that, with a small amount of top-up from the groundwater bore, the water system can meet the potable demand of the 75 residents supplied by the system. Modelling using historical climate data showed that the system is configured to be resilient even in relatively dry years, with most of the potable demand being satisfied from harvested roof run-off.

However, this decentralised water supply comes at an energy cost. The system required around 4.0 kWh/kL to treat and reticulate the rainwater/groundwater for potable demand. This equates to around eight times the energy required for centralised potable water treatment and pumping in SEQ, and is marginally more than the 3.2 kWh/kL energy required for desalination treatment by reverse osmosis (WaterSecure, 2011). However, there are opportunities to substantially improve the energy efficiency of this system through smaller pump sizing that would reduce both electricity demand and greenhouse gas emissions.

The hydraulic analysis demonstrated that the Rational Method was able to estimate runoff with reasonable accuracy when compared to a calibrated SWMM hydraulic model. The analysis demonstrated that the pipe network has been configured to cope with a design storm of 20-year ARI and 5-minute duration.

Communal rainwater systems offer a number of advantages over other alternative water sources at the development scale (i.e. stormwater, recycled water and desalinated water) as roof runoff can provide a relatively high quality water source, which can be directly used for non-potable uses, or with filtration and disinfection, for potable uses. The results reported have demonstrated that a communal rainwater system can reliably provide an alternative water source, with minimal reliance on back-up supply. A communal approach to harvesting and treating rainwater also means that individual householders do not have to maintain and operate their own tank (and treatment system). A communal system, such as the one studied at CDM, is managed by the Body Corporate which helps ensure management of a high quality is achieved. Also, communal rainwater systems may be more appropriate in medium density developments where there is high building ratio to allotment area, which limits space available for individual rainwater tanks.

3. GREEN SQUARE NORTH TOWER – RAINWATER HARVESTING IN A HIGH-RISE COMMERCIAL BUILDING

3.1. Green Square North Tower Overview and Background

Green Square North Tower (GSNT) is located in Fortitude Valley, which is on the edge of the central business district of Brisbane. GSNT is a twelve storey commercial office building that was designed to meet a 6 star standard under the Green Star Rating scheme (http://www.gbca.org.au/green-star/green-star-overview/). The initiatives for water conservation included waterless urinals, and the harvesting of roof runoff for toilet flushing and landscape irrigation in lieu of mains water.

Commercial water use is a significant component of overall urban water demand. In Australia, it was estimated that in 2007 commercial water use made up around 15%, or 650 GL, of the total demand for urban water (ABS, 2010). In developing integrated, consistent policies and strategies for increasing the resilience of urban water systems, there is a need to incorporate the commercial sector. However, policy development for integrated water cycle management in the commercial sector has been impeded by a lack of empirical studies that validate the performance of alternative configurations and their cost-effectiveness.

This monitoring study of a commercial building has evaluated the reliability of roof-harvested rainwater in meeting non-potable demand, and the associated energy use. The research also investigated the potential yield and quality of other non-potable sources that could be taken advantage of in commercial buildings to provide a reliable alternative water source while minimising energy and life cycle costs, and also reducing the complexities for managing and operating these decentralised water systems.

The research explores the potential of alternative water sources in commercial buildings for substituting imported potable water for non-potable applications. In the residential sector, the role of decentralised alternative water sources in reducing demand for imported potable water and reducing the environmental impact of urban development has received considerable attention both from researchers and policy makers (Coombes and Kuczera, 2003; Jones and Hunt, 2010; Khastagir and Jayasuriya, 2010; Kim and Furumai, 2012; Vialle *et al*., 2011; Villarreal and Dixon, 2005). However, there are limited studies on the implementation of alternative water sources for commercial buildings. Chilton *et al*. (2000) reported on the performance and value proposition of a rainwater harvesting scheme for a supermarket roof, used for toilet flushing. This study found that the configuration of the systems, particularly effective roof catchment area and storage volume, were critical in determining the reliability and payback period. Imateaz *et al*. (2011) undertook a retrospective optimisation of tank sizes for a system harvesting rainwater from large roofs at a university campus for landscape irrigation. This theoretical study, based on a daily water balance model, found that the storages were not resilient to dry years and that reducing the payback period was likely to be dependent upon correct tank sizing.

There is a lack of studies and published data on the performance of non-potable water systems in commercial buildings. The lack of empirical data means that developers lack the information required to make informed decisions in the design and planning stage that assesses different options for reducing demand for imported potable water through source substitution. Also, assessing the performance against triple-bottom line sustainability objectives for commercial buildings is difficult for water.

3.1.1. QDC MP 4.3 (Alternative Water Sources for Commercial Buildings)

The Queensland Development Code (QDC) Mandatory Part (MP) 4.3 specifies that commercial buildings supplied with mains drinking water must provide an alternative water source for suitable uses. The suitable measures identified are a rainwater tank, water storage tank, common tank, or a greywater treatment system. To fulfil the requirements of MP 4.3, rainwater systems need to be connected to any swimming pool on the lot, any external use, each pedestal (toilet), washing machine cold water taps, and any other fixtures specified by the local government planning provisions. The QDC requires that rainwater systems must have a minimum storage of 1500 L per connected toilet, and a connected roof area of 50 m^2 for each connected toilet, or the available roof area, whichever is the lesser. This research compared the performance of the GSNT rainwater system against the requirements specified in QDC MP 4.3, and provides some insights into the potential for wider uptake of rainwater harvesting in commercial buildings for non-potable applications.

3.1.2. GSNT Rainwater System

Figure 21 depicts the hydraulic circuit of the GSNT rainwater system, and the associated metering system that was used to validate the reliability of the systems in meeting non-potable demand and the associated pumping energy demand. Rainwater was harvested from an effective roof area of approximately 1,600 m^2 then fed via downpipes to an 80 kL basement storage tank. The water in the basement tank was then pumped back to the roof to two smaller header tanks (40 kL and 27 kL) that were used to supply toilet flushing and garden irrigation respectively, using a gravity feed system. The header tanks have pressure sensors, so that when the water level falls to a certain point, the pressure switch activates pumping from the basement tank. In the case of the toilet tank, there was back-up supply from the mains water supply if demand could not be satisfied by rainwater. Overflow from the basement rainwater tank, following heavy rainfall events, was directed to a wet well where it was then pumped into the stormwater system.

The toilet tank was used to supply the demand for flushing of 146 full and half flush toilets, with an estimated water use of 6 litres for a full flush and 3.8 litres per half flush. The irrigation tank was used for irrigating window planter boxes, however, as these planter boxes were not very extensive, their irrigation water demand was relatively minor.

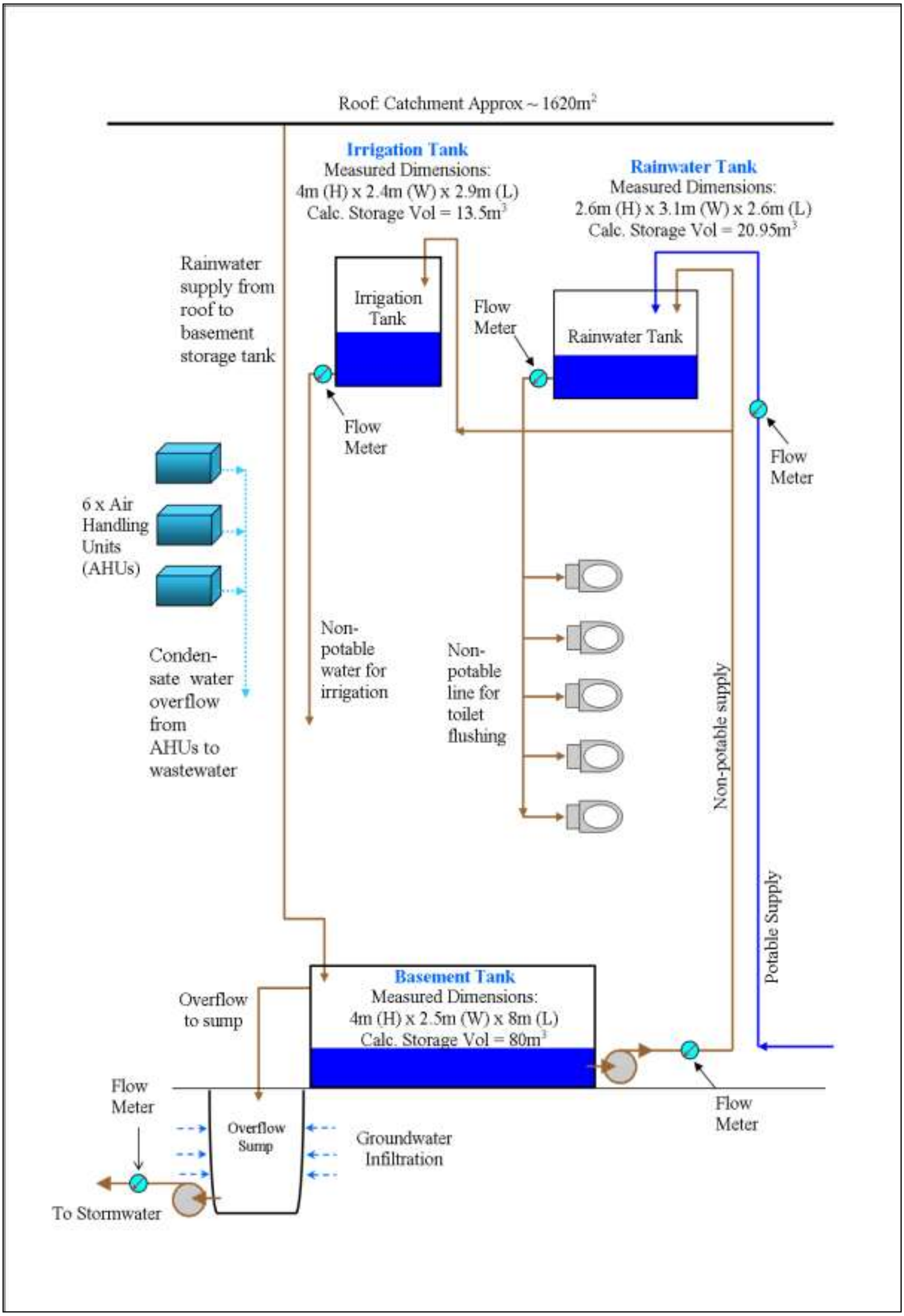

Figure 21: Hydraulic circuit of Green Square North Tower rainwater harvesting system.

3.1.3. Monitoring System

Monitoring of energy and water fluxes at GSNT was undertaken using a high-frequency logging device that recorded water flows and energy pulses at 6-minute time interval. A data logging system stored the data in 6-minute, hourly and daily data files. Manual recordings taken monthly from the water and energy meters were used to cross check on the electronically logged data (see Appendix A for more details on monitoring system).

3.1.4. Methodology

Figure 22 summarises the key steps of the research approach applied. The research was grounded in the monitoring study of water and energy fluxes for the GSNT rainwater system. This primary data collection and analysis provided a foundation for considering overall system performance, and then identifying potential opportunities for providing improved approaches for substitution of potable demand with non-potable water sources. This included social aspects by gathering perspectives from the building owners, designers and operating managers on issues experienced with the implementation and operation and maintenance of the rainwater harvesting scheme. The technical feasibility of other non-potable water sources available at GSNT was also investigated through measuring flow rates and water quality sampling. This information was used to consider the potential role that less recognised alternative water sources could play in providing cost effective solutions that maximise level of substitution as well as satisfying user operating requirements. The information developed through the application of the methodology provided a basis for assessing the relative strengths and weaknesses of different alternative water sources, and how a combination approach may provide the best outcome.

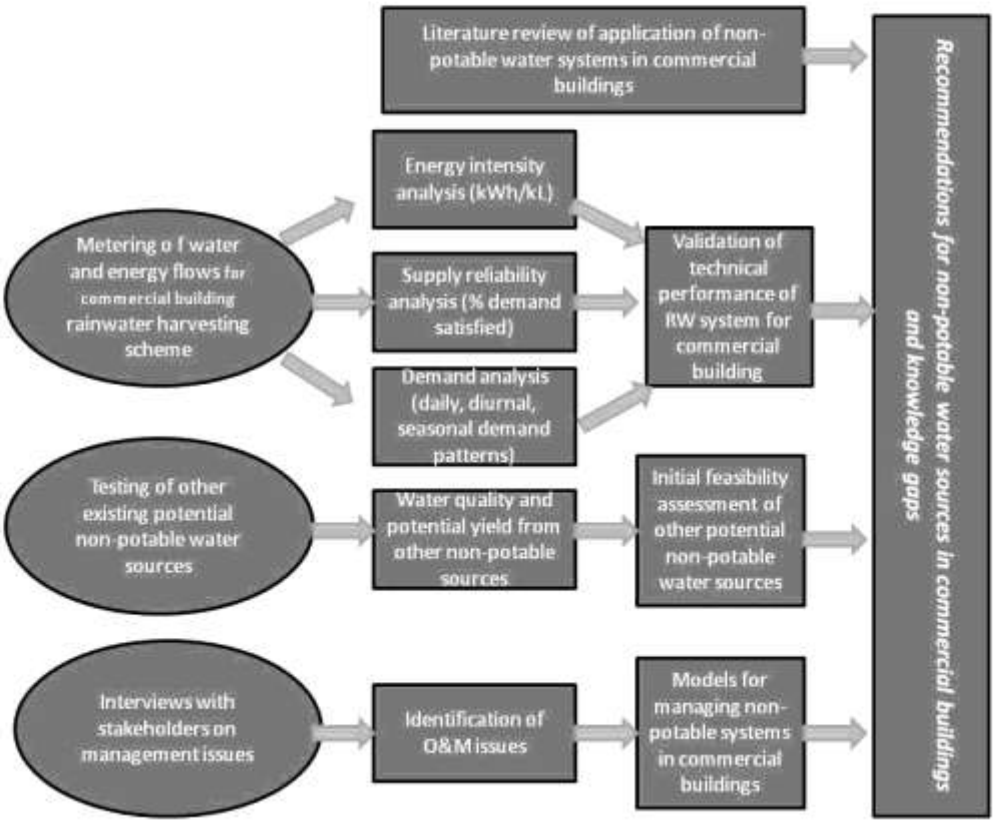

Figure 22: GSNT methodology for the investigation of non-potable water use in commercial buildings.

3.1.5. Climate

The historical climate data from the BoM weather station at the Brisbane Botanical Gardens, which is near GSNT, was analysed for a 30-year period. This showed that GSNT is located in a climate zone where the annual rainfall is around 1,000 mm. The inter-annual rainfall was characterised by a pattern of relatively wet years, around 1,500 mm, interspersed with drier years, with rainfall down to 600 mm. The rainfall distribution over a year was typical of a sub-tropical climate, with wetter periods over summer months and a drier period over the winter months. This inter-annual and intra-annual rainfall variability has implications for configuration of the system to maximise supply reliability, as well as assessing the potential for other non-potable sources to augment supply during periods of low rainfall or excess demand.

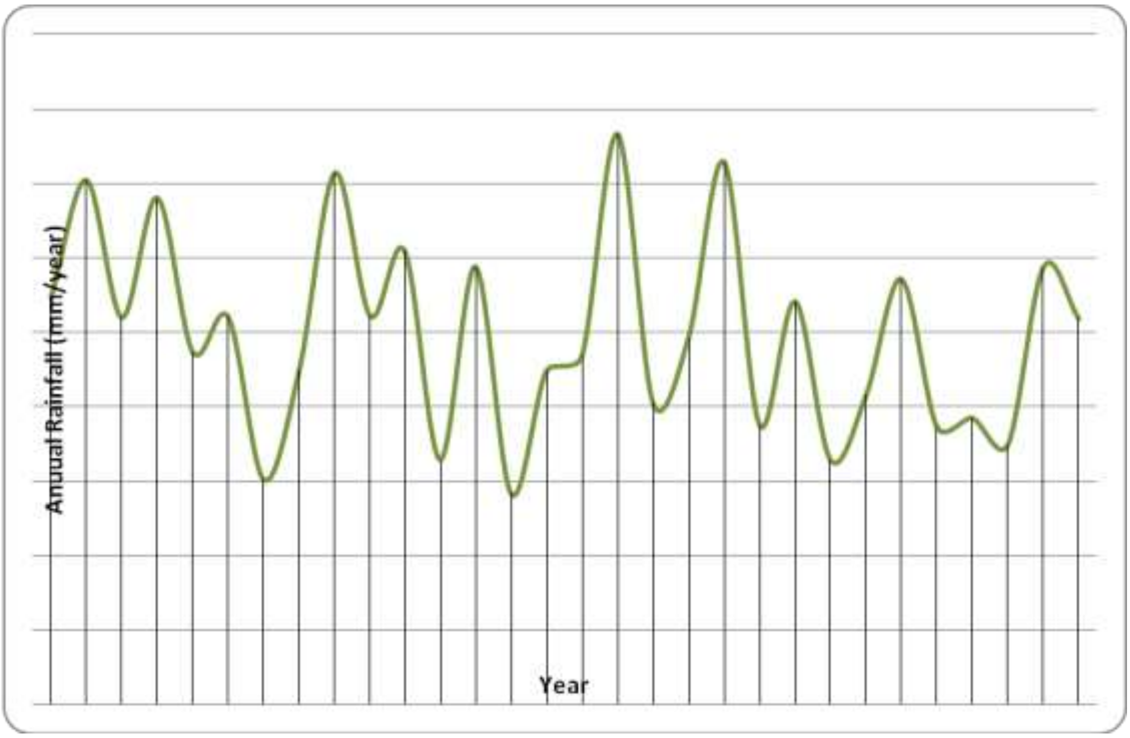

Figure 23: Annual rainfall for Brisbane Botanical Gardens (BoM Station 40215).

3.2. GSNT Monitoring Results

Table 13 summarises the performance of the GSNT rainwater system over the 26-month monitoring period from March 2010 to April 2012. This showed that the system could be characterised as having a fair to moderate level of reliability and low energy intensity. The daily demand for toilet flushing was around 7.8 kL per day. This equates to 54 litres per day for each of the 146 toilets or 15 flushes per day (3.5 litre average flush volume). There was minimal demand for planter box irrigation due to the limited potential application and also the relatively high rainfall over this period. The rainwater system provided 37% of the non-potable demand; the GSNT rainwater system has a storage capacity of 690 L per connected toilet compared to 1,500 L per connected toilet required under the QDC.

Table 13: Rainwater yield and energy use for GSNT rainwater scheme (monitoring period March 2010 to April 2012).

	Daily Average
Demand for toilet flushing	7.8 kL/day
Demand for irrigation planter boxes	0.7 kL/day
Overall demand for non-potable system	**8.5 kL/day**
Rainwater supplied from basement tank	3.2 kL/day
Mains water top-up for toilet flushing	5.3 kL/day
Specific Energy for Rainwater System	**Specific Energy**
Energy for rainwater system (pumping rainwater from basement tank)	0.44 kWh/kL

During the monitoring period, it was noticed that, at times when the basement storage tank was full, there was no or very irregular pumping from the basement to the header tank despite a relatively constant demand of around 8 kL per day. Investigations found a problem with a pressure switch that was not activating pumping from the basement when the level in the header tank fell to the trigger level. Figure 24 depicts hourly demand and supply data for the non-potable system over an eight-day period (first week of January 2012, which included a public holiday). This shows there was a reasonably consistent demand pattern over the working days. Despite the basement storage tank being full, no demand was being drawn from the basement tank to top-up the header tank that was drawn down. This pattern of no pumping from the basement tank was observable for many periods during the monitoring. Figure 25 depicts the daily flows from the basement tank, and rainfall depth. This highlights periods where, despite the daily (work day) demand for toilet flushing being relatively constant, there were extended periods where there was no pumping from the basement tank. This meant that, during these periods, supply was being met through mains water top-up to the header tank and demand for toilet flushing was being met by mains water.

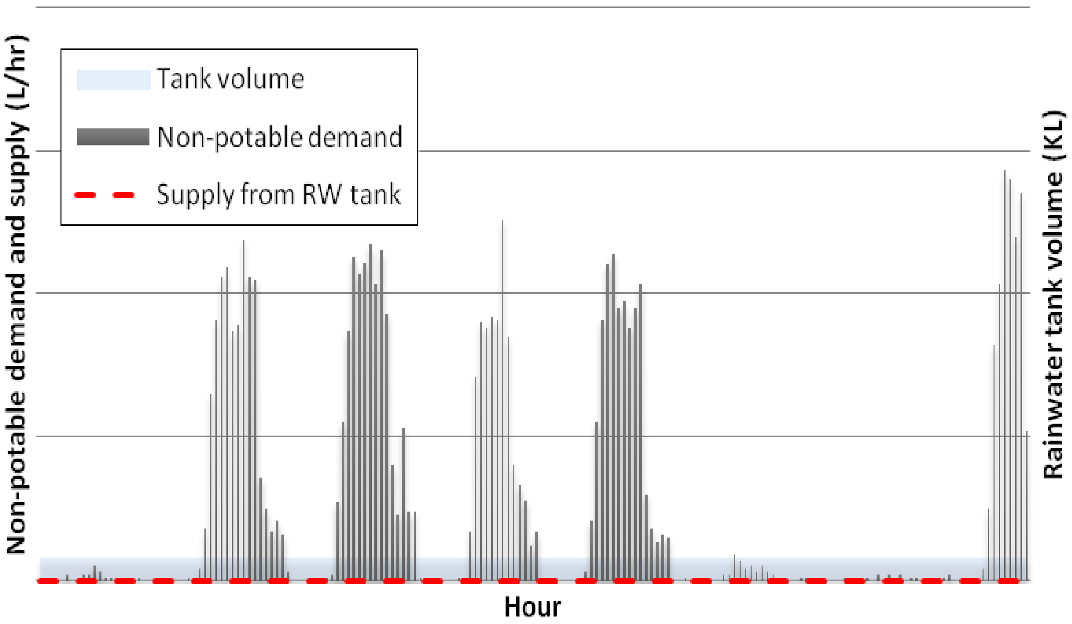

Figure 24: Toilet demand, pumping from rainwater basement tank and header tank water level over an 8-day period in January 2012.

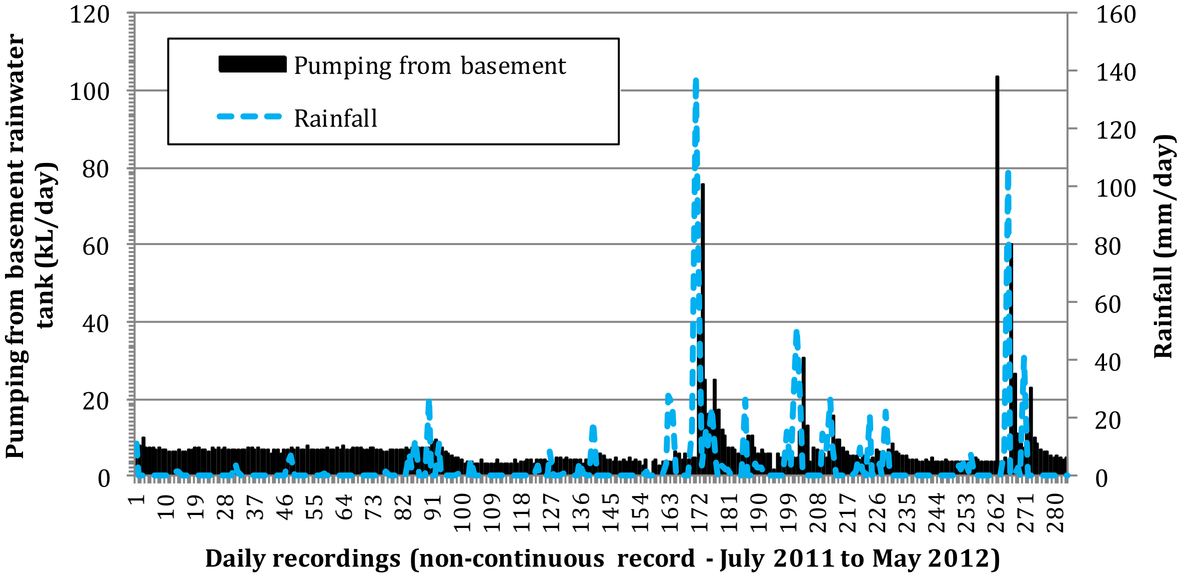

Figure 25: **GSNT pumping from rainwater basement tank and daily rainfall.**

Figure 26 depicts the average and maximum water demand for toilet flushing (weekends and public holidays excluded from the data). As expected, the demand pattern reflects the pattern expected in an average working day, with highest demand between 9 am and 4 pm. The maximum hourly demand was up to three times larger than the average hourly demand.

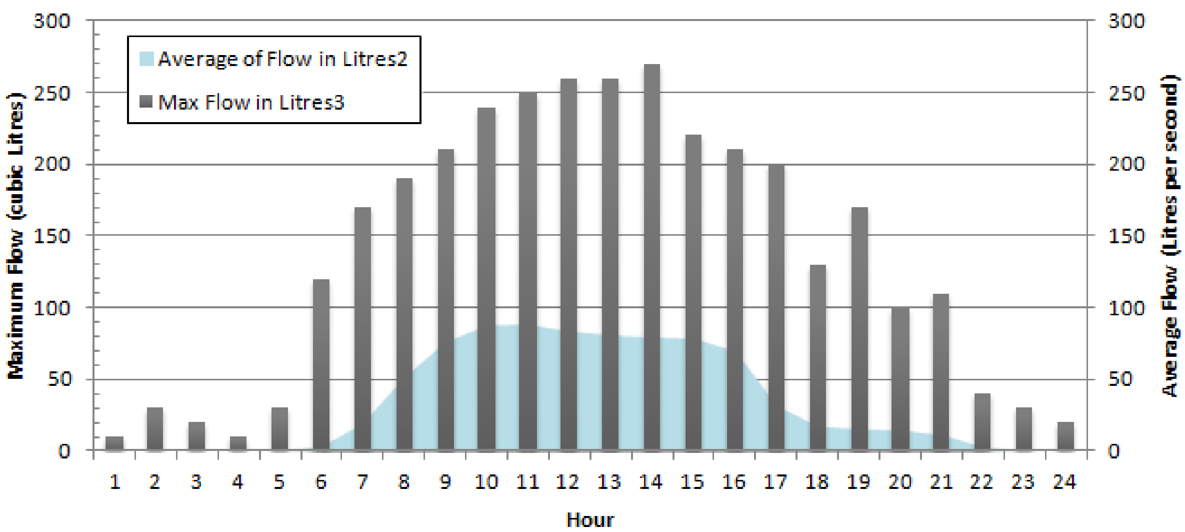

Figure 26: **Average and maximum (work days) toilet flushing water demand by hour during work days.**

3.3. Pump Efficiency Analysis for GSNT

Analysis of the monitoring data revealed that the GSNT rainwater system had a specific energy similar to the centralised system for Brisbane, with 0.44 kWh/kL of water supplied, which was much less than 3.2 kWh/kL observed for the CDM potable water supply pump. The following investigation was to determine if the 3,000 watt rainwater system pump at GSNT was configured for optimal efficiency. Despite start-up energy for rainwater pumping contributing significantly to overall energy demand of rainwater systems (Gardner *et al.*, 2006), in this study, start-up energy was not analysed as the data capture was too coarse. To analyse start-up energy would require a fine data resolution, such as 0.5 L/pulse. Therefore, the analysis undertakes a theoretical approach to exploring efficiency improvements.

Table 14: GSNT friction loss.

Parameters	Values	Equation Used
f, Friction Coefficient	0.025	$h_f = \frac{8fLQ^2}{\pi^2 g D^5}$
L, Pipe Length (m)	70m (56m + 25%)	
Q, Peak Flow (m^3/s)	$0.00225 m^3/s$ ($8.1 m^3/hr$)	
D, diameter of pipe (m)	0.1m	
g, gravitational acceleration (m/s^2)	$9.81 m/s^2$	

Friction loss, h_f, was estimated at 0.074 m, using the parameters and equation in Table 24. This value is considered to be very small.

To estimate a pump efficiency curve, total head loss is required and was estimated as per the parameters in Table 15.

Table 15: GSNT estimated total head loss.

Head Losses	Values
Friction Losses, m	0.074m
Elevation difference in pump and pipe outlet (max., approx.) *assuming 12 storeys + 2 basements + 2 levels from room to RW tank inflow; at 3.5m height each.	56m
Total Head Loss	**56.069m**

The monitoring data showed that peak flow for the GSNT rainwater systems was 8.1 m^3/hr. The required head used for pump efficiency curves was 56 metres. The pump efficiency curves were estimated using the online WebCaps application (available at: http://net.grundfos.com/Appl/WebCAPS/custom?&userid=GPA&lang=ENU).

Table 16 shows the estimated pump efficiency curves for the current GSNT pump configuration, while Table 17 and Table 18 explore the potential of alternative pump models to improve electrical efficiency of pumping at GSNT. While the pump at GSNT is appropriately sized for the peak flow and required head, a shift to alternative pump models that use less hydraulic power and deliver a lower pump head could reduce mains power demand for pumping at GSNT by 10 to 20% (Table 19). This highlights the need to ensure the pump is configured to meet the required head and flow rate, as the results demonstrate that matching the pump motor power to the site requirements is necessary to reduce mains power use.

Table 16: Estimated efficiency current GSNT pump configuration (3000 W).

	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., η_p	Shaft Eff., η_s	Overall Eff., η_o
Peak Flow	8.07	83.5	CR10-9	3000	1836	2770	3170	66.3%	87.4%	57.9%
Peak Flow (+20%)	9.68	75.4	CR10-9	3000	1990	2900	3330	68.6%	87.1%	59.8%

Table 17: Estimated efficiency for alternative pump 1.

	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., η_p	Shaft Eff., η_s	Overall Eff., η_o
Peak Flow	8.07	74.9	CR10-8	3000	1647	2480	2830	66.4%	87.6%	58.2%
Peak Flow (+20%)	9.68	67.6	CR10-8	3000	1783	2610	2980	68.3%	87.6%	59.8%

Table 18: Estimated efficiency for alternative pump 2.

	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., η_p	Shaft Eff., η_s	Overall Eff., η_o
Peak Flow	8.07	66.1	CR10-7	3000	1454	2190	2500	66.4%	87.6%	58.1%
Peak Flow (+20%)	9.68	59.8	CR10-7	3000	1577	2300	2620	68.6%	87.8%	60.2%

Table 19: Estimated theoretical power savings from alternative pump configurations at GSNT.

L/5min	m^3/hr	Installed Pump's Mains Power, Wh	Alternative Pump's Mains Power (CR10-8), Wh	% difference (Mains Power)	Alternative Pump's Mains Power (CR10-7), Wh	% difference (Mains Power)
133.7	1.34	1470	1320	10.2%	1180	19.7%
160.4	1.60	1540	1390	9.7%	1230	20.1%
801.8	8.02	3170	2830	10.7%	2500	21.1%
962.1	9.62	3330	2980	10.5%	2620	21.3%
			Average	**10.3%**	**Average**	**20.6%**

3.3.1. Specific Energy GSNT Pump

The pump at GSNT is a fixed speed pump and has settings for two flow, with averages of 8 m^3/hr and 1.34 m^3/hr based on the analysis carried out on the pump running flows. Specific energy was analysed by manually choosing data for when there was a continous running of the pump. The pump does not have a fixed time when it operates as it is (nominally) controlled by the level of the rooftop storage tank. Trends are as expected; higher flow rates have the lower specific energy (0.39 kWh/kL), compared to the lower flow rates with a specific energy of 0.55 kWh/kL. Figure 27 shows the specific energy-flow rate plot; flows are concentrated on either sides of the graph due to the two speed pump that had a high and low flow rate.

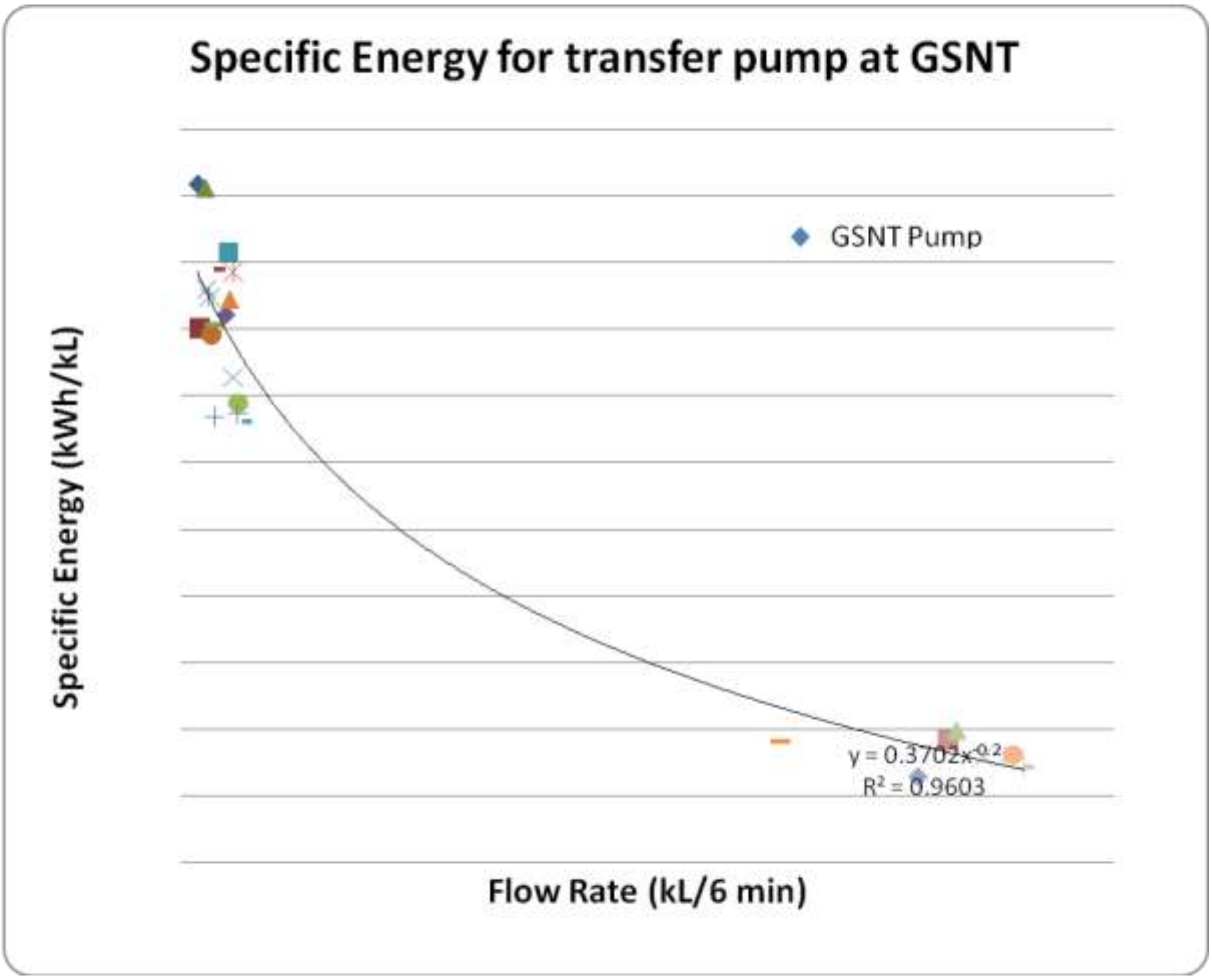

Figure 27: GSNT specific energy.

3.4. GSNT Hydraulic Analysis

3.4.1. Model Set-Up

The hydraulic analysis of the collection system in GSNT was carried out using a similar approach to Capo di Monte (CDM) (detailed in Section 2 and Appendix B), using both SWMM and the Rational Method, and then comparing the results of the two. Hydraulic modelling of the GSNT rainwater storage system was a straightforward task in SWMM as there were no complexities of a pipe network coupled with a number of sub-catchment areas (roofs) as was the case in CDM.

The main input into the model was rainfall data obtained from a rain gauge station sited on the top of the roof, which recorded data for every pulse (in this case 2 mm) of rainfall. The roof properties of GSNT for input into the SWMM model were as follows:

- Measured roof area was 1620 m^2 using GIS
- Width of roof is approximately 50 m
- Slope is shallow and estimated to be 10%
- Roof is made from impermeable corrugated metal, hence, imperviousness is 100%
- Manning's roughness for corrugated metal (roof material) is 0.022.

A holding tank, located at the basement of the building, was simulated using a storage unit in the model, with depth data obtained from a data logger.

Pipe length was not considered to be significant in affecting model results as flow was assumed to be conveyed instantaneously into the holding tank due to the 12-storey vertical drop from the roof into the tank. The drop in height, from roof top to basement, was approximated to be 55 meters, based on 3.5 meters for each of the 12 storeys with two basements and roof top plant room (total height equivalent to 16 storeys).

3.4.2. Calibration and Validation Results

Model calibration was carried out using a similar method to CDM with a chosen rainfall event captured and stored fully in the storage tank at the basement, i.e. no extraction from either the pumping system or the pressure vessels. Recorded tank levels were used as a means of calibrating the model, with validation runs carried out to ensure confidence in the model.

When the model was operated with 100% of roof area, results showed higher predicted depths of water in the tank than the observed data. Hence, calibration involved reducing roof sizes until modelled results matched observed data for the simulated rainfall event. This reduction in roof area provides the approximate runoff coefficient (in percentage terms), as 100% of the roof is assumed to be connected to the collection tank. Rainfall events chosen for the calibration and validation runs are shown in Table 20.

Table 20: Rainfall events used for calibration and validation purposes in GSNT.

Date	Investigated Time		Duration	Purpose	Peak Rainfall Intensity
	From	To	(h:mm)		mm/6 min
3-Oct	12:48 PM	3:24 PM	2:36	Calibration	1.8
21-Aug	8:12 AM	10:24 AM	2:12	Validation	0.6
30-Aug	3:48 PM	8:30 PM	4:42	Validation	1.0
8-Oct	7:36 AM	10:18 AM	2:42	Validation	1.4
13-Oct	4:12 PM	7:42 PM	3:30	Validation	5.6
9-Sep	12:00 PM	2:12 PM	2:12	Validation	1.2

Calibration of the model was carried out with 90%, 92.5% and 95% of the roof area and provided a close match to observed data (Figure 28). Results from validation runs supported these values. Figures 29 and 30 shows the results of some of the model outputs in comparison to the monitored data.

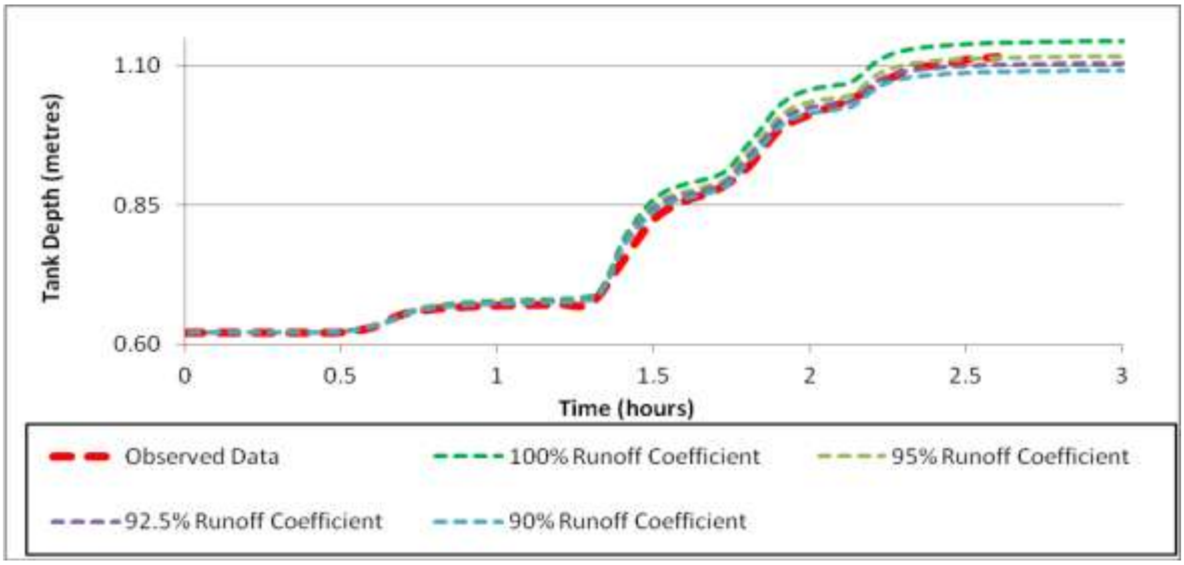

Figure 28: Calibration results from hydraulic model (Oct 3rd 2011).

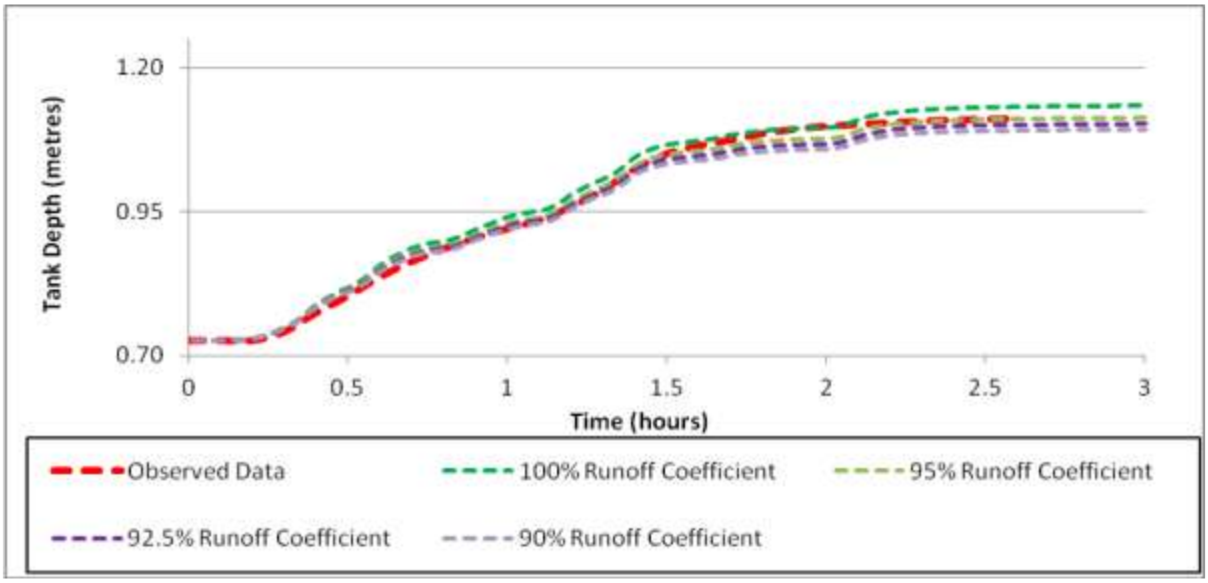

Figure 29: **Validation results from hydraulic model (Aug 21st 2011).**

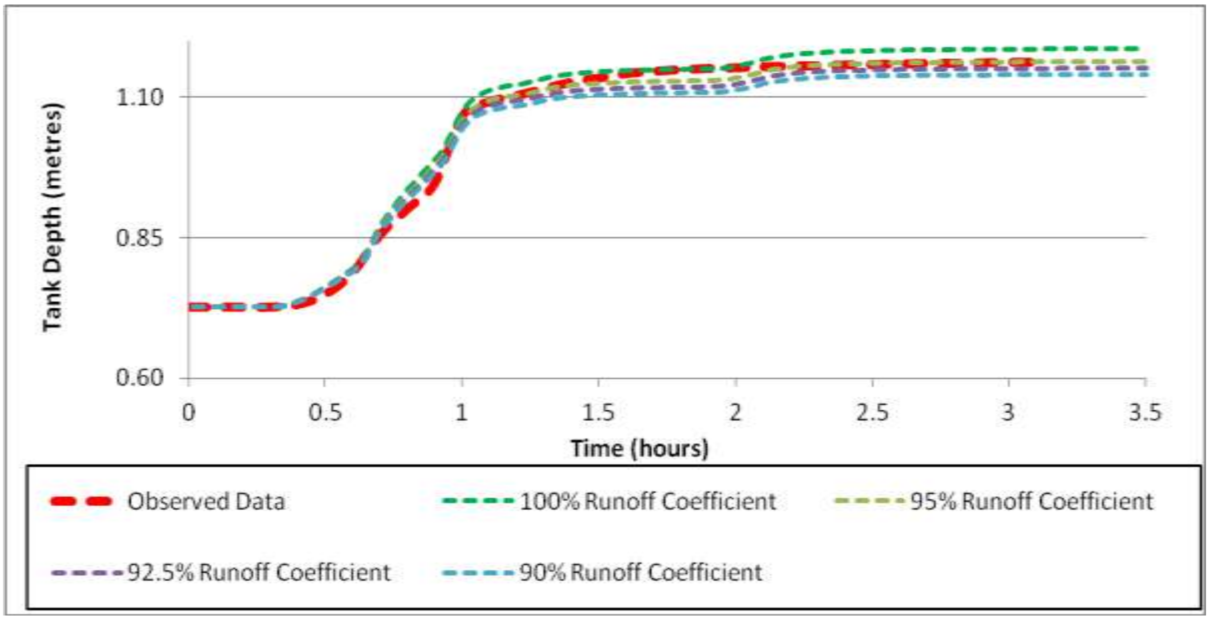

Figure 30: **Validation results from hydraulic model (Sept 9th 2011).**

3.4.3. Rational Method

The rational method was described in the earlier section in CDM and relates the captured rainfall volume as a direct function of the roof area, runoff coefficient and total rainfall depth. The total volumes in the storage tank are plotted against the respective total rainfall for each rainfall event to estimate the approximate runoff coefficient, as shown in Figure 31. From the linear regression of the plotted data, the effective roof area (roof area x runoff coefficient) is approximately 1,470 m^2, representing 91% of the total measured roof area of 1,620 m^2. This rational method further confirms the runoff coefficient values of 90% to 95% obtained from the hydraulic simulation.

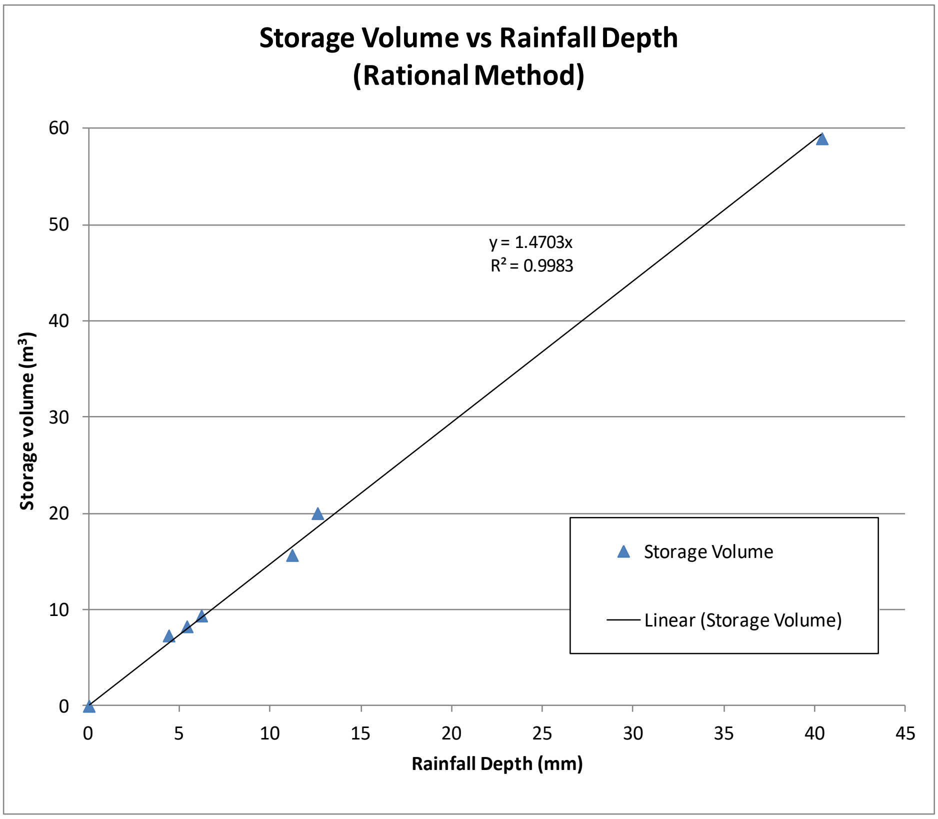

Figure 31: Regression analysis of the Rational Method for GSNT (storage volume vs. rainfall depth).

3.4.4. GSNT Hydraulic Modelling Conclusions

The analysis for GSNT showed that the runoff coefficient values obtained from the rational method matched well with that obtained from the hydraulic models. The rational method estimates values on the low side of the hydraulic simulation runs for GSNT. Runoff coefficients from the rational method were determined to be 91% against 90% – 95% from the SWMM method. The Rational method gives a conservative estimate on the potential amount of water that can be harvested from a rainwater collection system and the more accurate method would be to simulate the capture of runoff using a hydraulic model. Nevertheless, the preliminary design of a rainwater collection system could incorporate the use of the rational method to estimate potential captured volumes.

The efficiency of rainfall capture can be reduced due to losses on the roof edge due to a combination of spillage and wind effects. GSNT has a low ratio of roof area to perimeter, with a value of approximately 1 to 0.125. This means that for every square meter of roof there is approximately 0.125 m in roof perimeter for the losses to occur. An analysis of the effects of this ratio suggests that a higher ratio would result in a lower runoff coefficient, although validation is required to confirm this. This indicates that a single large area of roof is able to capture more runoff than a similar sized roof area distributed over smaller buildings. This insight could be used as a guideline when choosing runoff coefficients for estimating the captured rainfall volume for the different type of rainwater harvesting systems, i.e. individual, communal or commercial.

A runoff coefficient value of 0.9 (90%) is recommended when applying the Rational Method to estimate the potential capture volume for commercial buildings with large roof areas.

3.5. Water Balance Modelling

Water balance modelling was undertaken to explore the theoretical reliability of the rainwater harvesting system at GSNT under different operational configurations and its resilience to drier years. The modelling was under taken using the Urban Volume and Quality (UVQ) as applied in the CDM water balance analysis. The connected (effective) roof area applied the 1470 m^2 identified through the hydraulic analysis, while demand was based on the monitored demand for the non-potable system of 8.5 kL/day. The GSNT monitoring shows that irrigation demand was negligible, therefore this analysis focuses on the water balance associated with meeting toilet demand. The active storage comprises the 80 kL tank in the basement and the 40 kL tank on the roof, for a total of 120 kL.

Daily rainfall for the period 1980 to 2009 was used for the water balance modelling. The rainfall data was supplied from the Bureau of Meteorology from the Brisbane Botanical Gardens weather station, which is located near the Green Square North Tower development. The average annual rainfall for the period modelled was 1,000 mm.

The results are presented as annual averages over the simulation period of 30 years to account for both seasonal and yearly fluctuations in rainfall. The simulated reliability of the GSNT rainwater system in meeting daily demand for toilet flushing was 37%. This agreed well with the actual reliability of the system based on monitoring data, which was surprising given the operational problems faced by the GSNT system over the monitoring period. This could be explained by the relatively wet period that was experienced during the monitoring period, with above average rainfall in 2010 and 2011. Also, it may suggest that the daily time step used in the UVQ model may over estimated rainwater yield, as it does not account for losses during high intensity rainfall events of short duration.

Figure 32 shows that significant increases in storage sizes only demonstrated relatively marginal increases in reliability. A doubling of the active storage to 240 kilolitres only increased the reliability from 37% to 41%percent. However, Figure 33 shows that increasing the effective roof area had a significant impact on increasing the reliability, indicating that the effective roof area is the limiting factor for yield from this system.

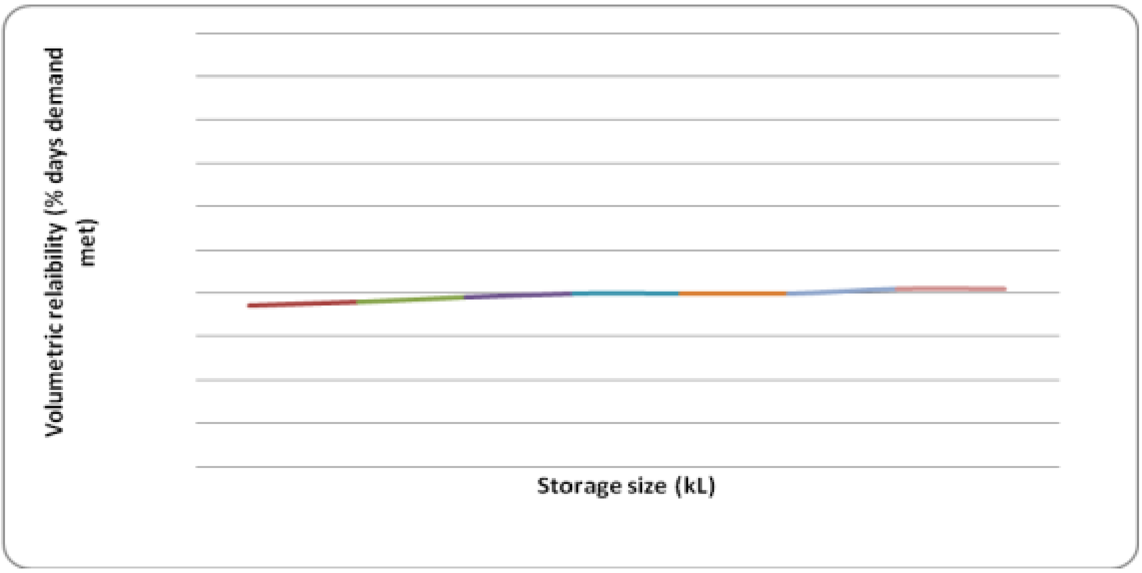

Figure 32: Impact of storage size on reliability of supply for toilet flushing at GSNT.

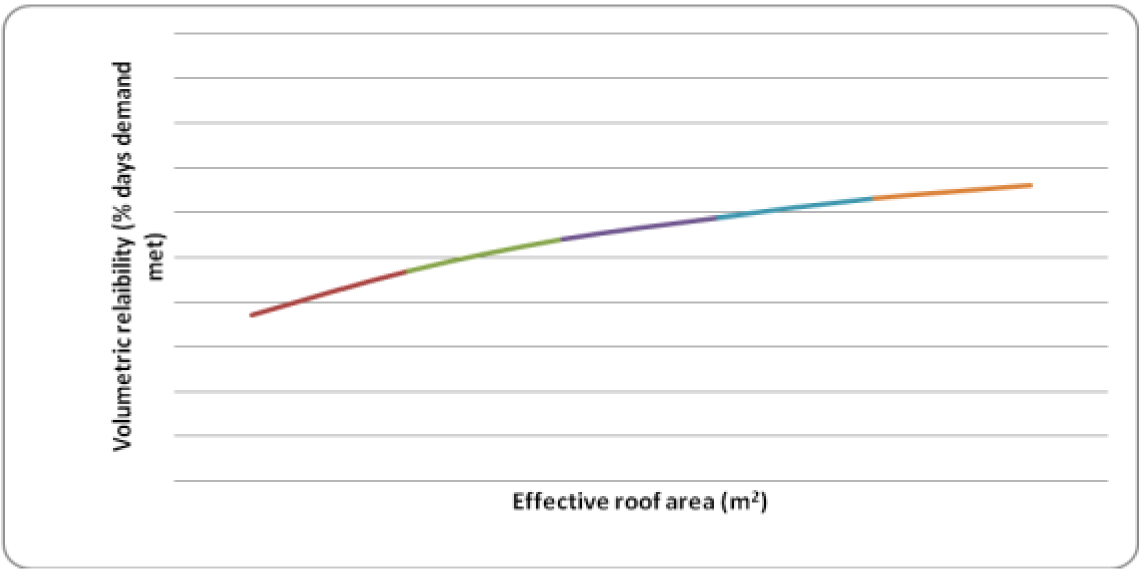

Figure 33: Impact of effective roof area on reliability of supply for toilet flushing at GSNT.

3.6. Alternative Sources at GSNT

Two other non-potable water sources were also investigated to determine their potential to increase the supply reliability of the non-potable system at GSNT. The two sources were: groundwater flows to a wet well in the basement that was then pumped to stormwater system, and condensate from air handing units that was part of the air conditioning system for GSNT (see Figure 21).

3.6.1. Condensate from Air Handing Units

Air handling units (AHU) produce condensate water as part of their normal operations, which typically drain to municipal sewer systems. Condensate water has been used in industrial and commercial buildings as it provides a relatively good quality non-potable water source with no additional treatment required (i.e., no energy requirements). Examples include hospitals (EPA, 2007) and for reuse in cooling towers. Condensate production is a function of the temperature and relative humidity of the outside air, relative to the design temperature and humidity in the building, and the amount of cooling provided.

The potential condensate that could be captured for non-potable reuse at GSNT was based on the following equation (Lawrence *et al.*, 2010):

$$q_{cond} = q_{air} \, dw_{lb} / v_{da}$$

Where:

q_{cond} is condensate generated in litres per minute

q_{air} is air flow in cubic metres per minute

dw_{lb} is the difference is specific humidity is the ration of water vapour to dry air (kg/kg)

v_{da} is the specific density of air in kilograms per cubic metre

GSNT has six AHUs with a combined capacity of 234 kW. Discussions with building managers revealed that the AHUs operate on working days for approximately ten hours. Time-proportional water quantity sampling was undertaken to validate the potential condensate yield. The five-minute tests were done in October when the outside temperature was 25°C and a relative humidity of 75%. The 5-minute sampling returned condensate yields of between 6 and 11 L/min. At this rate of condensate generation, around 6.5 kL of condensate could be generated per day. However, as the yield is dependent upon changes in outside temperature and humidity, there was a need to consider daily and seasonal variation in the likely condensate yield. Figure 34 depicts the mean monthly temperature and relative humidity for Brisbane.

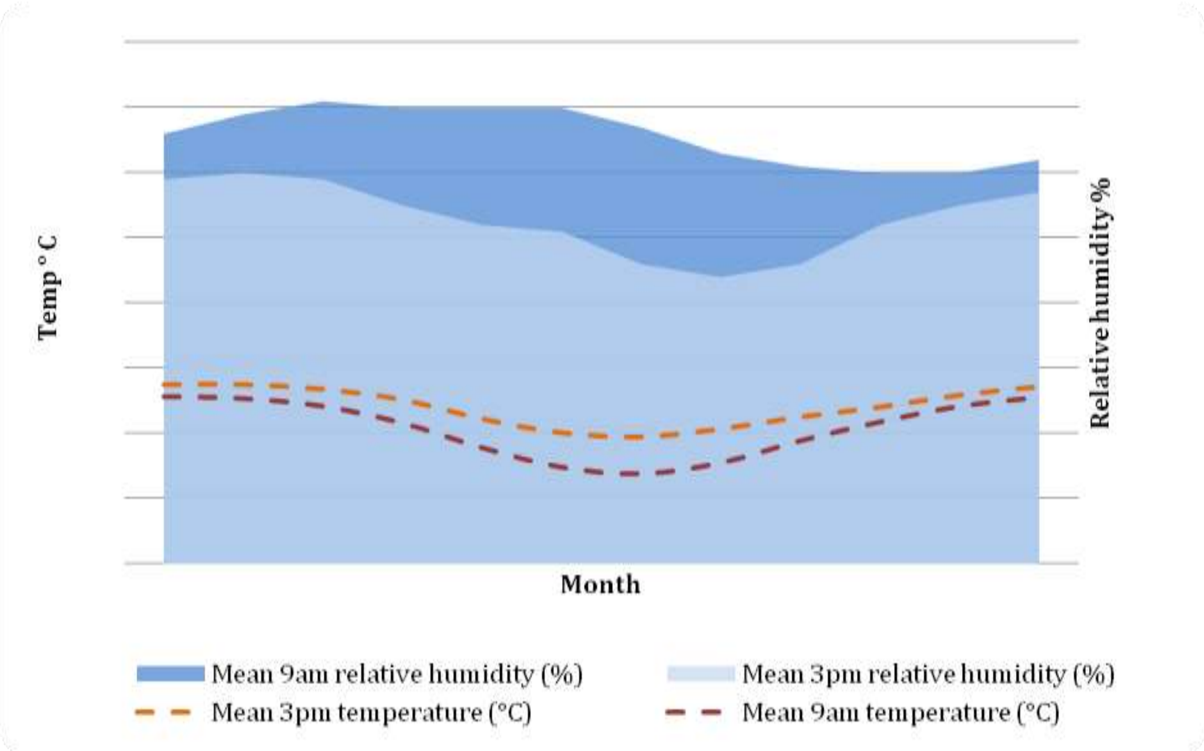

Figure 34: Brisbane: Mean 9 am and 3 pm temperature and relative humidity.

The following assumptions were made on the operation of AHUs at GSNT in estimating the rate of condensate at different times of the year: operating from 8 am to 6 pm for 220 days per year; air delivered to offices at a temperature of 21°C and a relative humidity of 40%. These values were based on the *ASHRAE Standard 55 - 2010 Thermal Environmental Conditions for Human Occupancy*. It was also assumed 20% of the air passing through the AHUs was outdoor air, with the remainder recirculating air.

Figure 35 depicts the theoretical condensate rate for GSNT based on mean temperature and relative humidity at 9 am and 3 pm. This showed that there was little diurnal variation in the condensate rate between 9 am and 3 pm conditions for any month, but significant variation in the average monthly condensate rates. The analysis showed that for Brisbane conditions, the potential yield from condensate would be negligible during the cooler winter months (approx. 270 L/day in July) but could be very significant during the warmer summer months (approx. 4200 L/day in January).

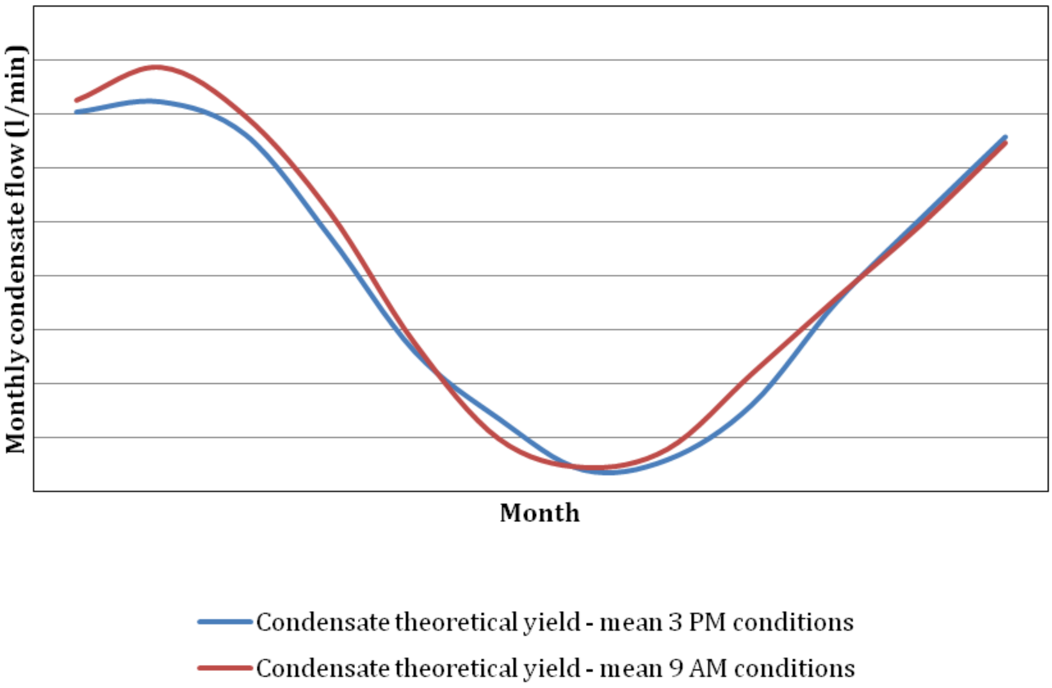

Figure 35: Theoretical monthly condensate rate at GSNT.

3.6.2. Reuse of Groundwater Inflow

Groundwater controls are often needed both during the construction of high rise buildings and during their operating life to manage the impact of groundwater inflows on the stability of soils. In particular, dewatering is often required when the foundations of building extend into the saturated soil zone.

GSNT was built on *Brisbane Tuff* geological formation which was formed during the late Triassic and is comprised of welded tuff, sandstone with shales and conglomerate near the base of the unit (Queensland Government, 2008). Groundwater flows occur through defects in the bedrock, such as fractures, joints and bedding planes. The groundwater depths and flows through this unit are highly variable as they are governed by the nature of the defects. The depth to groundwater in this geological formation near GSNT, have been found to vary significantly within a relatively small area, with field investigations at nearby site finding that groundwater table depth ranged from 1 to 11 metres (Queensland Government, 2008). It is also considered likely that groundwater inflows in this area are influenced by tides due to the proximity of the Brisbane River (Queensland Government, 2008).

Groundwater inflow to the basement at GSNT is collected in a wet well and then discharged into the municipal stormwater system. Flows to the wet well also include overflows from the rainwater tank during high rainfall events. Pumping from the wet well was monitored over a year with data collected at 6-minute time intervals. However, there was an 11-day period when the data logging system was not operational so the monitoring data is non-continuous. Figure 36 depicts the daily volume pumped to stormwater, which shows a steady 5 to 6 kL/d. Even when there had been no rainfall over the antecedent 20 days, the volume pumped from the wet well was consistently more than 5 kL a day (Figure 36). This indicates the contribution of groundwater inflows (non-rainfall sources) to water pumped to the stormwater system. However, there were also periods of very high pumping from the wet well which were associated with extreme rainfall events, as shown in Figure 36.

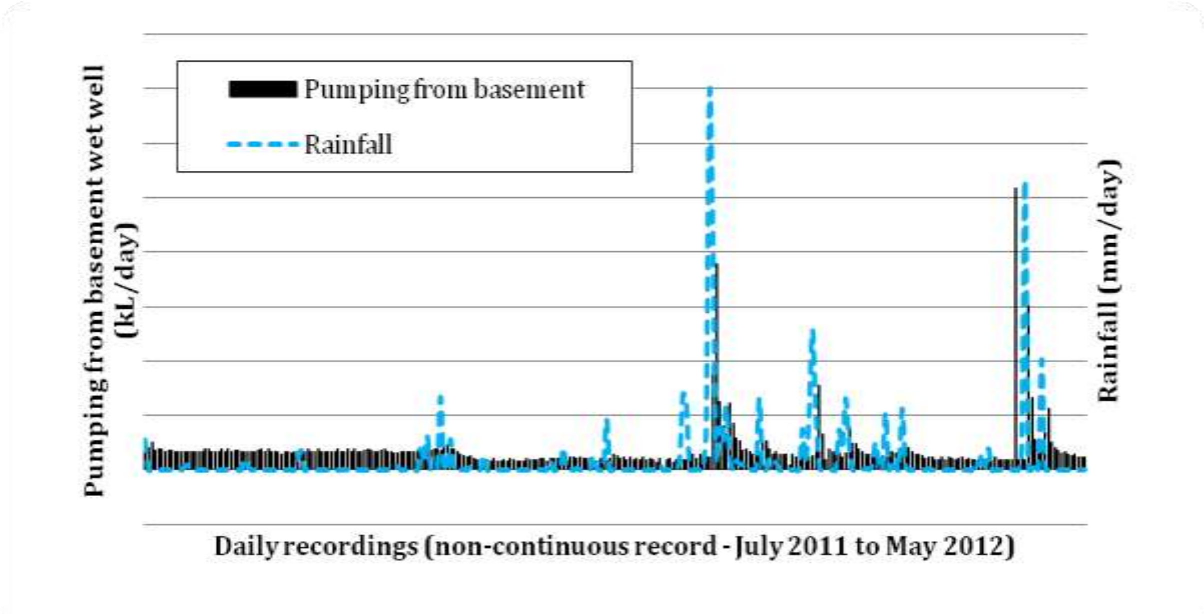

Figure 36: Daily volume of pumping from wet well to stormwater.

3.6.3. Water Quality of Alternative Sources

Table 21 compares the water quality of the alternative water sources available to the GSNT building. The AHU condensate and rainwater both provide a high quality of water source that is suitable, without prior treatment, as non-potable water sources for toilet flushing. However, the groundwater inflow can be classified as hard, which may lead to potential problems with scaling in metallic pipe network. This may limit the potential application of untreated groundwater at GSNT, and indicates that groundwater quality testing would need to be the first step in considering its potential use in commercial buildings if the water is to be distributed using metallic pipes. While there was some evidence of microbiological contamination of the groundwater inflow and harvested rainwater, the fact that intended use is only for toilet flushing means the risk exposure is low.

Table 21: Water quality of alternative water sources for toilet flushing at GSNT.

Water Quality Parameter	Air Handing Unit Condensate	Groundwater Inflow	Rainwater
pH	6.9	8.5	6.9
Total dissolved ions (mg/L)	-	332	44
Total hardness as Ca CO_3 mg/L	< 1	136	16
Conductivity µS/cm	12	495	77
Lead (mg/L)	<0.2	<0.2	<0.2
Total Nitrogen (mg/L)	0.67	3.05	0.5
E. coli (cells 100 ml plate)	1	-	-
Entero media (cells 10 ml plate)	0	124 - 147	61 - 87

3.6.4. Discussion

The results have demonstrated that there are potential non-potable water sources that may be used to reduce demand from mains water that are often overlooked in favour of more traditional rainwater and recycled water. The theoretical yields of condensate from AHUs demonstrated that this could be a useful seasonal non-potable water source during warmer months. Hence this source is only suited to relatively hot and humid climates in commercial buildings that have a matching non-potable demand. For the Brisbane climate, the potential reliability of condensate for non seasonal toilet flushing at

GSNT was limited during winter months, with a supply of less than 5% of demand. The analysis of the groundwater inflows showed that a median of 6 kL per day was pumped to stormwater, which represents around 75% of the daily demand for toilet flushing (Table 13). However, the hardness of the water limits its application due to potential scaling to metallic pipes and plant. Plastic pipes could be used in non-potable systems to avoid the corrosion or scaling problems associated with metal pipes.

3.7. Management of Non-Potable Water Sources in Commercial Buildings

While the monitoring of the GSNT rainwater system showed that only moderate supply reliability, this was achieved under extenuating circumstance, which is explored in the following text. In undertaking the monitoring study, a number of issues were identified with the system both from a failure for the constructed system to meet design intent, as well as technical difficulties that were not identified early and therefore persisted over an extended period of time. The following is based on the authors' observations and discussions with key stakeholders including: the engineering consultants involved in the design and implementation; representatives of the building owner – a listed property trust based in another city; and the facilities manager, who was responsible for the day to day running of plant at GSNT and organising any contractors for O&M tasks. The purpose of the study was to explore approaches that may facilitate rapid identification and rectification of commonplace equipment failure, such as pump problems, and also to highlight the need for monitoring and validation post-commissioning to ensure that the performance of the decentralised system is meeting the supply targets defined in the design stage.

The system at GSNT experienced ongoing problems with the pressure switch that activated pumping from the basement tank. When the level in the toilet header tank fell to the low mark, the pressure drop did not trigger the basement pump. This meant for some months the toilet tank was being supplied with mains water, even though the basement tank was full. The system fault was identified through the analysis of the monitoring data, and then communicated to the building facilities manager, owner, and contractors who rectified the fault. The building manager noted that he had no way to monitor if the rainwater or mains water was being used for toilet flushing without physical inspection of the tank levels. A simple monitoring system could be a pressure transducer to measure and display tank water levels, which is incorporated into the existing building management system. This type of monitoring would be able to highlight the system fault of a full rainwater storage but with mains water being used to meet non-potable demand. Given the capital investment in alternative water systems to meet building sustainability benchmarks, the additional small investment to ensure the system is operating correctly by electronic monitoring would be justified.

The design drawings of the non-potable system at GSNT indicated that other non-potable water sources were being captured to augment supply to the basement tank. These other water sources were air conditioning condensate, fire test water, and cooling tower blowdown. It was the understanding of the building manager and the researchers that these sources were being used. However, the monitoring results contradicted this assumption, as there were no inflows to the basement tank during extended periods of no rainfall. Discussions with the engineering firms involved in the construction phase of the development revealed that original design intent was not followed through due to uncertainties in water quality. Instead, these sources were discharged to wastewater. This again highlights the need for post-implementation validation of decentralised water systems to compare the operating performance of the as-built system against design intent. This is particularly important to determine if the system is delivering against the sustainability objectives intended in the design.

In considering the role of alternative water sources in improving the sustainability of commercial buildings, there is the need to undertake an integrated assessment that compares augmentation with demand side measures, as well as considers the implications across social, economic and environmental objectives. The assessment of potential alternative water supply in commercial buildings needs to move beyond technical feasibility and theoretical reliability to take into account the specific context of the building in terms of local water source opportunities and the associated management complexities in maintaining and operating these systems.

The technical skills and management burden needs to be aligned with the expectations of the owners and the capacity of the building managers. A prescriptive approach to meeting sustainability objectives for decentralised water use in commercial buildings impedes flexibility in meeting sustainability performance benchmarks that considers local water source opportunities and the legitimacy of the approach for building owners and managers. Some options, such as wastewater recycling, may offers good theoretical reliability in meeting non-potable demand in commercial buildings. However, the associated management burden and complexities of maintaining and operating these systems may result in inadequate O&M that is likely to increase failure rates. Figure 37 presents potential criteria that could be applied in evaluating potential decentralised water sources in commercial buildings that accounts for both technical feasibility and aptness of the management burden.

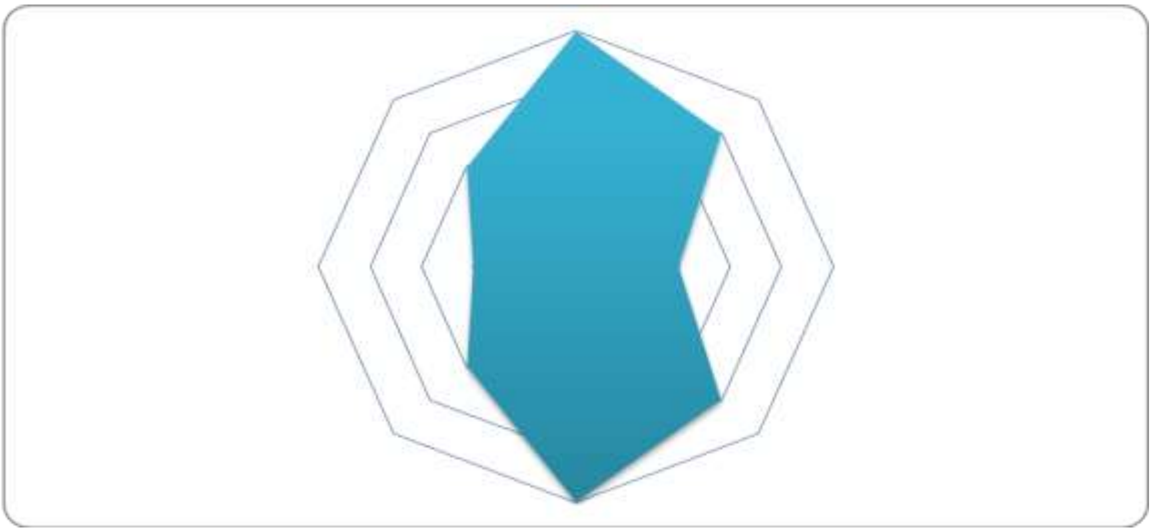

Figure 37: Example of suitability assessment criteria for non-potable water sources in commercial buildings.

3.8. Assessment against QDC MP 4.3

This study found that rainwater system provided 37% of the non-potable demand. This is due in part to a storage capacity of 690 L per connected toilet compared to 1,500 litres per connected toilet specified in MP4.3. However, water balance modelling indicated that effective roof area was the limiting factor for system yield (Figure 33). QDC MP 4.3 recommends a connected roof area of 50 m^2 per connected toilet. GSNT nearly makes full use of available roof area, with around 90% of the 1,600 m^2 roof area draining to the rainwater tank. However, to provide 50 m^2 for each of the 147 toilets would require an effective roof area of 7,350 m^2. Providing this roof area ratio in multi-level commercial buildings is highly unlikely.

3.9. Conclusions and Recommendations

Commercial buildings contribute significantly to the ecological footprint of urban areas so there is the need to consider opportunities for improved sustainability in this setting through more efficient use of water, energy and materials. The benchmarking of sustainability performance and building codes, such QDC MP 4.3, has provided the impetus for many commercial buildings to implement decentralised water servicing options to achieve best practice standards. However, there remains a paucity of monitoring studies to validate the performance of these systems against design objectives.

The GSNT monitoring results demonstrated that the system has provided moderate reliability with minimal energy requirements, with system yield impeded by construction and operational faults. The modelling also showed that effective roof area is the constraining factor in improving the reliability of rainwater harvesting for toilet flushing in commercial buildings. High-rise commercial buildings have a low ratio of available roof area for each potential connected toilet. This highlights the need to consider other potential sources of non-portable water that can reduce mains water demand. The investigation of other non-potable water sources for GSNT found that in hot, humid environments, the condensate from cooling systems could contribute a significant proportion of non-potable demand. In climates where there is considerable seasonal fluctuations, condensate capture may only be suited to a secondary, or supplementary, water source to reduce pressure on freshwater supplied from natural catchments. Groundwater inflows are another potential water sources in commercial buildings. The feasibility of this option needs to be considered on a site specific basis, as the quality and potential yield is dependent upon the proximal aquifer and the interaction with building footings.

It is suggested that incorporation of monitoring of decentralised water systems into the building management system in commercial buildings would enable the early identification of faults and their rectification.

APPENDIX A - Technical Specifications of Monitoring Equipment at Capo di Monte

Type	Description	Manufacturer	Specifications
Energy	Sand Filter	ANDELI	ADM 25 S 230VAC 5(30)A 50 Hz
	UV Rainwater	ANDELI	ADM 25 S 230VAC 5(30)A 50 Hz
	Bore Pump	ANDELI	ADM 25 S 230VAC 5(30)A 50 Hz
	Treated Rainwater Pump	ANDELI	ADM 100T AC240/415V 10(60)A 50Hz
Flow	Irrigation	Elster	50mm mechanical meter
	Bore-to-RainTank TopUp	Elster	20mm mechanical meter
	RainTank-to-Irrigation	Elster	20mm mechanical meter
	Pressure Transducer MainTank	GreenSpan	PS1000
	Pressure Transducer Buffer Tank	GreenSpan	PS1000
	Treated Rainwater	Trimec Industries	Model No:RT121DOFM Serial No:05371118
	Treated Recycled Water	Trimec Industries	Model No:RT121DOFM Serial No:05371118
	Tipping Bucket Raingauge	Hydrological Services	2 mm tipping bucket

Capo di Monte

Data Storage: Data stored automatically in FTP server (remote)
Data Logger
Model: Campbell Scientific CR1000
Data Interval: 5minute and Daily
Rain Gauge
Model: Hydrological Services TB3 – 2mm/tip
Logger: Mini Log Data Logger ML1
Data Interval: 5minute and Daily

Manual meter readings recorded once a month

GSNT

Data Storage: Manually download data from loggers (on-site)
Data Logger
Model: Campbell Scientific CR800
Data Interval: 6minutes, Hourly, Daily
Software used to download data: LoggerNet 3.4.1
Rain Gauge
Model: Hydrological Services TB3 – 2mm/tip
Logger: Mini Log Data Logger ML1
Data Interval: no fixed intervals, records data with every tip (i.e. 2mm) of rainfall
Software used to download data: WinComLog Rev 2.58

Manual meter readings recorded once a month

APPENDIX B - Set Up of Hydraulic Model at Capo di Monte

Pipe Properties

Geometrical properties of pipes, mainly, their sizes, lengths and roughness, are an essential component in hydraulic analysis. It is important that good and accurate data set are obtained to carry out the hydraulic study effectively and efficiently.

AutoCad engineering drawings of the pipe network system of the development were provided by the consultant (Bligh Tanner) and included pipe diameters and approximate locations of downpipe entry points.

A field survey using a dumpy level was carried out for several sections of the pipe network to obtain the slopes of the pipes. The survey points were chosen where there was an obvious change in the gradient of the ground so that an average slope could be used for the selected section of the pipe.

In all pipe flow calculations, SWMM uses the Manning equation to express the relationship between flow rate (Q), cross-sectional area (A), hydraulic radius (R), and slope (S), (Rossman, 2010). This equation is given as,

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

Where:

> n is the Manning roughness coefficient. The roughness coefficient of the pipe was determined by the material used, with higher values indicating a rough surface. Unplasticised Polyvinyl Chloride (uPVC) was used as the material for the pipes in the network. The recommended uPVC values in Australia are 0.011 by the Queensland Urban Design Manual (QUDM) and 0.008 – 0.009 by the Concrete Pipe Association of Australasia (CPAA). For the purpose of the analysis, a mid figure of 0.010 was chosen as the roughness for the pipes.

Hydraulic radius is the measure of flow efficiency in a pipe and is defined as the ratio of the cross sectional area of flow to the wetted perimeter. For a conduit of circular cross section, flowing at full capacity, the hydraulic radius is 1/4 of the pipe diameter.

Roof Properties

The roofs were modelled as sub-catchments in the hydraulic model with the relevant input parameters, such as area, width, slope, imperviousness, roughness coefficient and depression storage.

Roof areas within the monitored site were measured from Geographical Information System (GIS) software, ArcGIS, using a geo-referenced aerial photograph of the site. Measured roof areas ranged from 175m^2 to 280m^2. For simplicity and ease of modelling, each of the residential properties were assigned a roof area of 222m^2 (the average roof area), while the community hall's catchment, was measured to be 490m^2.

The width of the flow path is essentially a sub-catchment's area divided by the length of the longest overland flow path that water can travel. The length of the roof's rainfall flow path was taken as the longest measurement from the top of the roof to its edge. This gave values of between 7m to 8m for the residential homes and 10m for the community hall, giving the width as approximately 30m and 50m respectively.

Slope is the average slope of the surface over which runoff flows and is the same for both pervious and impervious surfaces. Most roofs in Australia have a roof pitch (slope) of 20^o to 30^o; hence, using this as a guideline, an average slope of 22^o (equivalent to a 40% gradient) was used for all roofs.

Imperviousness is the percentage of a sub-catchment area that behaves as an impervious surface through which rainfall cannot infiltrate. Since roofs were made of metal, an impervious value of 100% was used.

The amount of resistance that rainfall flow encounters as it runs off the roof surface is reflected by the roughness coefficient. As SWMM uses the Manning equation to compute overland flow rate, this coefficient is the Manning's roughness coefficient. Recommended roughness values for corrugated metal roofing material, is between 0.022 and 0.024 (Rossman, 2010). For the purpose of the study, a value of 0.022 has been used.

The amount of resistance that rainfall flow encounters as it runs off the roof surface is reflected by the roughness coefficient. This coefficient is the Manning's roughness coefficient. Recommended roughness values for corrugated metal roof material, is between 0.022 and 0.024 (Rossman, 2010). For the purpose of the study, a value of 0.022 has been used.

Model Calibration

Within the monitored period, a rainfall event was chosen as the main inflow input for model calibration, with recorded rainwater storage tank levels over the event providing the means to calibrate water capture by the model. Validation runs were also carried out with other rainfall events to ensure confidence in the model.

For the model to be calibrated and validated properly, it was essential that captured rainfall stored within the storage tanks was not extracted; i.e. the transfer pumps were not in operation. The rainfall events were chosen by analysing pump data and ensuring that the transfer pump for the rainfall periods was not in use. In addition, rainwater tank levels for the chosen events were to be rising rather than dropping to further confirm that the transfer pump was not in operation. Furthermore, to ensure chosen rainfall events were contributing fully into the rainwater harvesting system and that no overflow into the stormwater system was occurring, only lower intensity rainfall intensities were chosen. This would enable a more reliable means of model calibration as all the water captured by the roofs is accounted for in the storage tanks.

Rainfall events chosen for model calibration and validation purpose are shown in the following table, including their times, duration and peak intensity. Rainfall events shorter than 2 hours were not modelled in the SWMM model but were included in the analysis using the Rational Method described later.

Table 22: Rainfall events used for calibration and validation purposes in CDM

Date	Investigated Time		Duration	Purpose	Peak Rainfall Intensity,
	From	To	(h:mm)		mm/ 5 min
21-Aug	4:55 AM	7:50 AM	2:55	Calibration	1.2
9-May	4:00 AM	8:35 AM	4:35	Validation	0.4
19-May	8:45 AM	11:55 AM	3:10	Validation	1.4
12-Jun	3:55 AM	6:00 AM	2:05	Validation	0.4
29-Jun	8:15 AM	10:50 AM	2:35	Validation	2
6-Oct	4:00 AM	6:10 AM	2:10	Validation	0.4
22-May	12:35 AM	1:55 AM	1:20	Rational Method	0.6
16-Jul	6:20 PM	7:10 PM	0:50	Rational Method	0.4
17-Jul	12:35 PM	1:35 PM	1:00	Rational Method	0.4
25-Sep	2:30 AM	3:45 AM	1:15	Rational Method	0.8
29-Sep	11:00 AM	12:10 PM	1:10	Rational Method	0.6
24-Nov	12:00 AM	3:40 AM	3:40	Calibration for higher event	2.6

Design Rainfall Data

The process of estimating IFDs, known as frequency analysis, is an important part of hydrological design procedure. The Bureau of Meteorology (BOM) website provides an online IFD facility tool to obtain the IFDs for any location by inputting its mapping coordinates.. This tool generates a constant and six coefficients for the location and uses the following equation to estimate the rainfall intensity for each of the design storm's ARI and its duration.

$$\ln(I) = A + B(\ln(T)) + C(\ln(T))^2 + D(\ln(T))^3 + E(\ln(T))^4 + F(\ln(T))^5 + G(\ln(T))^6$$

Where:

I = intensity (mm/hr), T = storm duration (hrs), A = obtained constant, B, C, D, E, F & G = obtained coefficients.

Obtained coefficients were for the location in CDM (Easting: 517899, Northing: 6910931, Zone 56) are shown in Table 23.

Table 23: Constant and coefficients in IFD calculations

ARI in years	coeff A	coeff B	coeff C	coeff D	coeff E	coeff F	coeff G
1	3.61	-0.595	-0.0473	0.00577	2.83E-03	7.57E-05	-1.25E-04
2	3.86	-0.593	-0.0479	0.00552	2.97E-03	1.08E-04	-1.34E-04
5	4.08	-0.586	-0.0490	0.00531	3.18E-03	1.29E-04	-1.42E-04
10	4.19	-0.583	-0.0496	0.00502	3.32E-03	1.60E-04	-1.50E-04
20	4.32	-0.580	-0.0501	0.00494	3.39E-03	1.75E-04	-1.54E-04
50	4.47	-0.576	-0.0508	0.00463	3.56E-03	2.06E-04	-1.64E-04
100	4.57	-0.575	-0.0511	0.00467	3.61E-03	2.06E-04	-1.65E-04

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