Cost Effective Filtration System to Improve the Water Quality in Rainwater Tanks

By

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Certificate of original authorship

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Nomenclature

A	=	Surface area (m^2)
Å	=	Average pore diameter
ABS	=	Australian Bureau of Statistics
AC	=	Activated Carbon
ADWG	=	Australian Drinking Water Guidelines
ANZECC	=	Australian and New Zealand Environment and Conservation Council
APAH	=	American Public Health Association
ASTM	=	American Standard Testing and Methods
BMP	=	Best Management Practice
BOD	=	Biological Oxygen Demand
BOM	=	Biodegradable Organic Matter
C_b	=	The concentration of particles in a feed water
^0C	=	Degree Celsius
CC	=	Cubic centimetres
COD	=	Chemical Oxygen Demand
Da	=	Dalton
Dia.	=	Diameter
DOC	=	Dissolved Organic Carbon
DOM	=	Dissolved Organic Matter
DMF	=	Dual media filter
EfOM	=	Effluent Organic Matter
Eq.	=	Equation
g	=	Gram
G	=	Gravity
GAC	=	Granular Activated Carbon
hr	=	hour
HPSEC	=	High Pressure Size Exclusion Chromatography
kg	=	Kilogram
kL	=	Kilolitre
kPa	=	Kilo Pascal

L	=	Litre
m	=	Metre
m^2	=	Square metre
m^3	=	Cubic metre
MFI	=	Modified Fouling Index
mg	=	Milligram
mL	=	Millilitre
mm	=	Millimetre
MWD	=	Molecular Weight Distribution
MF	=	Microfiltration
N/A	=	Not applicable
NF	=	Nanofiltration
NOM	=	Natural Organic Matter
NTU	=	Nephelometric Turbidity Unit
Q1 – Q4	=	Quarter of the year
RO	=	Reverse Osmosis
RWOM	=	Rainwater Organic Matter
SWC	=	Sydney Water Corporation
T	=	time (s)
TC	=	Total carbon
TDS	=	Total Dissolved Solid
TIC	=	Total Inorganic Carbon
TMP	=	Trans-membrane Pressure
TOC	=	Total organic carbon
TSS	=	Total Suspended Solids
UF	=	Ultrafilter
µm	=	Micro metre
WHO	=	World Health Organisation
yr	=	Year

List of Publications

1. B. Kus, D. Pratheep, Jaya Kandasamy, S. Vigneswaran, H. K. Shon, G. Moody (2013) Reduction in water demand due to rainwater tanks in Sydney, Water Management, Proceeding of the institution of Civil Engineers, submitted for publication in February, 2013.

2. B. Kus, J. Kandasamy, S. Vigneswaran, H. K. Shon, and G. Moody (2013) Household rainwater harvesting system – pilot scale gravity driven membrane-based filtration system. Water Science and Technology, 13 (3) 790-797

3. B. Kus, J. Kandasamy, S. Vigneswaran, H. K. Shon, R. Sleigh and G. Moody (2012) Performance of granular medium filtration and membrane filtration in treating stormwater for harvesting and reuse. Desalination and Water Treatment, 45 (1-3) 120-127

4. G. Singh, M. A. H. Johir, J. Kandasamy, Dr. S. Vigneswaran, B. Kus, R. Naidu (2012) Stormwater Harvesting and Reuse, Encyclopaedia of Sustainability Science and Technology Robert A. Meyers (editor) pp 10095-10117 Springer Science DOI 10.1007/978-1-4419-0851-3_266

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Abstract

Although most Australians receive their domestic supply from reticulated mains or town water there are vast areas with very low population densities with few reticulated supplies (ABS, 2001). In many of these areas, rainwater collected in tanks is the primary source of drinking water. Small amounts of contaminants found in drinking water may have a chronic effect on the health over a human's lifetime due to its cumulative effect. Heavy metals have recently become a major concern as their concentration in stored rainwater was found to exceed recommended levels and proved to be unsuitable for human consumption.

Even in areas that are serviced by town mains water, many households, schools, community and commercial centres collect rainwater in tanks to augment supplies or provide an alternative and sustainable source of water. Widespread water restrictions in cities such as Sydney and Brisbane in recent years have brought to prominence water conservation measures, including the use of rainwater tanks.

The aim of this project is to develop a cost effective filtration system to improve water quality in rainwater tanks.

Pollutants in rain tank water can be generically described as containing colloidal solids, some microbial pollutants and micro-pollutants. The pollutant characteristics of sampled values of rainwater tanks in the Sydney metropolitan and rural areas of New South Wales, Australia, was analysed to determine the critical pollutants and those that do not comply with the Australian Drinking Water Guidelines (ADWG, 2011). The results indicate that before treatment, the rainwater complied with many of the

parameters specified in the ADWG (2011), though as previous studies demonstrate certain pollutants have the potential at times to exceed the limits specified in ADWG (2011). Additionally the characteristics of the first flush of roof runoff and its impact on the quality of water stored in a rainwater tank is presented.

Demand management analysis was conducted using data obtained for residential households throughout Sydney. The analysis provides the water demand for residential households in individual LGAs in Sydney. It also, for the first time, defines the reduction in water demand in all Sydney LGAs as a result of installing rainwater tanks. Such data can be used to size the volume of the permeate storage tank which is an integral element of a rainwater tank treatment system.

An experimental study of an affordable adsorption and membrane based treatment system was carried out. This included investigating the long term performance of GAC adsorption filter as a pre-treatment to micro-filter membrane filtration used to treat raw rainwater.

On completion of laboratory studies, filtration systems were tested to determine the operational performance of various media filter and membrane filter systems at the stormwater harvesting plant at Carlton, Sydney. This plant harvests stormwater baseflow whose quality was comparable to rainwater. The results provide useful information of how a comparable treatment system can treat rainwater.

Finally a gravity driven pilot scale rainwater treatment system was developed and tested in a residential property in Sydney. The results of the performance monitoring of the system is presented. The outcome of the study demonstrated that the water quality in rainwater tanks can be improved through a simple yet effective GAC & membrane filtration system driven only by the power of gravity leading to a cost effective setup that did not require expensive pumps and automation systems. The filter elements are periodically replaced to provide an ongoing high quality water supply.

Chapter 1

University of Technology, Sydney

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Information Technology

1. Introduction

1.1. Scarcity of water in Australia

Although most Australians receive their domestic supply from reticulated mains or town water there are vast areas with very low population densities with few reticulated supplies (Australian Bureau of Statistics, 2001). In many of these areas rainwater collected in tanks is the primary source of drinking water. Even in areas that are serviced by town mains water, many households, schools, community and commercial centres collect rainwater in tanks to augment supplies or provide an alternative and sustainable source of water. Widespread water restrictions in cities such as Sydney and Brisbane in recent years have highlighted the importance of water conservation measures, including the use of rainwater tanks.

1.2. Worldwide scarcity of water

Worldwide, scarcity of water is becoming a significant problem. Water shortages affect more than 80 developing countries that are home to half the world's population; in fact 80-90% of all diseases and 30% of all deaths result from poor water quality (Leitner, 1998). In the coming decade the number of people affected by severe water shortages is expected to increase four-fold (Engelman, 2000). A rapidly increasing population is placing pressure on existing water resources. In addition to the development of industrial and commercial activities around the world resulting in available water resources becoming polluted, the waste of natural sources, deforestation and climatic alteration due to global warming play a significant role in the reduction of average rainfall and runoff (North et al., 1995; World Water Assessment Program, 2003).

1.3. Alternative sources of water

1.3.1. Desalination

The creation of alternative sources of potable water is a significant issue worldwide and as a consequence desalination has become one of the most vital and valuable alternative resources of water for many countries. In 2012, it was estimated that there were approximately 15,000 desalination plants in 120 countries with a combined desalination capacity of up to 74 million cubic meters per day. Of this total, 47 million cubic meters per day was attributed to reverse osmosis (RO) (Poseidon Water, 2013). In 2009, there were 46 desalination plants in Australia that had capacities of greater than 10 kL/day. All but one utilised RO as the desalination process. The total output at the time was 291 ML/day for completed plants and 976 ML/day including plants that were still under construction or partially operational. The total output of proposed plants to 2013 was an additional 925 ML/day. About half of these plants were located in Western Australia although by 2013 this had become more evenly distributed across the states (CSIRO, 2009).

1.3.2. Stormwater harvesting

Stormwater harvesting is a vital strategy for improving urban water cycle management, given the stresses on water resources throughout Australia's cities and much of the world. Expanding the use of stormwater to add to the water supply and reducing stormwater runoff pollution are important objectives to alleviate this crisis. Stormwater is now acknowledged as a valuable reusable resource, rather than a nuisance as perceived by engineers in the past. In recent times engineers are utilising new best management practices and water treatment devices to effectively harvest, treat and reuse stormwater for non-potable uses such as watering fields and parks. Harvesting and reusing stormwater offers both a potential alternative water supply for non-drinking uses and a means to further reduce stormwater pollution in waterways.

1.3.3. Rainwater harvesting with rainwater tanks

Rainwater has been harvested throughout history as a means of providing for daily water supply requirements. Many methods of filtration have been used throughout history in many civilisations and cultures to remove sediments and large sized particles to improve the quality and appearance of collected water. Two common methods were to filter the water through sand or through folded cloth. Early methods like these were considered acceptable as it improved the water's appearance and on this basis was deemed safe to drink. Many harmful pollutants are, however, not visible to the naked eye or are soluble.

In modern society rainwater tanks are increasingly accepted as an alternative source of non-potable water which can be utilised for flushing toilets, washing clothes, watering gardens and washing cars. In New South Wales (NSW), it is currently a key design element of the Building and Sustainability Index (BASIX) for all new residential dwellings to achieve a 40% reduction in potable water usage through strategies including the use of native landscaping, water efficient water fixtures and utilising alternative water sources (Australian Government, 2012). According to one source, 84% of new BASIX homes use a rainwater tank or recycled water supply rather than mains water for toilets, laundry and/or irrigation (Australian Government, 2012). Residential use of rainwater tanks for non-potable purposes helps reduce the strain on potable water supplies. Furthermore the need for new water sources such as desalination plants and stormwater harvesting will be reduced and consequently postpone water supply and energy infrastructure expenditure.

Rainwater harvesting and treatment could be utilised as an alternative decentralised potable water source. RO desalination is expensive due to its energy requirements. Stormwater is rainwater that has washed over catchment surfaces such as roads littered with rubbish, debris and toxic contaminants from motor vehicles and over-fertilised gardens and grass. Treating this will require more extensive treatment with a high energy requirement to achieve potable grade water.

With advances in membrane technology, the cost and functionality of operating and replacing membranes has substantially fallen in recent times. Utilising a suitable and effective pre-treatment filter media followed by an efficient membrane filtration process and sterilisation could provide a potable water source in the backyards of each residential dwelling. The town water supply need only be utilised when the treated rainwater has run out. This would be an effective and efficient way to use a centralised water supply system that is only accessed when decentralised supplies (i.e. treated rainwater) have dwindled during periods of heavy consumption or periods of low rainfall. This would also lead to monetary savings for the resident as their water consumption from the water utility would be greatly reduced.

1.4. Pollutants in rainwater

Small amounts of contaminants found in drinking water may have a chronic effect on the health over a human's lifetime due to its cumulative effect. Heavy metals have recently become a major concern as their concentration in stored rainwater was found to exceed recommended levels and proved to be unsuitable for human consumption

(Magyar et al., 2007, Magyar et al., 2008, Han et al., 2006, Simmons et al., 2001). Rainwater storage tanks also accumulate contaminants and sediments that settle to the bottom. Characterisation of rainwater is required to devise treatment systems able to produce potable grade water.

1.5. First flush

When monitoring stormwater pollution during storm events it is common for the pollutant concentrations to peak before the discharge peaks. This effect - known as the first flush - occurs where large portions of accumulated pollutants that are easily disturbed become suspended in the overland surface runoff early on in the storm event. The first flush can also exist in rainwater runoff as pollutants and contamination accumulated on the roof surface between storm events. This first portion of rain can be highly polluted and needs to be examined. The impact of bypassing this portion of the roof runoff away from the rainwater tank and the improvement to the water stored in rainwater tanks is unknown.

1.6. Aim of study

The aim of this project is to develop a cost-effective filtration system to improve water quality in rainwater tanks. As a prerequisite, it is essential to characterise water in Sydney's metropolitan and rural rainwater tanks, and analyse the effects of the first flush rain runoff. A demand management analysis is helpful for optimising the volume of the permeate storage tank which is an integral element of a rainwater tank treatment system. A necessary component is to develop a cost-effective filtration system that will remove pollutants and improve the quality of the effluent, and in turn be suitable for potable water. The design should be inexpensive yet effective and ensure minimal operating and maintenance requirements. After completing the design, further fine tuning of the configuration and testing should be carried out. Finally a pilot scale study of the treatment system that was developed will be conducted.

1.7. Objectives of this study

The main objectives in this study are to: (i) identify issues with rainwater by analysing the first flush of roof runoff and characterise stored water in rainwater tanks (Chapter 4) and (ii) conducting a demand management analysis to determine trends in domestic residential water usage in the Sydney metropolitan area and define the role of rainwater tanks in reducing potable water demand. Such an analysis will also provide the data required to optimise the volume of the permeate storage tank of a rainwater tank treatment system (Chapter 5); (iii) laboratory design of a filtration system by developing an affordable membrane-based treatment system that will remove pollutants and provide significantly improved effluent quality that reflects potable water quality (Chapters 6 and 7); (iv) configure and test the pilot scale filtration system by utilising readily available comparable water (Chapters 8) and (v) conduct a pilot scale study of the treatment system developed in the study (Chapters 9).

1.8. Structure of the Study

Figure 1.1 presents the structure of the study which describes the project objectives and chapters to give a clear overview of the study.

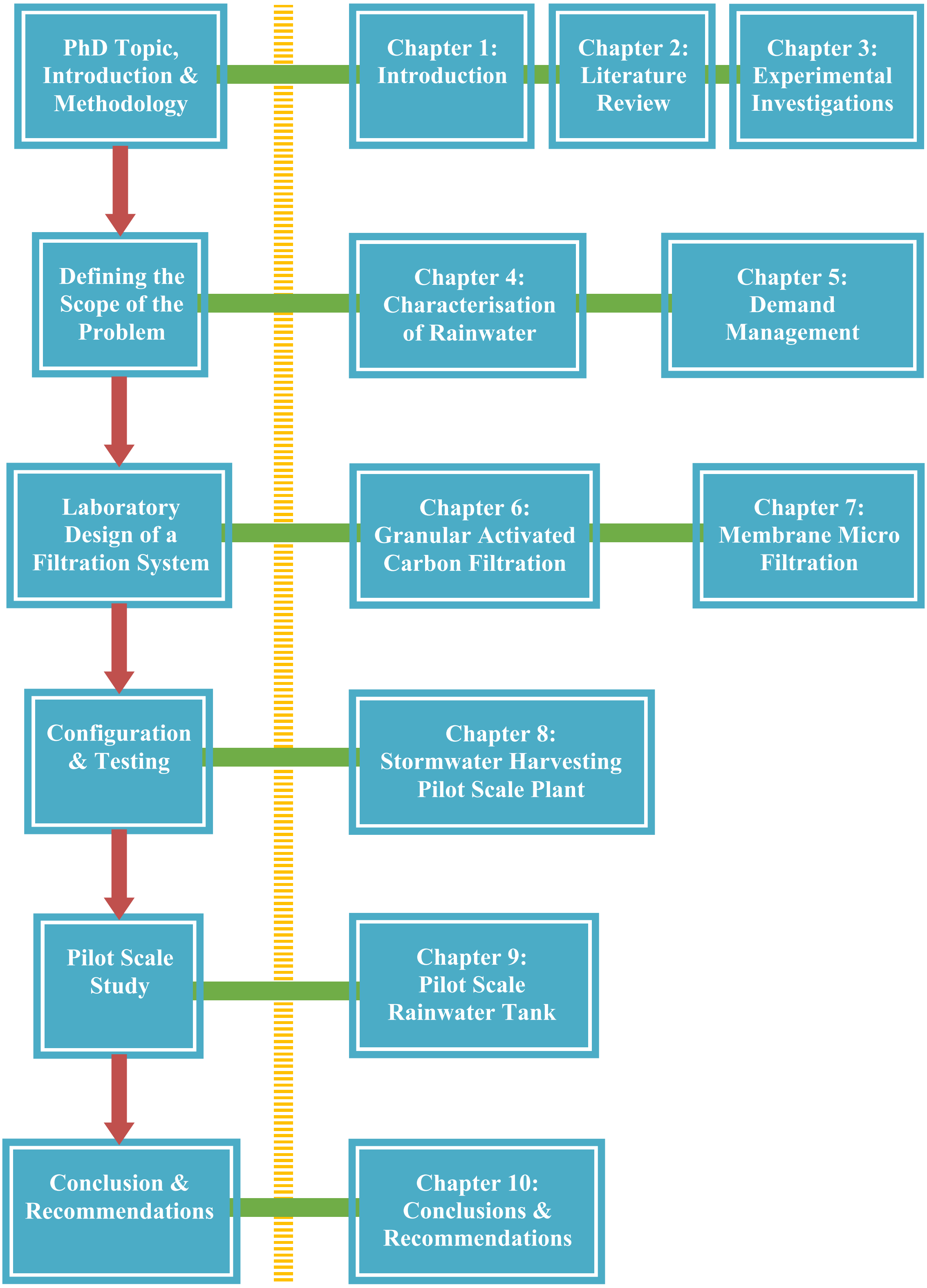

Figure 1.1 Diagram denoting structure of PhD research

1.8.1. Chapter 1 - Introduction to the Project

This chapter introduces the research problem and outlines the structure of the study.

1.8.2. Chapter 2 - Literature Review

This chapter consists of a literature review that focuses on: (i) characterisation of water stored in rainwater tanks and in the first flush; (ii) media (granular activated carbon, anthracite or sand) filtration of rainwater and membrane microfiltration; (iii) water demand; and (iv) characterisation and treatment of stormwater.

1.8.3. Chapter 3 - Experimental Investigations

Chapter 3 provides details on the materials, equipment, methodology, and procedures used in this study for the: (i) characterisation of rainwater and first flush; (ii) operation of rainwater treatment system; (iii) operation of stormwater filtration system; and (iv) pilot study of a rainwater filtration system.

1.8.4. Chapter 4 - Characterisation of Rainwater

The results of the characterisation of rainwater are presented in this chapter. Pollutants in rainwater tanks can be generically described as containing colloidal solids, some microbial pollutants and micro-pollutants. The concentration of pollutants in samples of stored water in such tanks in the Sydney metropolitan area and rural New South Wales were analysed to determine those that do not comply with the Australian Drinking Water Guidelines (ADWG, 2011).

Additionally this chapter characterises the first flush of roof runoff and the volumes of first flush that should be by-passed from the tank and its influence on the water quality in the rainwater tank.

1.8.5. Chapter 5 - Demand Management

A detailed demand management study was conducted using data collected from three sources: Kogarah Council, Sydney Water Corporation and the Australian Bureau of Statistics. Water demand and trends for residential areas in local government areas (LGAs) in Sydney were developed and reported. The impact of rainwater tanks in reducing water demand was analysed.

1.8.6. Chapter 6 and 7 – Filtration of Rainwater

These chapters investigate the use of pre-treatment such as deep-bed filtration, adsorption-filtration and membrane filtration technology to configure a cost-effective and energy efficient system that operates under the gravity head of the stored raw water. The aim is to develop an effective media and membrane filtration system requiring minimal maintenance and user intervention. An important consideration is to keep to a minimum the incremental cost of such a system over existing rainwater systems. This is achieved by using readily available, mass-produced components (valves, pumps, timers, etc.) for simple yet effective operation and control.

1.8.7. Chapter 8 – Stormwater Harvesting Pilot Scale Plant

This chapter presents the results for the operation of a filtration plant which is utilised for harvesting stormwater baseflow. Here the water quality is comparable to that of rainwater. The results will provide useful data on how a comparable treatment system can treat rainwater.

1.8.8. Chapter 9 – Design and Operational Simulation of a pilot scale rainwater tank

This chapter details the configuration of a rainwater treatment system developed from the results of this study and its operation at a residential property in Sydney. The system was monitored for 120 days and here the results are presented.

1.8.9. Chapter 10 - Conclusions and Recommendations

The final chapter concludes the findings of the study and provides recommendations for further areas of research.

Chapter 2

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2. Literature Review

2.1. Introduction

Rainwater harvesting (RWH) typically consists of collecting rainwater from a catchment area, its storage and subsequently using it. The harvested rainwater is usually used for non-potable applications such as flushing toilets, watering gardens and supplementing primary potable supply uses. Although most Australians receive their domestic water supply from reticulated mains or town water, there are vast areas with very low population densities with few reticulated supplies (ABS, 2001). In many of these areas rainwater collected in tanks is the primary source of drinking water. Even in areas that are serviced by town mains water, many households, schools, community and commercial centres collect rainwater in tanks to augment supplies or provide an alternative and sustainable source of water. Widespread water restrictions in cities such as Sydney and Brisbane in recent years have brought to prominence water conservation measures, including rainwater tanks. Agencies like Sydney Water have offered cash rebates to support the installation of rainwater tanks.

As the supply of mains water is readily available in developed countries such as Australia, and while it remains relatively cheap, people do not have the incentive to collect and store rainwater. As population increases, the strain on potable mains supply also increases and water sources such as RWH will play an important part in order to meet increasing demand. This literature review provides an overview of water harvesting practices, the quality of various sources of water and commonly utilised methods of filtration. A more detailed review on the water quality and different treatment methods such as media and membrane filtration is provided in Chapters 4 through 9.

2.2. Rainwater harvesting

2.2.1. History of water harvesting

There is a long history of RWH systems and they possibly originated in the early civilizations in the Middle East and Asia (Gould & Nissen-Peterson, 1999). Clay pots dating back 6000 years in the Gansu Province, China, may have been utilised for RWH off ancient shelters in times of need. Much larger systems have also been discovered in China consisting of excavated clay-lined bottle-shaped water cellars developed for underground RWH systems with volumes often up to 30 cubic metres and likely to have provided the domestic water requirements for thousands of people (Gould & Nissen-Peterson, 1999). There is also evidence of simple stone rubble structures for capturing rainwater dating back to the third millennium BC (Agarwal and Narain, 1997). In the Negev desert, Israel, which has only approximately 100 mm of rainfall each year, large cisterns dating back to 2000 BC have been found along hillsides and are believed to have collected runoff for habitation and cultivation in this arid place (Evenari et al., 1961).

In North Africa and around the Mediterranean, thousands of large RWH cisterns were located along the coastal deserts with the oldest dating back to at least 2000 years ago and ranging in volume from 200 to 2000 cubic metres, some of which are still in use today (Shata, 1982). Constant difficulties have plagued RWH ranging from available yield to hygiene issues. Various forms of filtration were used to improve its quality (Gould & Nissen-Peterson, 1999).

2.2.2. Rainwater harvesting today

In recent times the importance of water for human survival has been often forgotten in urban areas, especially in the developed world, as potable tap water has become readily available as a result of the introduction of large scale water treatment and water supply distribution systems (Gould & Nissen-Peterson, 1999). Consequently the use of RWH systems has declined in much of the developed world. Concerns are growing that large scale urban water treatment and distribution systems may be unsustainable as the density of population increases and the availability of drinking water catchments are progressively becoming more difficult to source and protect (Hiessl et al., 2001).

Rapidly developing countries such as China and India together comprise more than one third of the global population. Both countries face growing pressures on their finite fresh water resources and now recognize the important role of RWH techniques in order to help with resource management (Agarwal and Narain, 1997). Many other countries are also beginning to show an increasing interest in employing RWH techniques (Herrmann & Schmida, 1999; Argue, 2001; Konig, 2001; Villarreal & Dixon, 2005). It is becoming evident today that considerations be given to source control techniques and to supplement mains supplies with alternative water sources rather than remain reliant on large-scale centralised potable water systems.

Many potable and non-potable harvesting systems already exist globally (Lye, 1992; ABS, 2001; Heggen, 2000) for internal and external applications where water of different qualities are used for different purposes (Fewkes, 1999; Gould & Nissen-Peterson, 1999; Coombes et al., 2000; Cooper, 2001; Leggett et al., 2001; Ratcliffe, 2002; Weiner, 2003). In addition to easing reliance on mains supplies, de-centralised RWH can be beneficial in other applications such as utilising the RWH systems to potentially reduce peak flow rates and effective runoff volumes during a storm event, provided there is available storage at the commencement of rainfall (Coombes et al., 2001; Vaes & Berlamont, 2001).

2.3. Water demand in Australia

Australia is the driest continent and has one of the most variable rainfall intensities in the world. In the last 100 years, Australia has suffered six major droughts and 15 other droughts, the most recent one (2000-2009) being the worst on record. The last drought and concerns about climate change have all highlighted the need to manage water resources more sustainably. In Australia, potable water demand is expected to increase beyond the current available water supplies due to the predicted population increases in capital cities, and the reoccurrence of the recent drought exacerbated by concerns about climate change. This has forced governments to look at ways of securing alternative water supplies. A number of solutions can be utilised to save water either by reducing demand and increasing efficiency or by increasing the available supply. The former include: community education; water restrictions; retro-fitment of water fixtures to reduce consumption; and rebate schemes to promote the installation of rainwater tanks. Methods for increasing available supply include alternative water supplies such as desalination, recycling; and creating alternative lower quality supplies from sources such as grey water for local non-potable uses.

Table 2.1 shows the usage of rainwater and its distribution in 2001. 15.7% of Australian households use rainwater tanks, with 11.4% of households using tanks as their main source of drinking water (Table 2.2) (ABS, 2001).

In 2010, 32.2% of households with a dwelling suitable for a rainwater tank had a one installed compared to 24.0% in 2007 and 15.7% in 2001. Since 2001, the overall use of rainwater tanks has doubled. A comparison of capital city to non-capital city data from 2007 and 2010, shows the greatest increase in proportion of household rainwater tanks installed occurred in capital cities (from 15.4% in 2007 to 25.7% in 2010) compared with non-capital cities (from 38.0% in 2007 to 42.8% in 2010). Brisbane experienced the largest increase with 43% of households reporting a rainwater tank at their dwelling in 2010 compared with 18% in 2007. This was followed by Melbourne, with 28% of households reporting a rainwater tank in 2010 compared with 12% in 2007 (Table 2.2) (ABS, 2010; ABS, 2001).

In 2001, 80.7% of residents stated that the mains supply was their primary drinking source (Table 2.3). 11.4% of households relied on rainwater tanks, 6.9% relied on purchased bottled water and the remaining 1% relied on a combination of spring, bore/well, river/creek/dam or other sources of water (ABS, 2001). By 2010 there had been little change to the main source of drinking water. There was a slight increase to 82.7% for mains supply, a slight decline to 9.8% regarding reliance on rainwater tanks and a steady 6.6% for purchased bottle water (Table 2.4) (ABS, 2010).

In 2010, however, the ABS provided water usage data separately for capital and non-capital cities showing a greater disparity between the two. Over 90% of households in capital cities reported mains or town water as their main source of water for drinking compared with 69.3% of households living outside capital cities. For households living outside these cities, water from rainwater tanks was the second most popular main source of water for drinking (22.2%) (Table 2.4). Only 25.0% of South Australian households outside of Adelaide used mains or town water as their main source of water for drinking. The main source of water for South Australian households outside of Adelaide was rainwater tanks (66.2%) (Table 2.4) (ABS, 2010). This data shows the rather large reliance on centralised town mains water supply for capital cities in Australia.

In 2010, almost half of Australian households (45.1%) used mains or town water as their main source of water for gardening. The Northern Territory had the highest proportion of households (75.8%) using mains or town water for gardening and both Queensland and Victoria had the lowest (32.4%). Queensland and Victoria had the highest proportion of households using water from a rainwater tank as their main source of water for gardening (20.3% and 18.5%, respectively) (Table 2.5). Queensland and Victoria also had the highest proportion of households that relied on rainfall for gardening or did not water the garden (31.5% and 29.0%, respectively) (Table 2.5) (ABS, 2010).

Table 2.1 Households with rainwater tanks in 2001 (ABS, 2001)

State	NSW	VIC	QLD	SA	WA	TAS	NT	ACT	Total
Total households with rainwater tanks (%)	9.7	13.5	17.5	51.8	10.4	17.2	1.3	2.0	15.7

Note: Allocation of capital city versus non-capital city data was not available in this publication

Table 2.2 Households with rainwater tanks in 2007 & 2010 (ABS, 2010)

2010 / State	NSW	VIC	QLD	SA	WA	TAS	NT	ACT	Total
Total households with rainwater tanks (%)	23.7	35.5	42.3	57.2	15.9	26.6	9.1	17.7	32.2
Capital city households with rainwater tanks (%)	16.3	28.2	43.4	44.6	8.4	11.5	NA	NA	25.7
Non-capital city households with rainwater tanks (%)	33.1	51.7	41.5	89.3	34.7	36.2	NA	NA	42.8
2007/ State	**NSW**	**VIC**	**QLD**	**SA**	**WA**	**TAS**	**NT**	**ACT**	**Total**
Total households with rainwater tanks (%)	20.5	21.4	25.8	53.8	15.8	24.7	7.3	8.2	24.0
Capital city households with rainwater tanks (%)	10.3	11.6	18.4	44.5	8.1	14.6	NA	NA	15.4
Non-capital city households with rainwater tanks (%)	33.5	43.6	31.9	78.6	38.1	31.7	NA	NA	38.0

NA: denotes data was not available for publication but was included in totals where applicable

Table 2.3 Main source of drinking water (ABS, 2001)

Total households with:	NSW	VIC	QLD	SA	WA	TAS	NT	ACT	Total
Mains/town as main source of drinking water (%)	85.0	83.7	79.4	49.9	84.1	80.9	93.3	97.2	80.7
Rainwater tank as main source of drinking water (%)	7.1	10.5	13.9	33.1	7.3	13.6	1.3	0.5	11.4
Spring as main source of drinking water (%)	<0.1	<0.1	0.2	0.2	<0.1	0.2	<0.1	<0.1	0.1
Purchased bottled water as main source of drinking water (%)	7.5	5.4	4.7	16.0	7.4	3.7	2.2	2.3	6.9
Bore/well as main source of drinking water (%)	0.1	0.1	1.3	0.2	0.8	1.0	3.2	<0.1	0.4
River/creek/dam as main source of drinking water (%)	0.2	0.1	0.3	<0.1	<0.1	0.6	<0.1	<0.1	0.2
Other as main source of drinking water (%)	<0.1	0.2	0.3	0.6	0.5	0.1	<0.1	<0.1	0.2

Note: Allocation of capital city versus non-capital city data was not available in this publication

Table 2.4 Main source of drinking water in Australia (ABS, 2010)

Total households utilising:	**NSW**	**VIC**	**QLD**	**SA**	**WA**	**TAS**	**NT**	**ACT**	**Total**
Mains/town as main source of drinking water (%)	86.3	85.2	78.6	67.8	83.3	75.5	88.2	98.7	82.7
Rainwater tank as main source of drinking water (%)	6.2	7.8	13.6	23.1	8.9	17.1	NA	NA	9.8
Spring as main source of drinking water (%)	NA	NA	NA	NA	NA	NA	NA	NA	NA
Purchased bottled water as main source of drinking water (%)	6.4	6.7	6.5	8.2	7.6	4.9	NA	NA	6.6
Bore/well as main source of drinking water (%)	0.6	NA	0.9	0.6	NA	0.8	6.4	<0.1	0.6
River/creek/dam as main source of drinking water (%)	NA	NA	NA	NA	NA	NA	NA	NA	NA
Other as main source of drinking water (%)	0.4	NA	0.4	0.3	NA	1.8	0.8	<0.1	0.3
Capital city households utilising:	**NSW**	**VIC**	**QLD**	**SA**	**WA**	**TAS**	**NT**	**ACT**	**Total**
Mains/town as main source of drinking water (%)	91.3	92.7	88.5	83.7	90.2	89.7	NA	NA	90.5
Rainwater tank as main source of drinking water (%)	1.4	1.2	4.0	7.0	2.7	6.3	NA	NA	2.5
Spring as main source of drinking water (%)	NA	NA	NA	NA	NA	NA	NA	NA	NA
Purchased bottled water as main source of drinking water (%)	7.0	6.1	7.4	9.0	6.8	3.7	NA	NA	6.9
Bore/well as main source of drinking water (%)	NA	<0.1	<0.1	NA	0.3	NA	NA	NA	<0.1
River/creek/dam as main source of drinking water (%)	NA	NA	NA	NA	NA	NA	NA	NA	NA
Other as main source of drinking water (%)	NA	<0.1	<0.1	NA	<0.1	NA	NA	NA	<0.1
Non-capital city households utilising:	**NSW**	**VIC**	**QLD**	**SA**	**WA**	**TAS**	**NT**	**ACT**	**Total**
Mains/town as main source of drinking water (%)	78.5	66.0	71.2	25.0	64.5	65.4	NA	NA	69.3
Rainwater tank as main source of drinking water (%)	13.7	24.4	20.8	66.2	25.3	24.7	NA	NA	22.2
Spring as main source of drinking water (%)	NA	NA	NA	NA	NA	NA	NA	NA	NA
Purchased bottled water as main source of drinking water (%)	5.6	8.3	5.8	6.2	9.7	5.8	NA	NA	6.5
Bore/well as main source of drinking water (%)	NA	NA	1.6	NA	NA	NA	NA	NA	1.4
River/creek/dam as main source of drinking water (%)	NA	NA	NA	NA	NA	NA	NA	NA	NA
Other as main source of drinking water (%)	NA	NA	0.7	NA	NA	NA	NA	NA	0.6

NA: denotes data was not available for publication but was included in totals where applicable

Table 2.5 Main source of water for gardening (Houses with a garden) (ABS, 2010)

Total households utilising:	**NSW**	**VIC**	**QLD**	**SA**	**WA**	**TAS**	**NT**	**ACT**	**Total**
Mains / town water (%)	51.2	31.9	32.4	57.5	64.5	61.6	75.8	54.6	45.1
Rainwater tanks (%)	10.6	18.5	20.3	15.4	2.9	9.3	<0.1	11.2	14.0
Bore/well water (%)	4.4	2.1	5.6	3.8	24.0	2.3	NA	NA	6.2
River/creek/dam water (%)	2.1	1.2	1.2	1.6	1.5	4.2	NA	NA	1.6
Rainwater collected in other container (%)	0.8	0.6	0.4	NA	<0.1	NA	<0.1	0.9	0.5
Grey Water (%)	7.3	16.5	8.4	8.7	2.1	4.2	1.3	8.6	9.3
Don’t water/rely on rainwater only (%)	23.4	29.0	31.5	12.4	4.7	17.7	12.8	24.5	23.2
Other (%)	0.1	0.2	0.2	NA	0.4	NA	<0.1	<0.1	0.2
Capital city households utilising:	**NSW**	**VIC**	**QLD**	**SA**	**WA**	**TAS**	**NT**	**ACT**	**Total**
Mains / town water (%)	50.6	29.6	24.7	65.9	66.4	79.3	<0.1	<0.1	44.7
Rainwater tanks (%)	10.1	18.5	27.6	11.9	0.6	3.8	<0.1	<0.1	14.0
Bore/well water (%)	2.2	0.4	0.4	1.2	26.7	<0.1	<0.1	<0.1	4.7
River/creek/dam water (%)	0.5	0.4	0.4	<0.1	<0.1	0.9	<0.1	<0.1	0.3
Rainwater collected in other container (%)	1.1	NA	NA	NA	<0.1	<0.1	<0.1	<0.1	NA
Grey Water (%)	8.6	17.0	8.5	9.1	1.8	1.9	<0.1	<0.1	10.1
Don’t water/rely on rainwater only (%)	27.0	33.3	38.3	11.8	4.6	14.1	<0.1	NA	25.6
Other (%)	<0.1	NA	NA	NA	<0.1	<0.1	<0.1	NA	NA
Non-capital city households utilising:	**NSW**	**VIC**	**QLD**	**SA**	**WA**	**TAS**	**NT**	**ACT**	**Total**
Mains / town water (%)	51.9	37.3	38.3	34.6	59.4	49.7	<0.1	<0.1	44.6
Rainwater tanks (%)	11.4	18.5	14.8	24.9	8.9	13.1	<0.1	<0.1	14.4
Bore/well water (%)	7.3	5.9	9.7	10.9	16.7	3.9	<0.1	<0.1	8.5
River/creek/dam water (%)	4.3	2.8	1.8	6.1	5.6	6.4	<0.1	<0.1	3.6
Rainwater collected in other container (%)	0.4	NA	NA	NA	<0.1	NA	<0.1	<0.1	NA
Grey Water (%)	2.7	15.3	8.4	7.9	2.9	5.7	<0.1	<0.1	8.3
Don’t water/rely on rainwater only (%)	18.6	18.9	26.3	14.2	5.1	20.1	<0.1	<0.1	19.7
Other (%)	0.3	NA	NA	NA	1.4	NA	<0.1	<0.1	NA

NA: denotes data was not available for publication but was included in totals where applicable

Table 2.6 Household water use per capita (ABS, 2006b)

State	NSW	VIC	QLD	SA	WA	TAS	NT	ACT	Average
2004-05 (kL/capita)	84	81	124	94	180	143	153	95	103
2000-01 (kL/capita)	97	97	143	110	191	125	162	115	120

Table 2.7 Household water use per household (ABS, 2006b)

State	NSW	VIC	QLD	SA	WA	TAS	NT	ACT	Average
2004-05 (kL/household)	219	209	323	244	468	372	399	248	268
2000-01 (kL/household)	252	251	372	286	497	326	420	298	312

In 2004-05, Australians on average consumed 103 kL/capita during 2004–05 compared to 2000–01 when average water consumption per capita was 120 kL/capita (Table 2.6). Western Australia reported the highest household water consumption per capita (180 kL/capita), followed by the Northern Territory (153 kL/capita), Tasmania (143 kL/capita) and Queensland (124 kL/capita). Victoria had the lowest average household water consumption per capita (81 kL/capita) followed by New South Wales (84 kL/capita), South Australia (94 kL/capita) and the Australian Capital Territory (95 kL/capita) (Table 2.6).

In 2004-05, Australian households consumed on average 268 kL of water per household (Table 2.7), with an average of 2.6 persons per household (ABS, 2002b). Western Australia had the highest water consumption per household (468 kL per household) in 2004–05. This was followed by the Northern Territory (399 kL per household) and Tasmania (372 kL per household). Victoria had the lowest average water consumption per household (209 kL per household) followed by New South Wales (219 kL per household) and South Australia (244 kL per household) (Table 2.7).

Table 2.8 Composition of household water usages 2000-01 (ABS, 2004)

State	NSW	VIC	QLD	SA	WA	TAS	NT	ACT	Average
Bathroom (%)	26	26	19	15	17	NA	NA	16	20
Toilet (%)	23	19	12	13	11	NA	NA	14	15
Laundry (%)	16	15	10	13	14	NA	NA	10	13
Kitchen (%)	10	5	9	10	8	NA	NA	5	8
Outdoor (%)	25	35	50	50	50	NA	NA	55	44

NA: denotes data was not available for publication but was included in totals where applicable

Table 2.8 shows in 2001 that for all states and territories, the majority of household water was used for outdoor purposes (44%). Queensland, South Australia, Western Australia and the Australian Capital Territory all reported using at least 50% of the household water for outdoor reasons. New South Wales used 25% of household water for outdoor purposes and Victoria reported using 35% outdoors. Indoor use, including bathrooms (20%) and toilets (15%) accounted for a significant proportion of household water use in Australia. Nationally, 8% of water used by households (or less than 1% of total water use in Australia) was used in the kitchen. Table 2.9 combines each state's average household water usage from Table 2.7 with the average composition as shown in Table 2.8.

Table 2.9 Average water demands derived from household water usage at each location from Table 2.7 (2004-05) and Table 2.8 (ABS, 2004; ABS, 2006b)

State	NSW	VIC	QLD	SA	WA	TAS	NT	ACT	Average
Bathroom (kL/household)	56.9	54.3	61.4	36.6	79.6	NA	NA	39.7	53.6
Toilet (kL/household)	50.4	39.7	38.8	31.7	51.5	NA	NA	34.7	40.2
Laundry (kL/household)	35.0	31.4	32.3	31.7	65.5	NA	NA	24.8	34.8
Kitchen (kL/household)	21.9	10.5	29.1	24.4	37.4	NA	NA	12.4	21.4
Outdoor (kL/household)	54.8	73.2	161.5	122.0	234.0	NA	NA	136.4	117.9

NA: denotes data was not available for publication but was included in totals where applicable

Table 2.10 Households with water-saving products 1998 - 2010 (ABS, 2010)

2010 / State	NSW	VIC	QLD	SA	WA	TAS	NT	ACT	Total
Total households with water efficient shower heads (%)	64.9	67.4	72.2	64.5	61.4	52.5	46.9	63.5	66.1
Total households with dual flush toilets (%)	81.9	88.6	89.9	89.0	88.0	78.0	90.0	83.9	86.3
1998 / State	**NSW**	**VIC**	**QLD**	**SA**	**WA**	**TAS**	**NT**	**ACT**	**Total**
Total households with water efficient shower heads (%)	30.0	31.7	34.1	33.5	37.7	32.3	28.0	32.6	32.3
Total households with dual flush toilets (%)	46.2	64.2	53.1	63.2	63.1	48.1	63.0	48.1	55.2

The proportion of households with dual-flush toilets and water-efficient shower heads has increased in the last 12 years. In 1998, 55% of households had a dual flush toilet compared with 86% in 2010. Similarly the prevalence of water-efficient shower heads has increased from 32% of households in 1998 to 66% in 2010 (Table 2.10). Further household water demand was derived using data from the Australian Bureau of Statistics, State of the Environment Reports and Water Services Association Australia (WSAA) to provide an estimate of indoor water usage demand based on the number of people per household (Table 2.11).

Table 2.11 Average water demands derived for households at each location

Location	**Indoor water demand (L/day) versus household size (people)**						**Outdoor demand (L/day)**
	1	**2**	**3**	**4**	**5**	**6**	
Adelaide	112	230	347	465	583	700	300
Brisbane	116	215	343	495	610	726	258
Melbourne	150	288	425	563	700	839	136
Sydney	233	452	670	888	1107	1326	160

Studies in Perth, Western Australia and Yarra Valley in Victoria indicate that around 35-40% of total water used in domestic consumption was for external use, mainly for watering gardens with a minor component for swimming pools, washing vehicles, etc., (Stewart et al., 2005). Studies undertaken in Perth, Western Australia using smart meters showed that during the study period (2002 to 2009) there was no overall significant change in total indoor water consumption although there were some reductions for some components (for example showers). In Perth, the Water

Corporation claimed a significant reduction (17%) in annual per capita water consumption from 128 kL to 106 kL over comparative periods (1998-2000 and 2008-2009) (Loh and Coghlan, 2003; Water Corporation, 2009). This reduction was attributed to the water efficiency measures introduced in 2001 such as regulating garden watering to two days a week. This implies a significant reduction came from outdoor water usage. The average outdoor water consumption in Adelaide, Brisbane, Melbourne and Sydney are 300, 258, 136, and 160 L/day/household, respectively (ABS, 2005a; see Table 2.11).

In Sydney nearly all (96%) households connected to town water supply (excluding flats, units or apartments) have gardens. Furthermore of these households, only 62% within Sydney used mulch (which reduces water demand) in their gardens. 17% of total households in Sydney have swimming pools (ABS, 2002a) and most households in Sydney wash their vehicles at home as opposed to using a carwash facility. These findings imply that there is a large potential to reduce external water use in Sydney. This component is conveniently supplied with rainwater tanks. In Sydney, 11.1% of the households have rainwater tanks (ABS, 2010).

Kuczera (2008) analysed complementary centralised and decentralised storage systems to evaluate the extent to which rainwater tanks could reinforce mains water supply. Kuczera (2008) analysed conditions where urban rainwater tanks made the most significant contribution to urban water supply drought security. Although rainwater tanks typically draw down in less than a week and, as a result, may be empty for considerable periods, it is incorrect to conclude they cannot contribute to the drought security of the whole system. The use of rainwater tanks can improve city-wide drought security for a given centralised reservoir capacity and for a given threshold probability, rainwater tanks reduce the required centralised reservoir capacity. Rainwater tanks benefit drought security more when the centralised system is more stressed due to high supply variability or high load and when the per-capita household demand is lower. As the uptake of installing rainwater tanks increase, system drought security also increases. However, the growth rate of benefits diminishes with increasing uptake. The benefits of rainwater tanks were not particularly sensitive to reduced rainfall. This robustness may be an important consideration if and when facing periods of reduced rainfall.

Coombes et al. (2008) showed that in their continuous modelling and historical sequences the available storage or empty portion of rainwater tanks prior to rain events was primarily due to the seasonality of rainfall patterns at each location. Brisbane has a predominantly summer rainfall pattern that coincides with the expected higher water demands during the summer period and larger available volumes of storage. In contrast, Adelaide contains predominantly winter rainfall patterns when water demand is low. This results in less available storage volumes for rain events. The larger available storage volumes in tanks located in Sydney resulted from the higher indoor water demands that balanced the more even pattern of seasonal distribution of rainfall. The available rainwater tank storage volumes prior to rain events increased regardless of location, and were due to the increase of tank capacity, household size and rainfall intensity. The storage volume available in a rainwater tank, on average, decreased with larger areas of connected roof.

Moy (2011) provided published post-installation analysis of a group of rainwater tanks installed between 2005 and 2007 in the Wollongong and Shellharbour LGAs, towns south of Sydney and their effects on mains water consumption. The study aimed to compare the average mains water reductions achieved in households with and without rainwater tanks. The results reveal that households with rainwater tanks, located in both Wollongong and Shellharbour, reduced their consumption over the period of 2005-2007 by approximately 10.3%, however households without rainwater tanks also reduced their water consumption over the same period by 10.8%. Separate analysis was not provided for the individual towns. Given that this was a relatively short period the analysis did not adjust the data for the overall reduction of water consumption in the in the wider community. The total residential water consumption for Wollongong and Shellharbour over that period (2005-2007) showed a reduction in water consumption of 3.3% and 0% respectively.

Knights et al (2012) presents data from a rainwater tank incentive scheme in Marrickville LGA where pre and post rainwater tank installation water usage was analysed. The study also included real-time metering of mains water and rainwater use of an individual household in the program. The results from the study showed that rainwater tanks can reduce water consumption on average by 110 L/d with a range of 7 L/day to 390 L/day. This equates to a reduction of water consumption by 25%. The data was not adjusted for the overall reduction of water consumption that occurred in the wider community. An assessment of this was not possible since the time period over which the analysis was undertaken was not given.

Rahman et al. (2008) analysed the possibility of achieving "pay back" for a rainwater harvesting system. It was discovered that the most favourable financial condition for the rainwater harvesting system among various scenarios examined in the case study is 1600 m^2 roof area, 5% nominal discount rate, Aus$1.634/kL water price and inflation rate of 4.5% p.a. for water price which presents a benefit-cost ratio (BCR) of 1.39. The current price of water in Sydney is $2.13/kL. It should be noted that the average roof area of a residential house is 250 m^2. This implies a favourable application to schools, hospitals, commercial and industrial buildings.

Rahman et al. (2008) also analysed the impact of BASIX on rainwater harvesting systems. BASIX is one of the strongest sustainable planning measures to be undertaken in Australia and is implemented as part of the New South Wales Environmental Planning and Assessment Regulations (EP&A Regulations, 2000). BASIX applies to all residential dwelling types and is part of the development application process in NSW. It aims to make all residential dwelling types in NSW energy and water efficient. One of the key design principles of BASIX for all new residential dwellings to achieve a 40% reduction in potable water usage through strategies includes: firstly, using native landscaping, water efficient water fixtures; and secondly, utilising alternative water sources (Australian Government, 2012). Installing a rainwater tank is the most widely used measure for meeting the alternative water objectives with 84% of new BASIX homes using a tank or recycled water supply rather than mains water for toilets, laundry and/or irrigation. (Australian Government, 2012).

The BCR is smaller with the BASIX approach in comparison to the non-BASIX ones. However, the overall water demand for the non-BASIX approach is much higher. A higher roof and site area is more favourable than smaller ones in terms of water savings and financial benefit. Of the capital and maintenance costs over the whole life cycle cost of a rainwater harvesting system, plumbing cost forms the largest single component of the capital cost. Cost related to pump maintenance and replacement form a significant component of the total expenditure.

2.4. Rainwater harvesting in Australia

In Australia, domestic RWH systems are predominantly used for non-potable applications such as toilet flushing, garden irrigation and laundry washing. In this way mains supply water is to some degree being substituted by harvested rainwater and so RWH systems reduce the demand on centralised water supply and distribution infrastructure (Schilling & Mantoglou, 1999; Coombes & Kuczera, 2003).

A typical configuration of a residential RWH system is shown in Figure 2.1. Although many factors affect the quantity and quality of rainwater harvested ranging from a roof such as initial losses, overhanging trees, maintenance of the system, first flush devices, etc., its operation is the same. Rain falls on a catchment area such as a domestic residential roof generating rainwater runoff. Typically this runoff can contain dislodged contaminants from the catchment surface which had settled between storm events. The runoff collects in a gutter which grades to a downpipe and drains into a rainwater tank. Once the capacity of the tank is exceeded, the overflow discharges via an overflow pipe at the top of the tank. In urban areas the overflow is plumbed back into a down pipe/stormwater pit. In rural areas it is allowed to discharge over land.

Table 2.12 summarises the main water quality contaminants and their sources associated with rainwater tanks (Australian Government DHA, 2004). To reduce the occurrences of these contaminants, various types of pre-tank filters exist such as gutter guards, first flush devices or mosquito meshes. To improve the water quality post-harvesting, prior to its intended end use, some form of filtration system could be utilised. Literature on various methods of filtration is provided in Section 2.6.

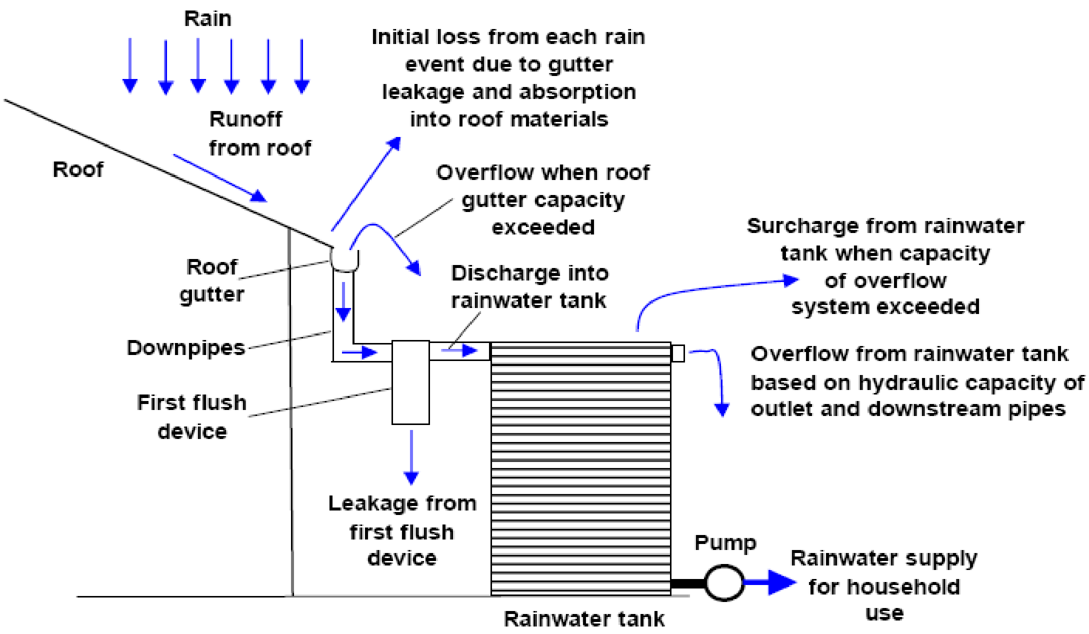


Figure 2.1 Schematic of typical rainwater tank RWH processes (Coombes et al., 2008)

Table 2.12 Typical contaminants within rainwater tanks (Australian Government DHA, 2004)

Contaminant	Source
Faecal contamination from birds and small animals	Overhanging branches on roof, animal access to tank
Faecal contamination from humans (above-ground tanks)	Human access to tank
Faecal contamination from humans and livestock (below-ground tanks)	Surface water ingress into tank
Mosquitoes	Access to stored water
Lead contamination	Lead-based paints on roofs, lead flashing on roofs, increased corrosion of metals due to low pH from long periods of contact between rainwater and leaves, re-suspension of accumulated sediment, air pollution in metropolitan areas
Other contamination from roof materials	Preservative-treated wood Bitumen-based materials
Chemical contaminants from tanks, pipework, etc.	Inappropriate material that does not comply with Aust. Standards relating to food grade products or products for use in contact with potable water
Dangerous plants	Overhanging branches
Other	Airborne particles carried by wind then settling on the roof

2.4.1. Current first flush practice

The average Australian new residential home in 2003 had a floor area of 228m² (ABS, 2005b). Although not all homes have this size of roof area as they could be double-storey, there is still a substantial amount that would have a roof catchment area of 250 m² given the extra overhang around the house. Current policies (BASIX, etc.) require all new homes to install a rainwater tank. With these new installations, plumbers have connected the tanks in such a way that all the down pipes from around the house connect together before rising out of the ground and into the rainwater tank. Due to the elevation difference between the roof level and the top of the rainwater, tank gravity allows the rainwater to enter the tank through a charged pipe system.

Retail packages of current first flush system for residential houses suggests *'allowing sufficient storage in the first flush device for 1 mm of runoff that drains from the roof. Alternatively, if the roof is considered dirty or littered with debris the instructions suggest allowing 2 mm or more'*. To bypass 1 mm of runoff from the rainwater tank, for each square metre of roof catchment area 1 litre of storage in the first flush system is required. A typical residential house, based on the ABS average floor area above, requires a first flush storage of at least 250 L. From personal observations of more than 100 first flush devices installed on these rainwater tanks, they generally consist of a 100 mm diameter PVC pipe with a maximum length smaller than the height of the tank. Generally, the first flush pipe is a maximum of 1.8 m long, which has a capacity of only up to 14 L. This is completely insufficient for a typical residential house.

2.5. Characterisation of water

2.5.1. Potable water

The Australian Drinking Water Guidelines (ADWG) (2011) describes the limits of parameters deemed suitable for potable grade water. Irrespective of what source the water came from, typically if it complies with the commonly associated limits as provided in Table 2.13, then it is likely to be suitable for human consumption. Note that the limits provided in Table 2.13 are the parameters typically associated with rainwater harvesting, which typically does not include other parameters such as mercury, chromium, pesticides, petroleum hydrocarbons, etc. Other common parameters associated with rainwater harvesting without a limit specified in the ADWG (2011) include total dissolved salts, sodium, potassium, calcium, magnesium, orthophosphate and total organic carbon (TOC).

Orthophosphate is used around the world as a corrosion inhibitor in some potable water supplies, especially where it has been observed that high concentrations of lead or copper exist, sourced from potable water pipelines. Concentrations of orthophosphate are dosed at up to 1mg/L (Edwards et al., 2002; Li et al., 2004) to reduce the metal corrosion in the water distribution pipes.

Table 2.13 Australian drinking water guideline limits

Parameter	ADWG (2011) limit
pH	6.5 - 8.5
Turbidity (NTU)	5
Water hardness (mg/L$CaCO_3$ equivalent)	200
Nitrate (mg/L n)	50
Nitrite (mg/L n)	3
Ammonia (mg/L n)	0.5
Total coliforms (cfu/100 ml)	1
Faecal coliforms (cfu/100 ml)	1
Aluminium (mg/L)	0.2
Copper (mg/L)	2
Iron (mg/L)	0.3
Manganese (mg/L)	0.1
Lead (mg/L)	0.01
Zinc (mg/L)	3

2.5.2. Rainwater

Typical collection and storage of rainwater introduces the potential for chemical, physical and microbial contamination. Heavy metals have become a major concern because their concentrations in rain water tanks were found to exceed recommended levels and therefore making it unsuitable for human consumption (Magyar et al., 2007 and 2008; Han et al., 2006; Simmons et al., 2001). Furthermore rainwater storage tanks accumulate contaminants, and sediments that settle to the bottom. An important consideration of rainwater collection especially for potable purposes is that small amounts of contaminants found in drinking water may have a cumulative and poisonous effect on our health over a life time. This could be true for certain factors that underlie many of the chronic illnesses that are becoming increasingly common in society as the population ages (Barzilay et al., 1999).

Detailed sampling of rainwater tanks was undertaken in Kogarah, Sydney. During a five month period (October 2004 to February 2005) the results indicated that pH, turbidity, total dissolved solids, total sodium, copper, iron, lead, magnesium and zinc were 7.3-7.7, 2.36 NTU, 32 mg/L, 3 mg/L, 0.081 mg/l,0.083 mg/l, 0.005 mg/L, 0.013 mg/L and 0.067 mg/L, respectively. Hydrocarbons, PAH and pesticides were not detected (Sydney Water, 2006).

As the use of rainwater tanks increases, especially in the cities, which already have higher recorded levels of air pollution, interest in the water quality being supplied from the roof has escalated. In particular the subject that has been widely investigated is the occurrence of microbial pollutants because they pose a potentially acute health risk.

Faecal coliforms or E. coli have been commonly been identified in domestic tanks (Thurman, 1995; Victorian Department of Natural Resources and Environment, 1997; Simmons et al., 2001; unpublished results for the SA Water and Victorian Department of Human Services reported in Australian Government Department of Health and Ageing (DHA), 2004). In one survey in Victoria, Campylobacter was identified in six of 47 tanks (Victorian Department of Human Services reported in Australian Government DHA, 2004). Evans (2006) investigated the microbiological and chemical quality of tank-stored rainwater in Newcastle, New South Wales, as to whether there was any contamination from airborne microorganisms as opposed to the common assumptions that rainwater is only impacted directly by roof catchment and subsequent runoff contamination. Evans (2006) indicated that airborne microorganisms did in fact represent a significant contribution to the bacterial load of roof water at the test site, and that the overall contaminant load was influenced by wind velocities, while the composition of the load varied with wind direction.

Watt et al. (2000) state that concerns about the neurotoxicity of lead, particularly in infants and young children, have led to a revision of blood lead levels which are considered to involve an acceptable level of human exposure. This coincides with lowering the limits of lead in drinking water guidelines over the past 20 years from 100 μg/L to 10 μg/L. The introduction of unleaded petrol, the reduction of lead concentrations in paint and the development of standards for materials to be used for rainwater harvesting were expected to reduce the potential for rainwater tank lead contamination. In addition, Sinclair et al. (2005) concluded that the level of metal contamination in rainwater tanks is unlikely to exceed ADWG (2011) values, except when there is a major source of industrial pollution located nearby.

Following Sinclair et al.'s (2005) conclusion, investigated rainwater contamination by modelling pilot tanks situated in a residential area in Melbourne's south-east. Furthermore Magyar et al. (2007) completed further studies of full-sized rainwater tanks located in Melbourne's north and south-east suburbs. These studies demonstrated that even when sampling a few days after a rain event with near full water level in the tank, samples collected from the tank outlets contained heavy metals that exceeded ADWG (2011). Therefore, it appears that the heavy metal, in particular lead contamination, is a widespread problem not localised only to high trafficable and industrial pollution sources.

According to Magyar (2007 & 2008), it is common to find contaminants in Melbourne rainwater tanks that exceed drinking water guidelines. In particular, Magyar was concerned with levels of lead exceeding the ADWG (2011) by up to 35 times the acceptable limit. Other heavy metals that exceeded the ADWG (2011) were aluminium, cadmium, iron and zinc. He believes this is of concern as some states such as South Australia report the highest dissatisfaction with the quality of potable reticulated water supply. In South Australia, 57.2% of households have a rainwater tank installed (ABS, 2010) and potentially a high number of people could be sourcing their potable water from their rainwater tanks.

Han and Mun (2007) investigated particle behaviour to maximize the settling capacity of rainwater storage tanks. It was recognized that sedimentation is an important step in maximising the output water quality of the tank. The actual system configuration and efficient collection of runoff also play key roles in improving water quality. It was observed that the efficiency in removing particles improved by having a considerable distance between the inlet and outlet in the rainwater tank. Furthermore Han and Mun

(2007) recommended that the effective water depth in a rainwater tank be designed to be more than 3 metres and that the rainwater be drawn from as close to the water surface as possible by using a floating suction device.

Changes in pH may also occur in rainwater collected in tanks. Han et al. (2006) reported the results of monitoring rainwater quality, such as pH, turbidity, and metals, for a year, in the rainwater harvesting system at student dormitories at the Seoul National University. The pH of stored water changed to neutral over time, and turbidity and metal concentrations decreased over time through sedimentation. The pH of roof runoff and stored rainwater ranged from 6.5-9.0 and 6.8-8.4, respectively. It was weakly alkaline but was neutralised naturally in the storage tank. The turbidity of the stored rainwater showed a constant range of 1.29-2.35 NTU, and metals were at low levels compared with the South Korean standards for drinking water.

The major factors that affect the design of a rainwater harvesting facility are rainfall pattern, atmospheric conditions, catchment conditions and characteristics, organisation of the rainwater harvesting system, conditions of rainwater use and method of operating the system (Lindberg et al., 1985; Park, 2005; Tanner, 2002; Polkowska et al., 2002; Han et al., 2003). Han and Mun (2007) suggest that if the rainwater harvesting facility is well designed, then it should require little or no electricity, chemicals or maintenance.

2.5.3. First flush and characteristics of roof runoff

When monitoring the characteristics of runoff it is noticed that the pollutant concentrations peak before the flow rate of the runoff peaks. This effect - known as the first flush – occurs when large proportions of accumulated pollutants that are easily disturbed become suspended in the initial portion of the surface runoff on in the storm event. There is limited published data available on first flush characteristics from domestic roof runoff. First flush devices are utilised with rainwater tank installations, however, rarely do these ever come close to the recommended 1 mm of rainfall to be bypassed from the tank.

To date first flush analyse has been primarily focused on the analysis of stormwater runoff from urbanised cities, specifically vehicular trafficable areas. First flush aspect of stormwater is briefly discussed here while the remaining stormwater issues are discussed in Section 2.7. In one investigation Bertrand-Krajewski (1998) quantitatively defined the first flush, which is derived from the analysis of the dimensionless curve of the cumulative pollutant mass versus the cumulative discharge volume, as allowing a non-ambiguous quantification of a phenomenon. Previously this had only been presented in a descriptive or qualitative manner. Bertrand-Krajewski (1998) further states that one can now assume that there is a significant first flush only if at least 80% of the total pollutant mass is transported in the first 30% of the volume discharged during a rain event (Bertrand-Krajewski, 1998).

Studies conducted in south-eastern Queensland on an urban road have demonstrated that in a typical one-in-3 month ARI 10 minute storm event with an intensity of 45 millimetres per hour, a first flush is produced in the first 2 to 4 minutes of runoff. This runoff occurs only after an initial pavement loss of 1 millimetre. The first 4 minutes of runoff equate to 3 millimetres, hence the first 4 millimetres of each storm event produce 3 millimetres of first flush runoff (Barry et al. 2004).

2.6. Methods of water filtration

2.6.1. Granular media filtration

The role of the adsorption media as a pre-treatment to membrane filtration is important as it is relatively affordable and easy to clean or replace compared to early fouling of membrane filtration processes. The ideal filter medium should be of such size that it will provide a satisfactory effluent and retain a maximum quantity of solids, at minimum head loss.

DoA-USA (1986) nominate three types of media filtration setups that can be utilised for pre-treatment: single media, dual media and sometimes mixed media filtration. These types of media are commonly used to treat water and are explained in more detail below:

- Single media: Single media filter generally consists of one medium of sand, anthracite, GAC, etc. Some of the desalination pre-treatment systems also use green sand to remove iron and manganese compounds.
- Dual media: Dual media filter consists of two media with different specific gravity such as sand and anthracite. Usually, less dense media are placed on the top of filter and dense media at the bottom. The use of dual media filters provides a larger quantity of filtered water and less head loss during operation.
- Mixed media: Consists of more than two media such as silica sand, garnet and anthracite. Mixed media filter provides a better coarse to fine filtration arrangement and creates a media flow pattern.

In selecting the filter media consideration is given to achieving best effluent quality such as retaining maximum quantity of solids, minimum head loss and ensuring the system is maintained.

The design of a rapid filter depends on the quality of water to be treated. General filter characteristics are suggested in Table 2.14 (and see DoA-USA, 1986).

Table 2.14 Characteristics of rapid filters:

Characteristics	**Sand filter**	**Anthracite filter**
Filtration rate, m/h	10-20	10-20
Depth of bed, cm	80	80
Particle size, mm	0.35-0.5	0.7-0.8
Max head loss (gravity filter), m	5	5
Max head loss (pressure filter), KPa	200-400	200-400
Backwash rate, m/h	40-50	40-50

2.6.2. Sand

It is well established that sand filters are effective in removing suspended particles and particulate bound contaminants from water and wastewater. For this reason sand is widely used around the world for various water treatment applications. Sand filtration is typically considered a secondary stage treatment which consists of finer particle sedimentation and filtration techniques to remove fine particles and attached pollutants. It belongs to the category of in-transit treatment systems which target the entrained pollutants such as those flowing in stormwater runoff. It does this in two ways: reducing the flow rate through the medium to encourage sedimentation; or by passing the runoff through a porous medium filter. Sand utilised in a slow biological filtration method can also be quite effective in removing Cryptosporidium oocysts and Giardia cysts from water (Graham et al., 1996).

Two key limitations of sand is that it is a non-adsorption media, making it ineffective at removing dissolved constituents (Sansalone, 1999) and that it is prone to clogging. The clogging is due to excess biofilm development and surface deposit. The size of the particles and the filter depth also influence its effectiveness. According to Rodgers et al. (2004), particle size is one of the key parameters to guarantee treatment efficiency as well as the reliability and durability of the system.

According to CSIRO (1999), the typical medium used is coarse washed sand which is readily available at a very economical price. Sand filters can be used for either large or small catchments. Small sand filters are best used in areas of high imperviousness and are generally designed to work in pits or underground chambers (CSIRO, 1999). If the catchment is located downstream of a building site or a highly erosive pervious surface then a sand filter system may not be suitable for this location (NSW EPA, 1997). Small sand filters should be located in areas that are accessible for inspection and maintenance with person or vehicular access (NSW EPA, 1997).

The performance of sand filter systems, with respect to stormwater runoff, will vary depending on the characteristics of the catchment and the quantity of sediments and pollutants in the runoff. The geology of the catchment and soil type can govern how large the filter area needs to be. If the catchment contains a large proportion of clay soils, a larger filter is required to extend the time between maintenance. CSIRO (1999) has estimated the expected filtration performance of a sand filter as shown in Table 2.15 below. Other media that have been used for filtration are peat, limestone and topsoil.

Table 2.15 Estimated treatment performance summary (CSIRO, 1999)

Pollutant	Performance
Gross Pollutants	Low
Coarse Sediments	Medium / High
Medium Sediments	Medium / High
Fine Sediments	Medium
Attached Pollutants	Medium
Dissolved	Low
Installation Costs	Medium / High
Maintenance Costs	Medium / High
Head Requirements	High

NSW EPA (1997) has provided the following table based on experimental investigation by Chiew et al. (1997) and states that the removal rates except for the exception of oxidised nitrogen perform similarly to that of a constructed wetland.

Table 2.16 Pollution retention rates for sand filters (Chiew et al, 1997)

Pollutant	Retention (%)
Suspended Solids	60 – 90
Total Phosphorus	35 – 80
Total Nitrogen	40 – 70
Lead	65 – 90
Zinc	10 – 80
Biochemical Oxygen Demand	60 – 80
Chemical Oxygen Demand	35 – 70

2.6.3. Granular Activated Carbon

Granular Activated Carbon (GAC) is the most commonly used and most effective adsorbent (Vinod et al., 2000; Cimino et al., 2005). GAC is generally used in filters that require high water quality output such as potable grade water although it is relatively high priced. According to Clark and Pitt (1999) activated carbon can be produced from carbonaceous materials with high carbon content by charring or burning. These include materials such as coconut, almond and walnut shells, other woods, peat, lignite and coal.

When exposed to high temperatures the charred particles are activated. This use to be from exposure to an oxidising gas or more commonly today steam. This activation process results in the particles becoming significantly porous which creates the large internal surface area available for adsorption. Most carbonaceous materials have an internal surface area of approximately 10 $m^2.g^{-1}$. The activation process significantly expands the internal surface structure and as a result an internal surface area of 1,000 $m^2.g^{-1}$ is obtained, and even larger under ideal conditions (Allen, 1996).

The GAC filter media can substantially extend the life and operation of a downstream membrane filter by removing suspended solids and by providing a pre-treatment of heavy metals, associated organic and inorganic matter, with some media being also able to adsorb organic and inorganic pollutants, colours and odours. This allows the membrane filtration process to achieve higher removal rates of any remaining heavy metals, organics, pathogens and some viruses for a longer period of time without premature fouling resulting in costly additional physical or chemical treatment (Chaudhary, 2003).

Powdered and granular activated carbons are useful for colour and odour removal and are commonly used in wastewater and potable water treatment, cleaning air emissions, chemical processing, such as solvent recovery, decolourisation and odour removal. Their application, however, is restricted due to their high production costs. As an alternative, the use of low-cost wastes and agriculture by-products to produce activated carbon has been shown to provide an economical solution (Cimino et al., 2005; Ioannidou et al., 2007). According to Srivastava et al. (2005) the cost associated with the activation and regeneration of the carbon, as well as its disposal, are obvious problems associated with using this material. To avoid or reduce the regeneration, activated carbon can be used for biofiltration.

2.6.4. Biological filtration

Biological filtration using GAC is an efficient process in the treatment of drinking water. Many studies showed that GAC biological filters have a great potential in removing disinfectant by-products, biodegradable organic matter and synthetic substances (McKay, 1996). Biological filtration or biofiltration is considered to be one of the most important separation processes available today being utilised to remove organic pollutants from air, water, as well as treating odours from waste gases (Nanda et al., 2011). Even though studies have proven the integrity of the biofiltration process, one which has been utilised for more than a century, it is still difficult to explain theoretically all the biological processes that occur within a biofilter (Chaudhary et al., 2007).

Biofiltration, although simple as a treatment method, involves a number of removal mechanisms. It involves the activities of micro-organisms that are immobilised on the supporting media. Therefore, the filter media and the relevant factors in the development of micro-organisms affect the performance of biological filters. The growth of a microbial community in biofilter is affected by the influent characteristics such as nutrients, toxics, pH and temperature. Operational conditions such as backwashing technique, and empty bed contact time will also affect the performance of biological filtration in water treatment.

Biological filter functions mainly rely on the activities of the micro-organism community attached onto the filter media. The activities of microbes determine the performance of biological filtration. The formation of a biofilm results from attachment and metabolism of biological matter, which includes micro-organisms and macro-organisms. Biofilm is defined as a surface accumulation, which is not necessarily uniform in time or space, and comprising cells immobilized at a substratum and frequently embedded in an organic polymer matrix of microbial origin (Xie et al., 2009). Once bacteria attaches to the filter media (Figure 2.2), it multiplies to produce extracellular polymeric substances, which develop into a viscous slimy gel. The biofilm formation process can be divided into five stages: the formation of conditioning film; bacteria transport; reversible and irreversible adhesion; biofilm development and accumulation; and bio-film detachment (Characklis and Marshall, 1990).

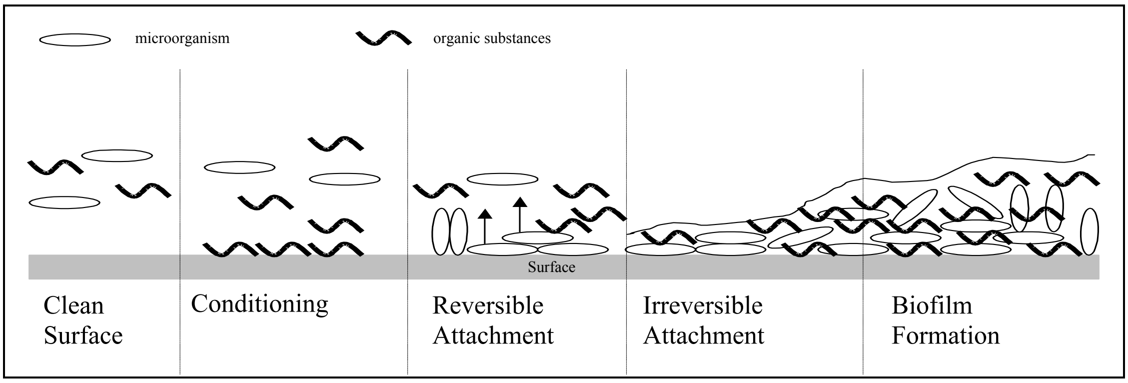


Figure 2.2 Formation of biofilm (adapted from Sheikholeslami, 2007)

According to Hatt (2007) there are several design implications associated with biofiltration. Although it is expected that biofilters provide adequate and ongoing removal of sediment and heavy metals, reductions in phosphorus and nitrogen concentration is affected by the selection and operation of the media. Removal rates can be expected to be as high as 80% phosphorus concentration. However, this is typically only observed if the soil media utilised contains much lower phosphorus content. A second implication lies in the managing of nitrogen pulses that occur upon re-wetting of the media.

Through laboratory experimentation, Hatt (2007) observed that extended dry periods and subsequent re-wetting caused a decrease in the media's infiltration hydraulic capacity. There was some recovery following each dry period, however, during the course of the experiment there was an overall decline. Hatt (2007) explained that this decline was likely caused by a combination of clogging and consolidation of the filter media. Hatt (2007) also observed that removal of sediment, heavy metals and phosphorus was not influenced by wetting and drying and remained consistent regardless of the length of the extended dry weather period.

According to Hatt (2007), the wetting and drying regime had a noticeable influence on effluent concentrations of nitrogen. Significantly higher concentrations were observed upon re-wetting the media following an extended dry period compared to the effluent concentrations recorded during the consistent wet periods. It is believed that these results could have implications for current design practices, since these nitrogen pulses could have significant ecological consequences for downstream receiving waters.

2.6.5. Membrane filtration

The use of membranes as a method of filtration is an attractive advanced technology for the removal of organic matter and controlling disinfection by-products. Reverse osmosis, nanofiltration, ultrafiltration and microfiltration membranes are the most common membrane processes used in water treatment. While nanofiltration membranes are specific in removing particles ranging from 200-1000 Daltons and divalent cations, ultrafiltration and microfiltration membranes are effective in removing pathogens and rather large particulates, about 1,000-5,000,000 Daltons (Clark et al., 2001). Membranes are highly effective in removing organic matter. Membranes are more suitable for smaller systems because the operation and maintenance costs of the membrane system are relatively high.

Recent advances in low pressure driven membrane technologies such as microfiltration (MF) and ultrafiltration (UF) have permitted their use in water treatment due to their high efficiency, ease of operation and small footprint (Qin et. al., 2006). MF includes pore sizes of 0.1 to 10 μm, although they are generally less than 0.45um. For UF, pore sizes generally range from 0.05 down to 0.005 μm (Allgeier, 2005). An earlier study on MF/UF has shown that MF and UF are able to consistently reduce turbidity to less than 0.1 NTU, removing total coliform, bacteria, Giardia and Cryptosporidium (Ebrahim et al., 1997). A clear difference between MF and UF membranes is the MF membranes' inability to remove viruses from water. Viruses, however, can still be effectively removed from water through other treatment means such as ultra violet (UV), ozone or chlorine dosing. Basic comparisons between MF and UF membranes are provided in Table 2.17. The ability of each type of membrane to remove pathogens from water is presented in Figure 2.3.

Table 2.17 Comparing MF and UF Membrane Processes (Wagner, 2001)

	MF	UF
Membrane	Symmetrical Asymmetrical	Asymmetrical
Thickness (µm) **Thin film (µm)**	10-150	150-250 1
Pore size (µm)	4-0.2	0.2-0.02
Rejection of	Particles, clay, bacteria	Macro molecules, proteins, polysaccharides, virus
Membrane materials	Polysulfone (PSO), Polyvinylidenedifluoride (PVDF), Polypropylene (PP)	Polysulfone (PSO), Cellulose acetate (CA), Polyvinylidenedifluoride (PVDF), polypropylene (PP)
Membrane module	Tubular, hollow fibre	Tubular, hollow fibre, spiral wound, plate-and-frame
Operating pressure (kPa)	100-1000	<200

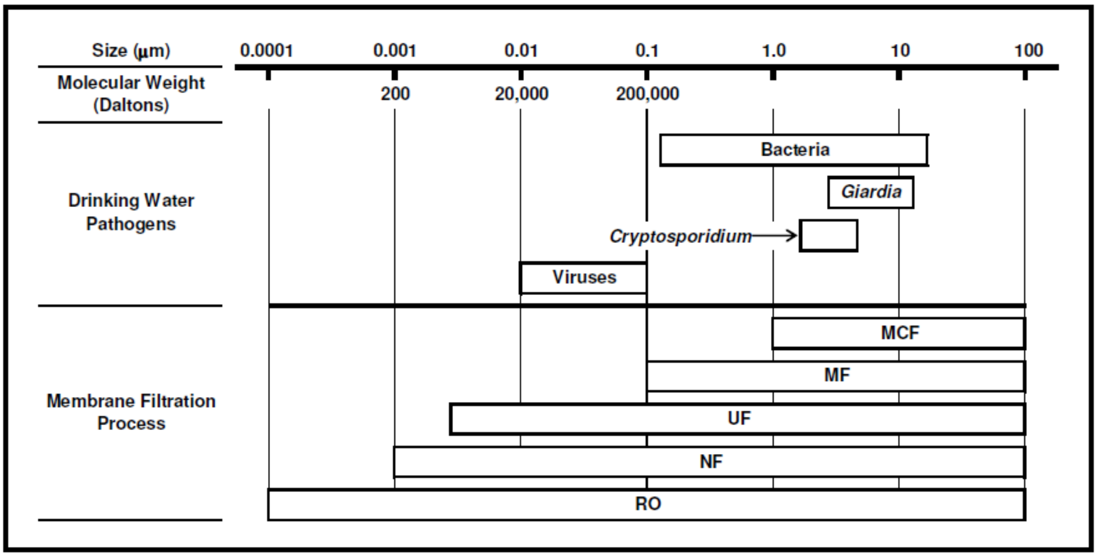


Figure 2.3 Pathogen removal ability of membrane filtration (adapted from Allgeier et al., 2005)

The processes associated with operating a membrane can be dead-end or cross-flow modes of filtration. Energy consumption in the cross-flow mode is higher than the dead-end mode (Glucina et al., 1998). Furthermore, solid removal efficiency in the cross-flow mode is also higher than in the dead-end mode (Tansel et al., 2005). Low-pressure submerged MF/UF systems are presently used successfully from small-scale (0.1 ML/d) to large-scale (375 ML/d) installations.

MF and UF membranes can be formulated in either flat sheet or hollow fibre configurations. Hollow fibre membranes have some advantages over flat sheet membranes. An experiment conducted with flat sheet and hollow fibre MF and UF by Howe et al. (2007) found that flat sheet membranes fouled more rapidly than hollow fibre membranes. Although MF and UF pre-treatment provides an excellent ability to remove pathogens from water, organic fouling remains a problem both for the pre-treatment (MF/UF) and the downstream treatment process. This is due to the existence of small organic molecules which pass even through UF membranes.

Fouling is one of the main disadvantages in membrane filtration processes and it is caused by the presence of pollutants in water. Filter clogging and membrane fouling causes the loss of membrane permeability due to the accumulation of solutes onto the surface of the membrane and/or into its pores. Membrane fouling is generally categorized into four areas of inorganic fouling, namely, particle/colloidal fouling, organic fouling and biofouling (Pontie et al., 2005).

Inorganic fouling is caused by metal hydroxides and carbonates which precipitate on and in the membranes due to changes in water chemistry (Pontié et al., 2005). Particulate fouling occurs when the suspended solids or colloids in the feed water are accumulated onto the surface of the membrane. Colloidal particles present in water range from 10 nm to 10 μm and consist of: firstly, hydrophobic colloidal particles such as clay particle, non-hydrated metal oxides, etc.; and secondly, hydrophilic colloidal particles such as humic acid, fulvic acid, protein, etc. (Binnie et al., 2002). Since colloidal particles constitute a major foulant, it is important to remove them from water before membrane application.

Organic fouling is very common with surface waters containing natural organic matter (NOM.). Organic compounds consist of humic acid, fulvic acid, polysaccharides, and aromatic compounds (Potts et al., 1981). Organic compounds are also energy sources for microorganisms. Scaling is caused by the exceeding solubility of soluble salt and has less effect on the membrane surface which can be controlled by adjusting pH and adding anti-scalants. On the other hand it is very difficult to prevent fouling from colloidal, organic and biological matter.

Biological fouling results from the formation of a biofilm formed by attachment and metabolism of biological matter, which includes micro-organisms and macro-organisms. Biofilm is defined as a surface accumulation that is not necessarily uniform in time or space and comprises cells immobilized at a substratum and frequently embedded in an organic polymer matrix of microbial origin (Characklis and Marshall, 1990). Once bacteria attaches to the membrane (see Figure 2.2) it multiplies to produce extracellular polymeric substances (EPS), which develop into a viscous slimy gel. The biofouling process may be divided into five stages: the formation of conditioning film, bacteria transport, reversible and irreversible adhesion, biofilm development and accumulation, and bio-film detachment (Characklis and Marshall, 1990). Due to low pollutant levels in rainwater compared to stormwater, biofouling may not occur for a rainwater tank filtration device.

Flocculation and adsorption are becoming attractive pre-treatments before the application of membrane filtration. Earlier studies found that flocculation and membrane (microfilter, MF; ultrafilter, UF) filtrations could efficiently remove the NOM from water (Qin et al., 2006; Leiknes, 2009). High rate fibre filters were successfully used in tertiary water treatment. The fibre media consists of bundles of U-shaped fine polyamide fibres. Compared with the conventional rapid sand filter, the filtration velocity of a fibre filter is over five times faster and the specific surface is more than double that (Lee et al., 2006; Lee et al., 2007). Fibre packing combines the two advantages of a large specific surface area and very large porosity (more than 90%) which results in high removal efficiency and low pressure drop despite the high filtration velocity (Lee et al., 2007). In-line additions of flocculants enhance the pollutant removal capacity for both dissolved organics and trace metals.

Yeo Kyu-Seon et al. (2006) studied a reuse system using the membrane process to treat rainwater runoff from an urban parking area containing non-point pollutants. The rainwater reuse system consisted of a pre-filter, membrane and disinfection. A hollow fibre membrane with a pore size of 0.4mm made of polyvinyl di-fluoride (PVDF) was used in this system because of its stable flux and strength. The treated water met all the parameters of the South Korean guidelines standard for reclaimed water treatment. Turbidity was less than 0.3 NTU in the final effluent. COD concentration decreased from 23.0mg/L to 13.1mg/L and BOD_5 decreased from 5.3mg/L to 1.7mg/L after treatment by a pre-filter and membrane process. E. coli was completely removed by this system. While membrane technology can be successfully employed for wastewater reuse, membrane fouling has proved to be a major obstacle in treating rain water (Yeo Kyu-Seon et al., 2006).

Kim et al. (2005) investigated the use of metal membranes for filtration of rainwater. Kim concludes that metal membranes appear to be suitable for clarifying rainwater because of their high treatment efficiency of microorganisms and particulates. However, the filterability depended highly on the rainwater sources, nominal pore size of filter, filtration conditions, and mode of operation. The major fouling mechanism for the metal membrane filtration was observed to be pore blockage. Points of interest that can be drawn from the investigation are:

- Metal membranes efficiently reduced microbial and particulate pollutants in the rainwater. These major pollutants must be removed if the rainwater is to be used for toilet flushing or gardening.

- The 1µm metal membrane filter showed worse permeability than the 5µm metal membrane filter because of its pore size. This was observed to be true of catchments such as roof and roof garden runoffs. Ozone bubbling significantly reduces the increase in the trans-membrane pressure due to membrane fouling while filtering roof garden runoff.

- Flux decline was substantial in continuous operation of the 1µm metal membrane filter where there was no aeration and no recycling of permeate.

2.7. Stormwater

Urban stormwater runoff has been traditionally managed by the concept that stormwater runoff is a nuisance water with no meaningful value as a resource. Consequently the conventional urban stormwater management solution has focused on quickly and effectively removing the stormwater runoff from developed areas through underground pipes and linear engineered overland flow paths, or “out of sight – out of mind”. This viewpoint, as well as the effects of urbanisation, have upset and modified the natural water cycle on which all life forms depend (Dunphy, 2007). The increased rates of stormwater runoff associated with traditional urban development coupled with a dramatic increase in stormwater runoff volume and associated contaminants such as litter, sediments, heavy metals and nutrients, have caused significant degradation of the natural environment (Wong, 2006).

Many pollutants exist in urban stormwater runoff and the major categories include total suspended solids, heavy metals, polycyclic aromatic hydrocarbons (PAHs) and nutrients (Aryal et al., 2010). Pollutants originate from either point or non-point sources. Point sources are specific identifiable locations where stormwater pollution originates. These could include a discharge pipe from a factory or sewage plant. Non-point pollution has been considered one of the major sources of pollution in developed urban areas and is comparatively difficult to identify and control (Drapper et al., 2000; Ngabe et al., 2000). It may include natural processes such as rainfall or snowmelt or from human activities such as litter, use of fertilisers and more. The effect of these pollutants and their likely urban sources found in stormwater are outlined in Table 2.18 (CSIRO, 1999).

2.7.1. Common water quality parameters

Suspended solids and sediments;

Sediment pollutant levels can be measured as Total Suspended Solids (TSS) and/or turbidity (NTU). The term suspended solids (SS) refers to the mass (mg) or concentration (mgL^{-1}) of inorganic and organic matter, which is held in the water column of a stream, river, lake or reservoir by turbulence. Many researchers worldwide during the last 50 years or more have established that suspended solids (SS) is one of the major reasons for water quality deterioration downstream of urban areas leading to aesthetic problems, more costly water treatment, and serious ecological degradation of aquatic environments (Borchardt et al., 1997; Bilotta et al., 2008). Suspended solids in runoff are enriched with several types of organic and inorganic pollutants and during wet weather they are washed off along with associated pollutants. These suspended solids can play an important role in transporting and partitioning chemicals in aquatic ecosystems.

Heavy metals

In urban environments, the term heavy metal usually refers to toxic metals that originate from anthropogenic activities. Some metals are naturally found in the human body and are essential to human health. Iron, for example, prevents anemia, and zinc deficiency in human beings results in growth failure, immune disorders affecting T helper cell 1 (Th1) functions, decreased interleukin-2 (IL-2) production, and cognitive impairment (Prasad et al., 2001). Therefore human and living organisms require some metals in trace amounts to function properly. However, excess levels of these heavy metals can damage human health and ecosystems.

Due to their toxicity, heavy metal discharges into the environment have been regulated by laws throughout the world. Researchers have been investigating heavy metal levels in road and soil particles, river, lake and coastal sediments and in urban runoff. The heavy metals of most concern in the environment are chromium (Cr), nickel (Ni), zinc (Zn), copper (Cu), lead (Pb), vanadium (V), cobalt (Co), cadmium (Cd) and mercury (Hg). They can severely damage organisms and their accumulation over time in the bodies of animals can cause serious illness (Sharma et al., 2005).

Ellis et al. (1982) studied several heavy metals in the sediment on urban street surfaces as a function of sediment particle size. This is important as sediment that becomes airborne can subsequently become associated with rainwater contamination. The distribution pattern of Pb, Cd, Cu and Zn was related to the level and type of traffic densities. Their study showed that concrete motorways had particles less than 250 μm whereas other locations having asphalt surfaces exhibited varying degrees of wear, which provided a considerable amount of free coarse materials.

Nutrients

Nutrients (nitrogen and phosphorus) are essential elements in all aquatic ecosystems. However, excessive levels of these nutrients are often responsible for eutrophication. Eutrophication associated effects can have an adverse impact on the aquatic ecosystems, the most important being the growth of plants and blue-green algae that, upon decaying, deprive the waters of life-sustaining oxygen (Environmental Protection Agency, 1992 & 1993).

Common sources of these nutrients are chemical fertilizers which were applied to agricultural land, lawns, gardens, golf courses, landscape areas, etc. Cooper et al. (1992) and Cherkauer et al. (1989) studied the impact of urbanization on water quality during a flood in small catchments in the urban areas of Milwaukee, USA. They reported a significant response of pollutants to rain in urban basins compared to rural catchments. Several studies have also identified urban stormwater as a major contributor to nitrogen and phosphorus loading to surface waters, second only to agriculture (King et al., 2007). Furthermore, urbanised coastal catchments of Western Australia (WA) have been linked to nutrient enrichment of ground and surface waters (Gerritse et al., 1990). Point sources of nutrients are also major contributors to nutrient enrichment. A study in the south-west region of Western Australia found that a single piggery contributed 24% of the Serpentine River's phosphorus (P) load to the Peel-Harvey estuarine system (Weaver, 1993). Other prominent point sources are wastewater treatment plants.

Table 2.18 Types and causes of urban stormwater pollution (adapted from CSIRO, 1999)

Pollutant	Effect	Urban source
Sediment	Reduces the amount of light in the water available for plant growth and thereby reducing the supply of food to other organisms. Damages sensitive tissues such as the gills of fish. Smothers organisms which live on or in the bed of lakes and streams when suspended material settles out.	Land surface erosion Building and construction sites Organic matter (for example leaf litter, grass) Atmospheric deposition.
Nutrients	Stimulates the growth of aquatic plants, aquatic weeds and algae that may choke lakes and streams and lead to exhaustion of dissolved oxygen levels.	Fertiliser Sewer overflows, septic tank leaks Detergents (car washing)
Oxygen-demanding substances	The drop in oxygen levels may kill fish and other aquatic organisms. Anaerobic conditions may occur and unpleasant odours can result.	Organic matter decay Sewer overflows, septic tank leaks Animal faeces
pH acidity	Increased acidity damages plants and animals	Organic matter decay Erosion of roofing material.

Pollutant	Effect	Urban source
Micro-organisms	Bacteria and viruses can cause illnesses, including hepatitis and gastroenteritis.	Animal faeces Sewer overflows, septic tank leaks
Toxic organics	Can poison living organisms or damage their life processes.	Pesticides Herbicides Sewer overflows, septic tank
Heavy metals	Poison living organisms or damage their life processes in some other way. Persists in the environment for a long time.	Atmospheric deposition Vehicle wear Sewer overflows, septic tank leaks
Gross pollutants (litter, debris)	Unsightly. Animals can eat and choke on this material.	Pedestrians and vehicles Leaf-fall from trees, shopping precincts
Oils, detergents (surfactants)	Highly toxic poison to fish and other aquatic life forms.	Asphalt pavements Spillage, illegal discharges Leaks from vehicles
Increased water temperature	High temperatures are lethal to fish and other aquatic organisms. Elevated water temperatures stimulate the growth of nuisance plants and algae.	Runoff from impervious surfaces Removal of riparian vegetation.

Without appropriate stormwater treatment devices, the quality of waterways will be compromised and can be devastating, not only for aquatic ecosystems but also to community values such as aesthetics, recreation, economics and the health of receiving water bodies.

Changes to the water cycle resulting from conventional stormwater drainage systems and urbanisation have been observed across all regions and have been specifically identified in North America and urban areas in south-eastern Australia (Walsh et al., 2004). To combat the degradation of the natural environment in Australia, the Water Sensitive Urban Design (WSUD) initiative has come to fruition. It is gaining prominence as a contemporary approach for managing urban stormwater to minimise the impact of urban development on waterways and estuaries.

2.8. Water Sensitive Urban Design treatment

Water Sensitive Urban Design (WSUD) is used to preserve water and remove pollutants from stormwater before it reaches downstream drainage systems. The key objectives are to reduce impervious surfaces, treat stormwater runoff and mitigate changes to the natural water balance through on-site reuse of the water as well as through temporary storage, (DEC, 2006). Applying WSUD is most successful when it is considered in the early stages of design so that careful considerations can be given to all aspects of a treatment train's design life. Strategies are now more regularly considering the total life cycles of treatment devices as early as possible to enhance planning and design choices. The parties involved in the planning and design stage of a project are in the best position to reduce risks and plan for the life cycle costs of an asset.

These requirements have been lacking in the past, resulting in poor ongoing performance of WSUD devices which have not had ongoing maintenance and replacement. Local councils are starting to appreciate the additional costs associated with a device that have to be considered over its entire life cycle and are enforcing stricter development consent requirements to ensure long term cost-effective and functional devices are proposed for inland developments. This is to ensure safe operation while in use and during maintenance procedures, easy and cost effective maintenance over the life of the device and effective replacement when it is no longer performing within specification. Below are a range of typical WSUD devices.

Litter baskets: Typically a metal or plastic framed basket installed in an urban gully pit which may consist of only a basket to collect larger litter or sometimes containing a geo-fabric insert to capture smaller pieces of litter. The key advantage is capturing the litter near the source rather than having it accumulate in waterways downstream, although regular maintenance is often costly to local councils. Failure to maintain the baskets can result in blockages followed by localised flooding. They can also restrict the inlet capacities of the pits (DEC, 2006).

Trash racks: Typically a series of vertical metal bars located across the width of a water course. Trash racks can be effective at trapping debris and trash if sized effectively and regularly maintained. However, the disadvantages of trash racks are that they can cause hydraulic restrictions when blocked and can be difficult to maintain if maintenance is not carefully considered at the design stage (DEC, 2006).

Sediment Traps: Typically a pond or tank which is designed to trap coarse sediments from entering a downstream watercourse or basin. Sediment traps are useful in reducing the sediment loadings and can be installed underground in a tank although they are limited in their ability to remove fine sediments, pose a risk of re-mobilising trapped sediments and can become a habitat for mosquito breeding (DEC, 2006).

Gross Pollutant Traps: Typically a sediment tap combined with a trash rack as one serviceable unit often located on the downstream end of a catchment. The advantages often include having a smaller footprint than separated treatment devices and its ability to be retrofitted into existing urban areas such as a park. Disadvantages include the cost of regular maintenance which could be quite high for areas that have high litter rates or many trees prone to dropping leaves. If un-serviced, the device will operate in bypass mode which renders it ineffective and is prone to developing odours (DEC, 2006).

Wetlands and bioretention basins: Constructed wetlands are widely used to control polluted urban stormwater discharges typically consisting of shallow water bodies with extensive amounts of vegetation designed to both treat urban stormwater and control runoff volumes (Greenway, 2005). Wetlands represent a very effective stormwater practice in terms of removal of sediment and pollutants and furthermore they can contribute to flood mitigation, aesthetic value and wildlife habitat (Dallmer, 2002; Wong and Breen, 2002; Georgia Stormwater Management Manual, 2001). Constructed stormwater wetlands differ from natural wetland systems in that they are engineered facilities designed specifically for treating stormwater runoff and typically have less biodiversity than natural wetlands both in terms of plant and animal life. However, as with natural wetlands, stormwater wetlands require a continuous base flow or a high water table to support aquatic vegetation.

2.9. Evaluation of WSUD

The potential benefits and limitations of WSUD are summarised in Table 2.19 for a range of issues including water balance, water quantity and quality, environmental values, and cost. Among the most significant limitations are the following: land footprint required for adequate levels of treatment; ongoing maintenance requirement and costs; and the lack of reliability of water quality of the effluent over a range of influent scenarios.

Table 2.19 Potential Benefits and Limitations of WSUD

Potential Benefits of WSUD
Water balance: ▪ maintains the hydrological balance by using natural processes of storage, infiltration and evaporation. ▪ Aids in groundwater recharge. **Water Quantity** ▪ promotes reuse and recycling of stormwater **Water Quality** ▪ maintains and, where possible, enhances water quality ▪ minimises waterborne sediment loading ▪ minimises the export of pollutants to surface or groundwater ▪ minimises the export and impact of pollutants from sewage **Environmental Values** ▪ protects environmentally sensitive areas from urban development. ▪ restores and enhances urban waterways ▪ minimises the impact on the environment of urban development ▪ can increase the diversity of natural habitats and suburban landscape **Visual/Amenity values** ▪ high visual amenity ▪ opportunities to link community nodes through public open space ▪ protects existing riparian or fringing vegetation **Costs** ▪ may reduce capital costs (pipework and drains) and reduces construction costs (for example grading, tree clearing) ▪ potentially reduces the costs of water quality improvements, by retaining existing waterways ▪ incorporating water sensitive features, water frontages, networked public open space and preserving and enhancing ecological systems tends to make developments more desirable and marketable

Constraints/limitations of WSUD
• potentially increases maintenance and operation costs. Maintenance can potentially reverse the benefits of stormwater treatment by increasing the nutrients loads to the receiving bodies. • WSUD is not effective for removing heavy metals, dissolved nutrients (mainly nitrogen) and pathogens. Stormwater filtration may be a more suitable alternative for gross removal of all pollutants and pathogens. • it may be necessary to supplement WSUD treatments (such as swales) with pipes, to accommodate minor storm events and steep terrain. • opportunities are limited in areas with high water tables, deeply dissected terrain and steep slope, or poor soil and shallow depth to bedrock.

2.9.1. Stormwater harvesting and recycling

Stormwater is an alternative source of potable water to mitigate the water shortage problem. Stormwater treatment has become an important strategy for improving urban water cycle management, given the current and increasing stresses on water resources throughout urban centres of Australia, and much of the world. Expanding the use of stormwater to add to the water supply and reducing water pollution are important objectives in the face of the water crisis. Stormwater is now acknowledged as a valuable resource, rather than an irritant to be disposed of quickly, especially in large urban centres. Harvesting and reusing stormwater offers both a potential alternative water supply for non-potable uses and a means to further reduce stormwater pollution in our waterways.

Studies have shown that a large number of pollutants, both organic and inorganic, may be present in stormwater (Beecham et al., 2011; Kandasamy et al., 2008), both in their dissolved and colloidal forms and associated with particles. Stormwater harvesting and reuse offers a potential alternative water supply for at least non-potable uses. It complements other approaches to sustainable urban water management such as rainwater tanks, the reuse of wastewater and grey-water and demand management. Collectively these areas form the basis of developing sustainable water technologies.

In Australia, water recycling is increasingly a valuable contributor to the conservation of drinking water although stormwater harvesting has been neglected (Dillon, 2004). The consequences of urbanisation are the increase in impermeable area (roads, car parks, paved areas) resulting in more runoff. The average annual volume of urban storm water runoff in Australian cities is almost equal to the average annual urban water usages, of which at least 50% is for non-potable use (Mitchell et al., 1999). The benefits of a successful stormwater harvesting scheme are reductions in: (i) demand for town water, (ii) stormwater pollution loads to downstream waterways and estuaries, and (iii) stormwater volumes and discharges. Stormwater pollution is a major source of pollution in receiving water, for example Sydney Harbour and Melbourne's Port Phillip Bay. Stormwater in the 1990s contributed 94% of sediments and 50–60% of nutrients to Sydney Harbour (NSW Premiers Dept., 1997).

Stormwater harvesting and reuse offers a potential alternative water supply for at least non-potable uses. It complements other approaches to sustainable urban water management such as rainwater tanks, the reuse of wastewater and grey water and demand management. Collectively these areas form the basis of developing sustainable water technologies.

Stormwater Treatment Targets and Objectives

To date the analysis and evaluation of stormwater quality has been on the basis of surrogate components. Further assessments have predominantly targeted the impacts on receiving water and not for reuse purposes. Table 2.20 summarises the indicative values for a range of non-potable uses including residential non-potable applications such as toilet flushing, irrigation, construction applications, and fire-fighting. Additional criteria may be applicable for more specific application such as industrial reuse strategies.

The three aspects of stormwater quality of particular relevance to stormwater harvesting and reuse schemes are (DEC, 2006):

- pathogens, including faecal coliforms and E. coli – for public health implications.
- chemical constituents - for public health and environmental considerations, and some end-use requirements (e.g. irrigation).
- suspended solids and turbidity - for their potential impact on both the effectiveness of disinfection and the function of irrigation schemes.

Table 2.20 Stormwater indicative targets criteria for reuse application (DEC, 2006)

Application	Typical Indicative Targets
Level 1: Reticulated non-potable residential uses (e.g. garden watering, toilet flushing, car washing)	E. coli <1 cfu/100 mL, Turbidity ≤ 2 NTU, pH 6.5–8.5, pathogen reduction[B]
Level 2 (with human exposure). Spray or drip irrigation of open spaces, parks and sportsgrounds, dust suppression, construction site, ornamental water-bodies, fire-fighting.	E. coli <10 cfu/100 mL, Turbidity≤2 NTU, pH 6.5–8.5, pathogen reduction[B], TP < 0.05 mg/L[A], TN< 5 mg/L[A]
Level 3 (no human exposure). Spray or drip irrigation or subsurface irrigation of open spaces, parks and sportsgrounds, Industrial uses–dust suppression, construction site.	E. coli <1000 cfu/100 mL, pH 6.5–8.5, TP < 0.05 mg/L[A], TN< 5 mg/L[A]

A: Indicative values for long term irrigation,

B:1 mg/L Cl2 residual after 30 minutes or equivalent

High rate treatment systems

WSUD treatment devices such as wetlands, bioretention, and permeable pavement are widely used in stormwater management. According to Hatt et al. (2006) the current WSUD devices used in Australia for stormwater pollution control do not provide reliability in water quality necessary for recycling due to their inadequate removal efficiency for a range of inflow conditions. Furthermore, for adequate levels of treatment, WSUD devices require a considerable amount of land area which is often not available in urban inner city areas.

Stormwater discharge is relatively high and therefore needs to be treated at a high rate. These treatment systems have been used successfully in water and wastewater treatment and include fibre filters, deep bed filters and biofilters (Singh et al., 2012). The alternative is to store the stormwater before treatment. Raw stormwater in storage has low value and will degrade under anaerobic and anoxic conditions while pre-treatment of stormwater adds value to the stored water and can be beneficially reused.

Following high rate treatment, the effluent stormwater is of similar quality to roof rainwater and both can be stored in the same tank (Singh et al., 2012). This increases the contributing catchment from just the building roof to the whole site increasing the amount of water that can be captured and stored for reuse. In medium scale developments rainwater tanks typically empty quite quickly due to demand and often are augmented by town water.

High rate treatment systems can be used to create a sustainable urban development with a low demand on town water, low stormwater pollution export and reduced stormwater discharges. By products of the treatment process (concentrated pollutants and sludge) can be discharged to the sewer alleviating sludge disposal problems and is attractive in creating a low maintenance system. Utilising these treatment systems for water reuse can significantly reduce the stormwater pollution exported from a site that is then transported downstream into the receiving water.

Chapter 3

University of Technology, Sydney

Faculty of Engineering & Information Technology

3. Experimental Investigation

3.1. Introduction

This chapter describes in detail the materials used and procedures for the various experiments conducted in this study. Chapter 3 provides details on the materials, equipment, methodology, and procedures used in this study for the; Chapters 4 and 5 identify the issues with rainwater through the characterisation of rainwater and first flush and water demand analysis; Chapters 6 and 7 laboratory testing and designing the rainwater treatment system; Chapter 8 operation of stormwater filtration system enabling fine tuning and configuration of a pilot study system utilising a readily available comparable water supply; and Chapter 9 pilot study of a rainwater filtration system.

3.2. Experimental materials

3.2.1. Rainwater

Location of metropolitan rainwater tanks

In this study, detailed sampling and experiments were carried out on metropolitan rainwater tanks located in different parts of Sydney, New South Wales and in Wollongong, a town located south of Sydney (Figure 3.1). The topography of the Sydney basin is that of a classic "closed" basin, bounded by high ground to the south, west and north, and by the temperature differential between land and ocean which is on the eastern side. From early morning onward, air pollution is generated from primary sources (industry, road transport, etc.) and collects over the Sydney basin. Offshore afternoon sea breezes, typically from the north-east, pick-up this air pollution and smog and carry it inland where it concentrates in the south-western corner of Sydney. Air quality is the worst in the state capital's south-western suburbs and it is here where most of the rain water tanks selected for sampling were located. Rainwater tanks along the path of the onshore air current were also selected for sampling.

Table 3.1 summarises the characteristics of these rainwater tanks. The tanks ranged in age from 1 to 50 years, in size from 500 L to 120,000 L, and were constructed from a variety of materials including PVC and concrete. They collected water from concrete tiled, Zincalume, fibro and Colorbond galvanised roofs.

Location of rural rainwater tanks

In addition to the data collected from metropolitan rainwater tanks, detailed sampling was carried out on rural rainwater tanks located approximately 160 km south-west of Sydney (Figure 3.2). Their characteristics are listed in Table 3.2. The rainwater tanks ranged in age from 10 to 25 years and were constructed from various materials including PVC, concrete and galvanised steel. These tanks all collected water off Colorbond galvanised roofs. The houses were located in a rural country area with less vehicular activity compared to Sydney's metropolitan area. As there was no town water supply, all residents relied on these rainwater tanks as their main household water supply.

Location of first flush rainwater tank and sampling regime

First flush sampling was carried out at a rainwater tank located in the south-western corner of the Sydney basin, specifically the suburb of Ingleburn (T1, Table 3.1, and Figure 3.3) where air pollution is the worst. The rainwater tank was 2 years old and collects water off concrete roof tiles from a 30-year old house, which was located near an industrial area and a freeway. The tank was made from polyethylene and was plumbed using PVC fittings from the gutter to the tank. At the time of the experiment, the first flush device was disconnected from the rainwater tank system so that a true representation of the first flush runoff from the roof could be collected.

Location of the pilot scale rainwater tank

The rainwater tank used for filtration experiments was located in the Sydney metropolitan basin at Peakhurst (T17, Figure 3.1). The tank specifications are listed as T17 in Table 3.1. Further details are provided in Section 3.4.

Figure 3.1 Locations of metropolitan rainwater tanks in Sydney (Google Maps, 2011)

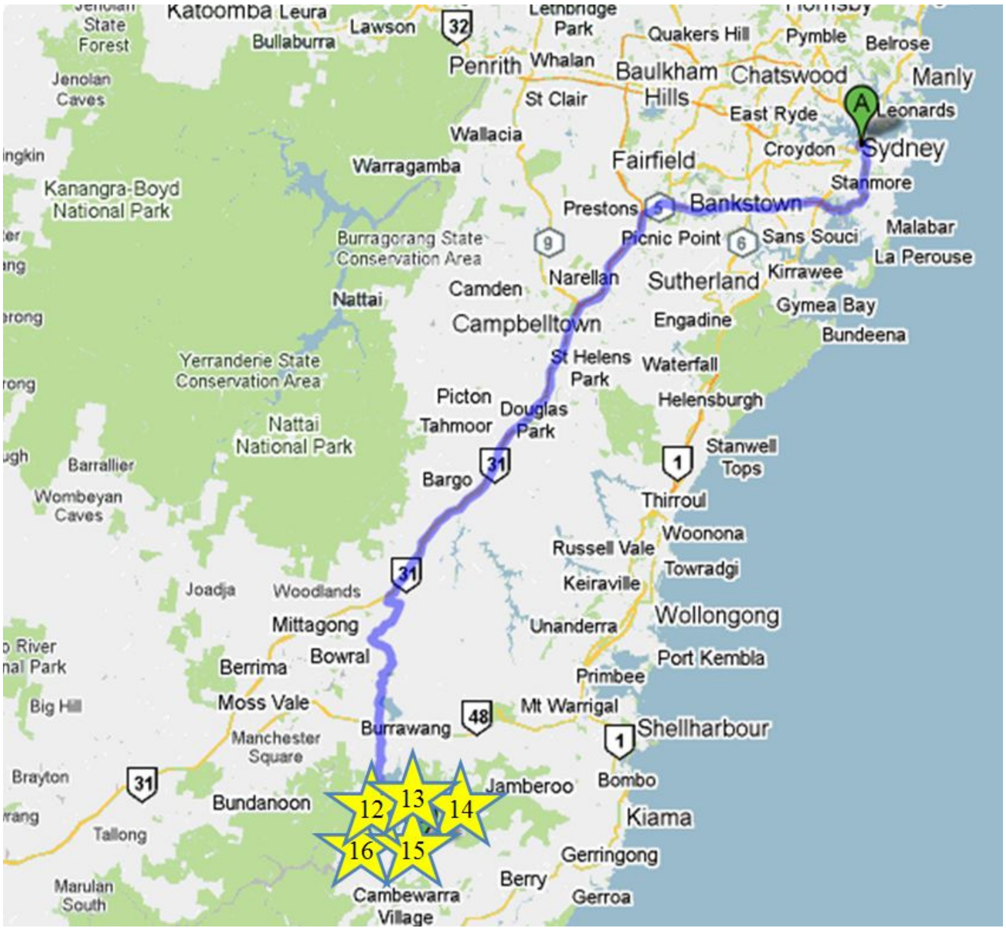


Figure 3.2 Location of rural rainwater tanks in relation to Sydney (Google Maps, 2011)

Table 3.1 Characteristics of Metropolitan Rainwater Tanks

Location in Sydney, Fig. 3.1	**ID #**	**Number of Samples taken**	**Tank Material**	**Tank Size (L)**	**Age (year)**	**Tank Serviced/ Maintained**	**Roof Catchment**
Ingleburn	T1	3	PVC	500	2	No	Concrete Tile
Wollongong	T2	3	PVC	3,000	2	No	Concrete Tile
Kogarah	T3	3	PVC	9,000	3	No	Fibro
Mount Hunter	T4	3	Concrete	30,000	30	No	Concrete Tile
Kirkham	T5	3	PVC	6,000	2	No	Concrete Tile
Narellan	T6	3	PVC	2,500	3	No	Colorbond Galvanised
Kemps Creek	T7	3	PVC	5,000	2	No	Concrete Tile
Cawdoor	T8	3	Concrete	20,000	50	Yes	Concrete Tile
Theresa Park	T9	3	Concrete	120,000	5	Yes	Zincalume
Glen Alpine	T10	3	PVC	1,500	1	No	Concrete Tile
Newtown	T11	3	PVC	3,000	1	No	Colorbond Galvanised
Peakhurst	T17	3	PVC	3,000	5	No	Concrete Tile

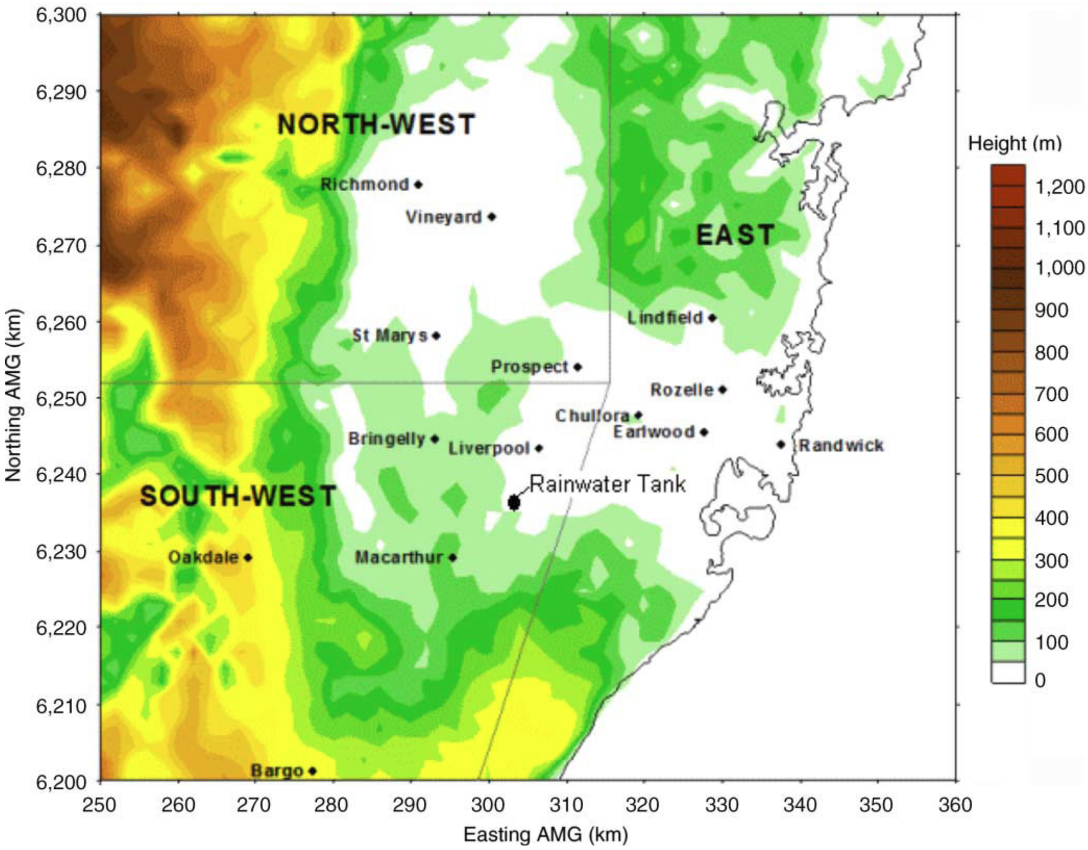


Figure 3.3 Location of first flush rainwater in relation to the Sydney Basin (DECCW, 2009)

Table 3.2 Characteristics of Rural Rainwater Tanks

Location in Sydney, Fig. 3.2	ID #	Number of Samples taken	Tank Material	Tank Size (L)	Age (year)	Tank Serviced/ Maintained	Roof Catchment
Kangaroo Valley	T12	2	PVC	2 x 8,000	25	No	Colorbond Galvanised
Kangaroo Valley	T13	2	PVC	35,0000	15	No	Colorbond Galvanised
Kangaroo Valley	T14	2	Galvanised Steel	2 x 20,000	15	No	Colorbond Galvanised
Kangaroo Valley	T15	2	Concrete	80,000	20	No	Colorbond Galvanised
Kangaroo Valley	T16	2	PVC	10,000	10	No	Colorbond Galvanised

3.2.2. Laboratory water quality analysis

Methods of analysis

Detailed laboratory analyses were carried to determine pollutants existing in the sampled water. The pollutants analysed were: firstly, heavy metals (aluminium, arsenic, cadmium, chromium, copper, iron, lead, manganese, mercury, nickel, selenium, silver and zinc); and secondly, mineral salts otherwise known as cations and anions (calcium, magnesium, chloride, potassium, sodium and sulphate). Other parameters measured were nitrate and nitrite, pH, ammonia, orthophosphate, conductivity, hardness, turbidity, total suspended solids, total dissolved salts and bicarbonate. The testing methods are summarised in Table 3.3. The laboratory analyses were carried out primarily by the Environmental Analysis Laboratory (EAL) at Southern Cross University (SCU) and the Author at the University of Technology Sydney (UTS). Additional analysis was also cross checked using Australian Laboratory Services (ALS). Averages of results were calculated by determining the mean.

Turbidity

Turbidity was measured using a 2100P turbidity meter (HACH, USA). Results indicating a suspension value were displayed in NTU (Nephelometric Turbidity Units). Samples for this measurement were taken from a plastic hose for the influent and from outlet pipes connected to each experimental column.

Table 3.3 Water quality parameters and measurement methods (Eaton et al., 2005)

Parameter	Measurement Method
Heavy metals (aluminium, arsenic, cadmium, chromium, copper, iron, lead, manganese, mercury, nickel, selenium, silver and zinc)	APHA 3120 ICPMS - Inductively Coupled Plasma - Mass Spectrometry
Chloride	APHA 4500-CL^{-} - G - Mercuric Thiocyanate Flow Injection Analysis
Nitrate	APHA 4500 NO_3^{-} - F - Automated Cadmium Reduction Method
Nitrite	APHA 4500 NO_2^{-} - B - Colorimetric Method
Mineral salts (calcium, magnesium, potassium, sodium and sulphate)	APHA 3120 ICPOES - Inductively Coupled Plasma - Optical Emission Spectrometry
pH	APHA 4500-H+ - Electronic Method
Ammonia	APHA 4500 NH_3-H - Flow Injection Analysis
Orthophosphate	APHA 4500 P-G - Flow Injection Analysis for Orthophosphate
Conductivity	APHA 2510-B - Laboratory Method
Water Hardness	Calcium & Magnesium Calculation
Turbidity	APHA 2130 - Nephelometric Method
Total Suspended Solids	GFC equiv. filter - APHA 2540 - D - Total Suspended Solids Dried at 103°c - 105°c
Total Dissolved Salts	Calculation using EC x 680
Bicarbonates	Total Alkalinity - APHA 2320 - Titration Method

* APHA - American Public Health Association

Molecular weight distribution

Samples were collected and pre-filtered using a 0.45µm micro-filter attached to a syringe. The samples were then analysed for rainwater organic matter (RWOM) in terms of molecular weight distribution (MWD) using high performance liquid chromatography (HPLC). High pressure size exclusion chromatography (HPSEC, Shimadzu Corp., Japan) with a SEC column (Protein-pak 125, Waters Milford, USA) was used to determine the MW distributions of RWOM. Standards of MW for various polystyrene sulphonates (PSS: 210, 1800, 4600, 8000, and 18000 Daltons) were employed to calibrate the equipment. Table 3.4 below shows the relationship between the size in nm and MW in Daltons.

Table 3.4 Relationship between the size in nm and MW in Daltons

Size (Daltons)	Size (nm)
500 *	0.394
1,000 *	0.496
5,000 *	0.846
7,000 *	0.946
10,000 *	1.065
20,000 *	1.341
100,000 *	10.0
500,000 *	50.0

* The equation used to compute the size is: Size (µm) = $\frac{0.0001*(MW)^{0.3321}}{2}$ (Mulder, 1996)

Total organic carbon analysis

The primary measurement conducted for filtration experiments aimed to find total organic carbon (TOC). TOC concentration of water was measured using Multi N/C 2000 analyser (Analytik Jena AG) (Figure 3.4). The sample was oxidized into end products in the combustion tube under high temperatures of 700 – 950^{0}C in a process described by the following equations:

$R + O_2 \longrightarrow CO_2 + H_2O$ (Eq. 1)

$R\text{-}N + O_2 \longrightarrow NO + CO_2 + H_2O$ (Eq. 2)

$R\text{-}Cl + O_2 \longrightarrow HCl + CO_2 + H_2O$ (Eq. 3)

Where *R* is substance containing carbon

The amount of CO_2 is quantified by a non-dispersive infrared gas reactor (NDIR) and calculated to give total carbon (TC). Inorganic carbon is determined by reactions between the sample and acid in the total inorganic carbon (TIC) reactor. The TOC value (mg/L) is then determined by Equation 4:

$TOC = TC – TIC$ (Eq. 4)

Figure 3.4 Photograph of Multi N/C 2000 analyser (Analytik Jena AG)

3.2.3. Granular Media

Adsorption media

The role of the adsorption media as a pre-treatment to membrane filtration is important as it is relatively affordable and easy to clean or replace compared to fouling in the membrane filtration processes. The ideal filter medium should have pore sizes that will provide a satisfactory effluent and retain a maximum quantity of solids with minimal head loss. Single medium, dual media and mixed media filters are widely used in water treatment.

Sourcing granular media materials

Granular activated carbon, anthracite and sand were all sourced from local suppliers in the Sydney region. The materials were purchased in bulk to ensure the grade and quality remained uniform throughout the experiments.

Granular activated carbon (GAC) filter media

The GAC filter media (Figure 3.5) can substantially extend the life and operation of a membrane by removing suspended solids and by providing a pre-treatment of heavy metals, organics, colours and odours. This pre-treatment can allow the membrane filtration process to achieve higher removal rates of any remaining heavy metals, organics, pathogens and some viruses for a longer period of time without premature fouling resulting in costly additional physical or chemical treatment. The properties of the GAC utilised in all experiments are shown in Table 3.5.

Anthracite filter media

The Anthracite filter media (Figure 3.6) is a non-adsorption media which has been widely used in conventional filters along with other media such as sand. The properties of the anthracite utilised in all experiments are shown in Table 3.6.

Sand filter media

The Sand filter media (Figure 3.7) is a non-adsorption media which has been widely used in conventional filters. Sand filters are effective in removing Cryptosporidium oocysts and Giardia cysts from water (Graham et al., 1996). The properties of the sand utilised in all experiments are shown in Table 3.7.

Table 3.5 Physical properties of Granular Activated Carbon (GAC)

Specification	Estimated Value
Iodine number, mg/(g.min)	800
Maximum ash content	5%
Nominal size, m	3×10^{-4}
Maximum moisture content	5 %
Bulk density, kg/m^3	748
BET surface area, m^2/g	1112
Average pore diameter, Å	26.14

Table 3.6 Physical properties of Anthracite

Parameter	Anthracite
Effective Size (mm)	1.0-1.1
Uniformity Coefficient	1.30
Acid Solubility	<1%
Alkali Solubility	1.5%
Hardness (Hardgrove Grindability Index Friability)	50 20% max for 15 minutes
Durability	Attrition loss < 0.35% per year
Specific Gravity	1.45
Bulk Density (kg/m^3)	660 to 720

Table 3.7 Physical properties of Sand

Parameter	Sand
Effective Size (mm)	0.55-0.65
Uniformity Coefficient	<1.5
Acid Solubility	<2%
Specific Gravity	2.65
Bulk Density (kg/m^3)	1500

Figure 3.5 Photograph of Granular Activated Carbon

Figure 3.6 Photograph of Anthracite

Figure 3.7 Photograph of Sand

3.2.4. Micro-filtration membranes

Membranes are available in numerous grades, sizes, materials, qualities and costs. With appropriate selection, membranes can effectively and efficiently remove microbial and particulate pollutants from sources of water. Membrane filtration experiments were carried out using a stainless steel membrane from Steriflow (Table 3.8), two polymeric membranes from INGE Watertechnologies AG (Table 3.9) and Ultra Flo (Table 3.10). These were tested in a dead-end mode of filtration.

The Steriflow Filtration has a surface area of 0.03 m^2 and pore size of 0.3 μm (Table 3.8). The patented Multibore® membrane developed by INGE Watertechnologies AG combines seven individual capillaries in a single fibre within a highly resistant supporting structure (Table 3.9). This arrangement significantly increases the stability of the membrane and reduces the possibility of fibre breakage. The Ultra Flo has a surface area of 0.3 m^2 and pore size of 0.1 μm (Table 3.10).

Table 3.8 Physical properties of Steriflow stainless steel micro-filtration membrane

Name	Membrane
Manufacturer	Steriflow Filtration System
Material	Metal - Stainless Steel
Pore Size (μm)	0.3
Membrane Dimensions (mm)	450 Long, 20 Dia.
Filter Area (m^2)	0.03
Method	In - out

Figure 3.8 Photograph of Steriflow membrane inside of Perspex column

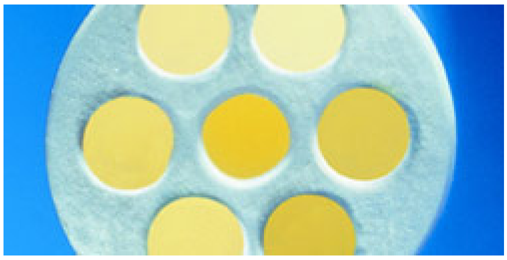


Figure 3.9 Photograph of INGE Multibore® membrane (INGE Watertechnologies AG 2010).

Table 3.9 Physical properties of INGE Watertechnologies AG Multibore micro-filtration membrane.

Item	**Characteristics**
Membrane manufacturer	INGE Watertechnologies AG, Germany
Material	PESM
Nominal pore size	0.02µm
Outer diameter	4.3 mm
Inner capillary diameter	0.9 mm
No. of capillaries	7
No. of fibres	12
Length of fibre	500 mm
Surface area	0.1 m^2

Table 3.10 Physical properties of Ultra Flo hollow fibre micro-filtration membrane.

Name	**Membrane**
Membrane Manufacturer	Ultra Flo
Material	Polysulfone
Pore Size	0.1 μm
Outer diameter	1.9 mm
Inner diameter	0.7 mm
No. of fibres	40
Length of fibre	400 mm
Filter area	0.3 m^2
Method	Out - in

3.2.5. Filter columns

The filter columns were constructed using either 20 mm or 100 mm diameter Perspex tubing, clear plastic and rubber hoses, plastic stopcocks and a range of sealants and adhesives. This ensured that the filter columns for adsorption and membrane experiments functioned adequately. The diameter of the columns selected for each experiment were based on the required flow rates and are detailed in each experiments methodology. The samples were collected from the stopcocks and the incremental tap heights were utilised for maintaining the required water within the column. Overflows from the columns were returned back to the feed tank. Media was packed into the column and pre-saturated with distilled water prior to commencement of an experiment. The specifications of the filter columns are listed below in Table 3.11.

Table 3.11 Physical properties of filter columns utilised in experiments

	Column 1	**Column 2**
Column height (mm)	500	2000
Internal diameter (mm)	20	100
Media height (m)	0.3	0.3
Flow rate (m/hr)	1	1 - 5
Incremental tap locations	100	250

3.2.6. Monitoring equipment

Flow measurement

The influent and effluent water flows to the filter columns were monitored both manually by using a stopwatch and measuring cylinder, and automatically using a flow meter and data logging equipment (Table 3.12). This parameter was constantly controlled and recorded along with every sample collected. The flow was controlled using inline stopcock valves.

Head loss

The head loss was monitored using a calibrated manometer. The level of water in the manometer was observed and recorded manually. The head loss development is the difference in the manometer reading taken at the top and bottom of the filter media.

Data logging

Data logging equipment, flow meter and pressure transducer were employed to monitor the flow rates and trans-membrane pressures associated with experimental procedures. The equipment items are specified in Table 3.12.

Table 3.12 Specifications of data logging equipment

Name	**Data Logger**	**Flow Meter**	**Pressure Transducer**
Manufacturer	Endress & Hauser	Endress & Hauser	Endress & Hauser
Item Code	Eco Graph T RSG30	Promag 10H	PMC 131
Specifications	6 Analogue Channels 3 Digital Channels	0 – 500 mL/minute	0 – 200 kPa
Accuracy	N/A	1.8% – 0.3 % from 0 – 500 mL/minute	3.5 %

3.2.7. Pilot Scale Cartridge Rain Filter

A rainwater filtration experiment was carried out using a cartridge-based GAC filter and membrane filter from Watts (redistributed by Ultra Flo, Figure 3.10 and Table 3.14). The features and benefits of using the cartridge system are: (i) quick and easy installation even for retrofitting to an existing water supply; (ii) the cartridges are removed with a single turn; and (iii) a double o-ring seal prevents water leakage and results in no pressure problems. The manufacturer's specifications are outlined in Table 3.13.

This membrane filter was operated in dead-end mode. The membrane has a surface area of 0.4 m^2 and pore size of 0.1 μm (Table 3.14and Figure 3.11).

Figure 3.10 Photograph of cartridge filtration system

Table 3.13 Cartridge Filter Specifications

Type	Microns	Maximum Lifetime Treatment Capacity (L)	Media Volume or Membrane Area
Granular Activated Carbon	5	5700	0.400 L
Hollow Fibre Membrane (MF)	0.1	5700	0.400 m^2

Table 3.14 Physical properties of Watts (Ultra Flo) hollow fibre micro-filtration membrane

Name	Membrane
Membrane Manufacturer	Watts (Ultra Flo)
Material	Polysulfone
Pore size	0.1 μm
Outer diameter	0.45 mm
Inner diameter	0.25 mm
No. of fibres	1000
Length of fibre	300 mm
Filter area	0.4 m^2
Method	Out - in

Figure 3.11 Photograph of cartridge filtration system

3.2.8. Design and construction of filter column filtration system

A suitable filter column holding system was constructed so that multiple filter columns could be contained within the small footprint in the Kogarah stormwater harvesting plant (see Section 3.3.4). A thick stainless steel base plate mounted on an aluminium frame was constructed as a platform for the filter columns (Figure 3.12).

Figure 3.12 Construction of filter column filtration system – Constructing and reinforcing the base plate for stability

As this filtration system was located in a damp environment exposed to raw water of varying quality, the entire system had to be painted to prevent corrosion. To add mobility to the system four wheels were incorporated so the filtration system could easily be moved in and out of the harvesting plant (Figure 3.13).

Figure 3.13 Construction of filter column filtration system – Painting of the base

plate to protect against corrosion and add wheels for mobility

After completing the base of the filtration system two aluminium poles were installed and attachment brackets were constructed to allow six filter columns to be attached to the system (Figure 3.14).

Figure 3.14   Construction of filter column filtration system – Constructing and assembly of supporting towers and filter column attachment brackets

The brackets were attached to the two poles and the entire system was under-coated to prevent corrosion (Figure 3.15). A final mat black coat of paint was applied to the completed system before the filter columns were attached.

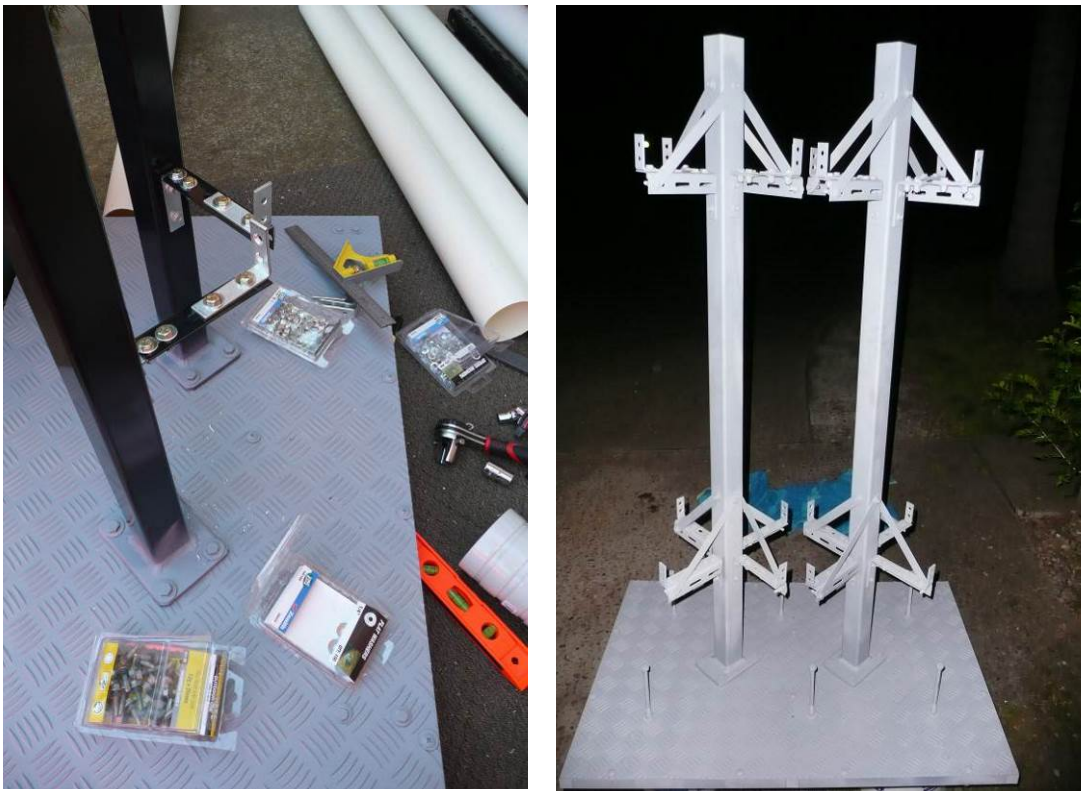


Figure 3.15 Construction of filter column filtration system – Constructing and assembly of filter column attachments

The six filter columns were connected to the support system and installed in the cabinet of the harvesting plant (Figure 3.16). The hydraulic lines were then connected to the columns to allow the influent water to pass through the filter columns as discussed in each experiments methodology for the filtration process (Figure 3.17).

Figure 3.16 Filter column filtration system in stormwater harvesting plant cabinet

Figure 3.17 Hydraulic flow lines of filter column filtration system

3.3. Experimental methods

3.3.1. Characterisation of rainwater

The characteristics of rainwater collected in tanks at various locations in Sydney are analysed in detail in Chapter 4. The three areas in which rainwater tank characterisation was conducted were:

- Characterisation of rainwater that collected in metropolitan rainwater tanks;
- Comparison of the metropolitan results with a characterisation of rainwater that collected in rural rainwater tanks; and
- Determining whether a first flush exists in rainfall runoff from the roof of a metropolitan residential house and whether diverting it from the tank could help improve rainwater tank quality.

Water quality measurements in terms of physical, chemical and organic characteristics were taken and compared against drinking water standards (ADWG, 2011). The molecular weight distribution (MWD) analysis of organic matter in rainwater was undertaken to see how it compares with potable water, bottled water and rainwater before it came into contact with the roof. The effect of ageing of rainwater in the tank was also investigated.

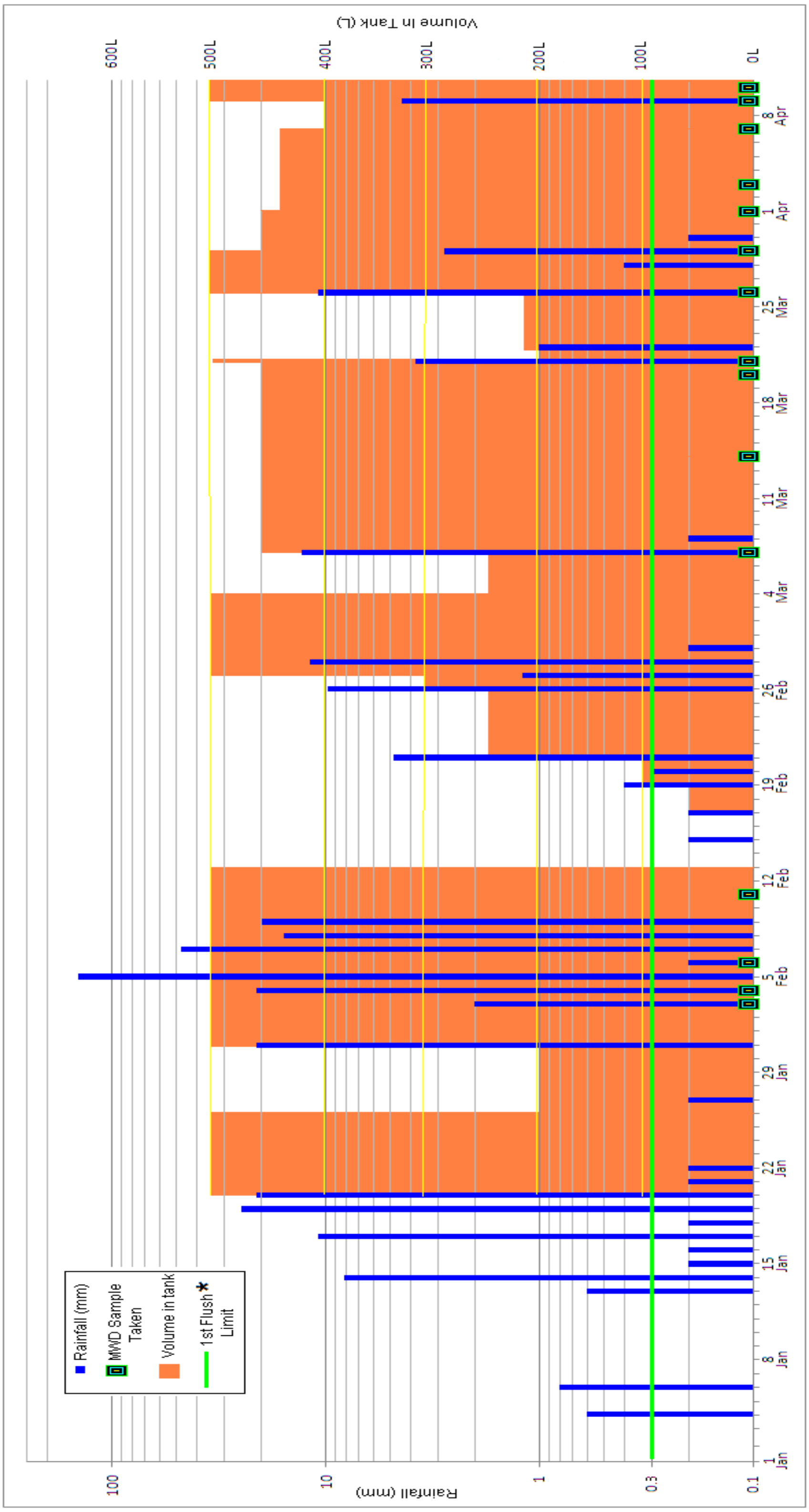
Characterisation of metropolitan rainwater tanks

In this study, detailed sampling was carried out on rainwater tanks located in different parts of metropolitan Sydney and Wollongong (Figure 3.1). Table 3.1 summarises the characteristics of these rainwater tanks. Most tanks were retrofitted to existing houses and therefore were between 1 and 5 years of age except for T4 and T8 which have been in use for 30 and 50 years, respectively.

The rainwater tank (T1) where most of the detailed sampling (including first flush sampling) was carried out was 2 years old and collects water off concrete roof tiles of a 30-year old house at Ingleburn in south-west Sydney. The house is located near an industrial area and a freeway. The tank is made from polyethylene and is plumbed using PVC fittings from the gutter to the tank.

Sampling was carried out 3 times at each rainwater tank to determine the average rainwater quality in the tanks; the sampling details and methods are listed in Section 3.2.2. Samples were usually taken between 3 to 7 days after a rain event. The water samples were collected from either the tap located on the base of the rainwater tanks or the closest tap to the tank if a tank tap was not available. Following each storm event, the contents of the first flush system was discarded and resealed ready for the next event. Additional sampling was conducted at T1 at Ingleburn to compare the rainwater in the tank with the rain before it came into contact with the roof. Samples from the potable main water supply (Sydney Water) were also collected and analysed for comparison.

Molecular weight distribution (MWD) samples were also collected regularly from tank T1 after a storm event; 4 days, 8 days and longer after a storm event (Figure 3.18). The samples collected at T1 were analysed for rainwater organic matter (RWOM) in terms of MWD using a high performance liquid chromatography (HPLC). The rainwater samples were pre-filtered using 0.45μm microfilter attached to a syringe.



* denotes approximate depth of rainfall required to fill first flush system prior to filling the rainwater tank

Figure 3.18 Ingleburn rainwater tank ID 1 rainfall and tank volume with MWD date in 2008

Characterisation of rural rainwater tanks compared with metropolitan rainwater tanks

In addition to the data collected from the rainwater tanks sampled in the metropolitan area, detailed sampling was done on rural rainwater tanks located approximately 160 km south-west of Sydney (Figure 3.2) with their characteristics listed in Table 3.2. Sampling was carried out twice at each rainwater tank location to determine the average rainwater quality in the tanks. The sampling details and methods are listed in Section 3.2.2.

Characterisation of the first flush in a metropolitan rainwater tank

The first flush, a term commonly used in stormwater management, occurs when large proportions of pollutants are transported in the first part of the rainfall runoff. Because these pollutants are easily disturbed they become suspended in overland surface runoff early on in the storm event. Due to the lack of experimental data for quantifying the first flush runoff from a residential roof, this study aims to measure concentrations in the first flush runoff of pollutants commonly associated with rainwater tanks.

Samples of runoff from a residential roof (T1) located in the metropolitan area during rain events were collected and analysed. The rain events, summarised in Table 3.15, that were analysed for this investigation had to comply with the following conditions:

- A previous significant storm event washed the roof;
- At least a two-week dry period followed the significant storm event; and
- Adequate and consistent rainfall occurred after the two-week dry period to enable at least 5mm of first flush sample to be collected for laboratory analysis.

The samples were collected from the downpipe approximately every 2.5 minutes for up to 25 minutes (Table 3.15). The records of both one on-site electronic weather station and one on-site manual rain gauge were examined to determine how many millimetres of rain fell between during the collection of each sample. Samples were taken directly from the downpipe from the gutter before the first flush system (Figure 3.19). Samples of rainfall (before coming into contact with the roof) were also taken to document the concentration of pollutants in rainfall. The sampling details and methods are listed in Section 3.2.2.

Table 3.15 Summary of rain events that were monitored for first flush runoff

Date	**Rain Start Time**	**Number of preceding dry days**	**Rain Duration (minutes)**	**Total Rainfall (mm)**	**Average Intensity (mm/hr)**
15/11/08	19:37	15	24	5	13
11/01/09	18:40	22	83	3.5	3
9/02/09	22:58	19	11	3	16

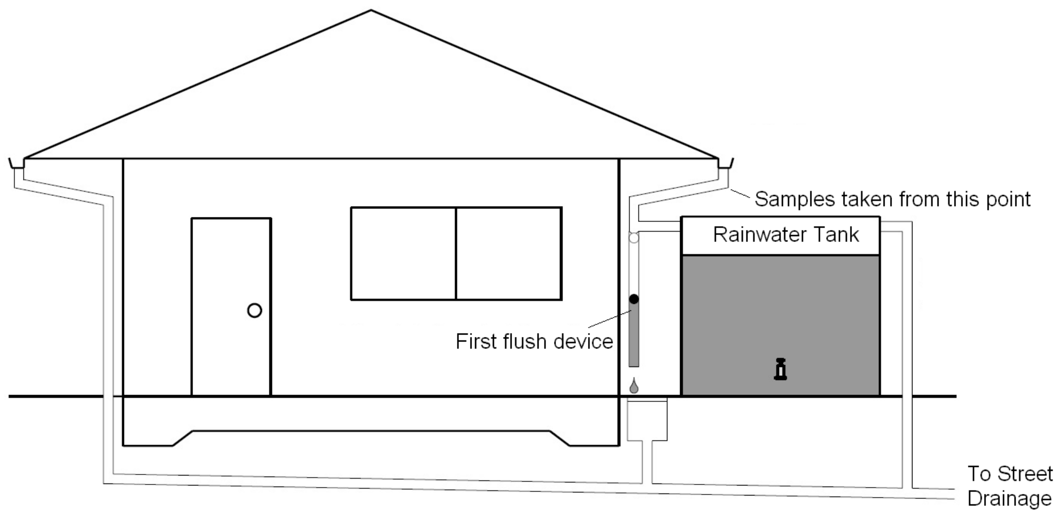


Figure 3.19 Schematic diagram of typical isolated rainwater tank and drainage system in a contemporary Australian house

3.3.2. GAC adsorption filtration

Long-term experiments of GAC adsorption were carried out as pre-treatment for rainwater. Rainwater for these experiments was sourced from tank T1 (Table 3.1).

Laboratory GAC adsorption filter system

Detailed laboratory experiments were conducted using Filter Column 1 (Table 3.11). This column was 500 mm in length with an internal diameter of 20 mm. The column contained tap junctions at 100 mm increments along both sides of its length with an open top for the influent hose and a tap junction at the base for the effluent (Figure 3.20). This experiment utilised GAC adsorption filter media (Table 3.5) which was packed in the column up to a depth of 300 mm. The flow rate was regulated at 5 mL/min (which equated to 1 m/hr through the column). The water table above the GAC adsorption media was kept at 100 millimetres in height with the overflow excess sample being gravity drained back into the raw feed tank (Figure 3.20).

The raw feed tank was situated in the laboratory room and constructed from similar polyethylene as the original rainwater tank source water. The feed tank contained a volume of 300 L which was periodically topped up from the residential dwelling's rainwater tank (T1) when required. As such, the uniformity of the feed water varied over time as periodic rainfall occurred and topped up the rainwater tank. Sampling of the influent water was done at the same time as that of the effluent water, in order to monitor any changes in the rainwater over time. Sampling of the influent and effluent water was carried out according to the procedures detailed in Section 3.2.2.

During the filter operation, backwashing was applied to remove excess particles that can cause the filter to clog. The backwashing was conducted only when the flow rate of the filter was not able to achieve 5 mL/min or 1 m/hr by pumping tap water in an upward flow direction from the bottom of the column. During the backwashing process, the overflow pipe was rerouted from the feed tank to a waste tank. The GAC adsorption bed was expanded up to approximately 30% for 2 minutes. Excess free particles were removed from the column through the overflow pipe along with the tap water. This experiment was repeated a second time due to a pump failure that led to the premature ending of the first experiment (see Chapter 6). The results for both experiments are presented in Chapter 6.

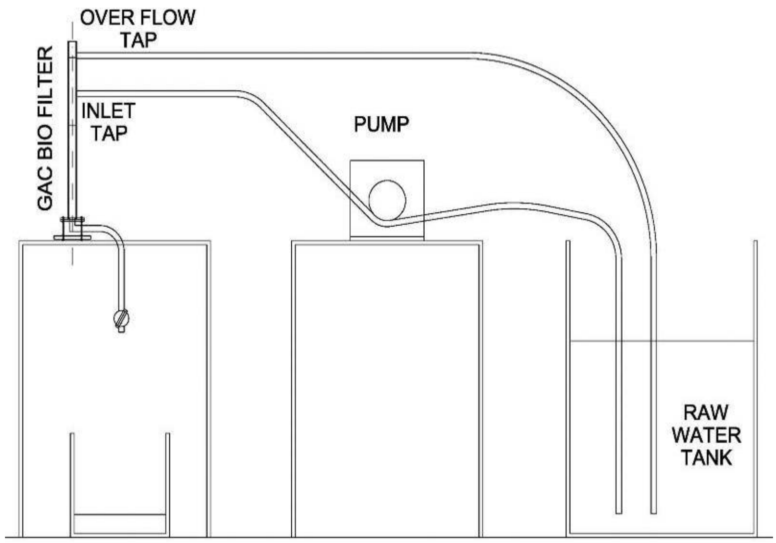


Figure 3.20 Schematic diagram of GAC adsorption filter apparatus

Pilot scale pre-rainwater tank inline GAC adsorption filter

An experiment was conducted by connecting a GAC adsorption filter column to the downpipe of the residential dwelling to assess its actual performance while operating during actual rainfall events. The apparatus set-up is shown in Figure 3.21.

The filter media housing was constructed from stormwater grade PVC piping with a media chamber diameter of 150 mm and an inlet and outlet pipe diameter of 90 mm to match existing residential plumbing. Both the inlet and outlet contained a stainless steel wire mesh with 0.5 mm gaps to contain the GAC adsorption media. Sampling points were installed before and after the filter media to measure the pollutant parameters in the influent and effluent during a rainfall event. Sampling of these parameters was carried out according to procedures detailed in Section 3.2.2.

The filter chamber was installed with a 'U' bend to ensure that the GAC adsorption media remained saturated at all times between rainfall events (Figure 3.21). The GAC adsorption chamber component of the filter was 50 cm long with a total media volume of 8.84 L. The flow rate through the filter media was variable as this was governed by the intensity and duration of rainfall that occurs during each rainfall event.

A rain gauge was used to monitor rainfall events to document the approximate intensity and amount of rainfall that had fallen. A first flush system was also installed prior to the GAC adsorption filter to remove the first 12 L of roof runoff that generally contained leaves and a higher concentration of pollutants. Since this experiment relied on actual rainfall the times and amount of flow through the filter could not be planned. Furthermore because actual rainwater was used, the influent pollutant concentrations also varied, providing a realistic scenario in which GAC adsorption could be tested.

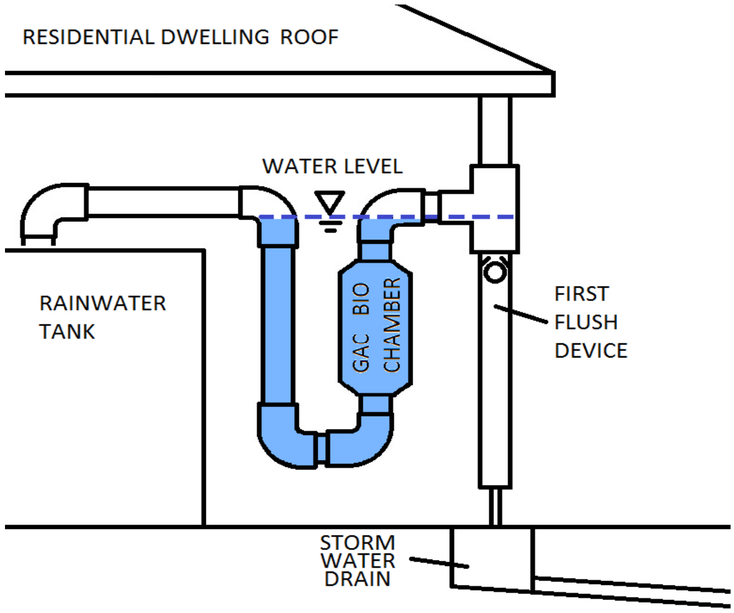


Figure 3.21 Schematic diagram of pre-rainwater tank inline GAC adsorption filter

3.3.3. Membrane micro-filtration

Membrane filtration experiments were carried out using a stainless steel membrane from Steriflow Filtration Systems (Table 3.8), two polymeric membranes from INGE Watertechnologies AG and from Ultra Flo (Tables 3.9 and 3.10, respectively). These systems were tested in a dead-end mode of filtration with and without pre-treatment. This was done to quantify the impact of pre-treating rainwater in terms of membrane fouling, filtration performance and flux decline.

Clean membrane flux determination of micro-filtration membranes

Flux determination was undertaken to test the membranes' flow characteristics in their clean membrane state under various pre-set trans-membrane pressures. This made it possible to determine their suitability in treating water in a rainwater tank. The membranes were prepared in accordance to the manufacturer's specifications and tested using distilled water as feed water. During these experiments the filter column (filter column 2,

Table 3.11) was not filled with media. It was used to maintain a constant head over the membrane. The testing of the membrane for each pre-set pressure was analysed for a suitable period of time until a relatively stable flux reading was observed. The preferred range of water head was up to a maximum of 20 kPa (2 m) as this is about the limit available in above ground rainwater tanks at residential dwellings.

The stainless steel Steriflow Filtration Systems membrane (Table 3.8) was first tested with a pre-set gravitational head pressure of 20 kPa (2 m of head) (Figure 3.22). As there was negligible flux under gravity head the experiment was repeated under a pump pressure of 50kPa (5 m of head) (Figure 3.23).

The two polymeric membranes from INGE Watertechnologies AG Multibore and from Ultra Flo hollow fibre (Tables 3.9 and 3.10, respectively) were also tested with a gravitational head pressure at 20 kPa (2 m of head). The apparatus for the gravitational flux test for INGE Watertechnologies AG Multibore and Ultra Flo hollow fibre are shown in Figure 3.22.

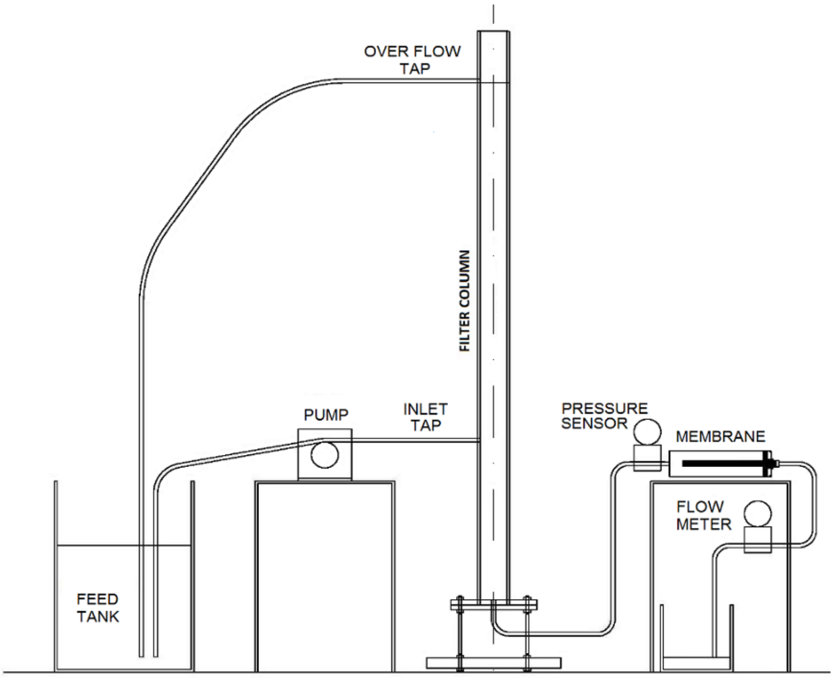


Figure 3.22 Schematic diagram of clean membrane flux determination under gravitational head. Note pump and filter column were used to simulate a constant gravity head.

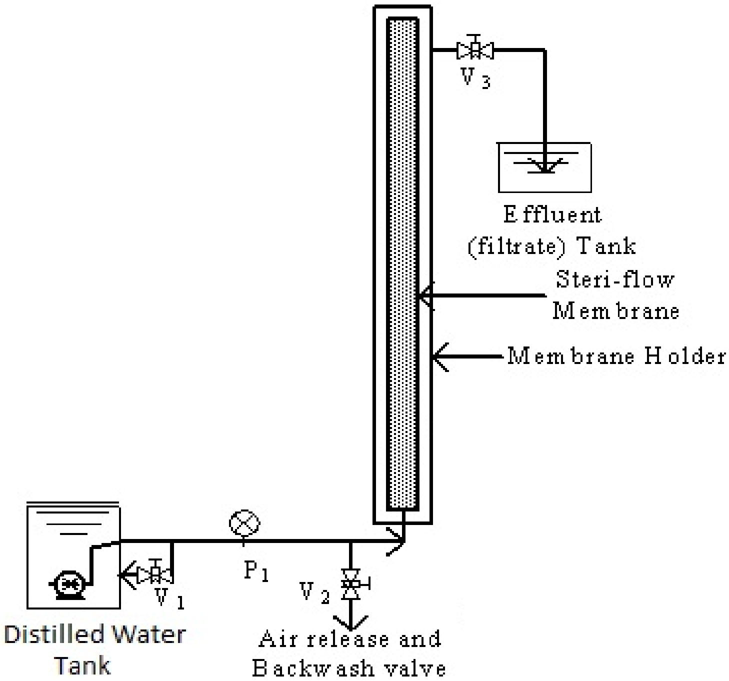


Figure 3.23 Schematic diagram of Steriflow Filtration System micro-filtration clean membrane flux determination under pumped head.

Ultra Flo hollow fibre micro-filtration membrane under gravitational head

Membrane filtration experiments were carried out using a polymeric hollow fibre membrane (Ultra Flo, Table 3.10). The experimental apparatus set-up is shown in Figure 3.22. At the beginning of each experiment the clean membrane flux was measured using distilled water at 10 kPa to ensure it was consistent with a clean membrane. For these checks distilled water was used as the feed water. Furthermore the filter column was not filled with media. Instead it was used to maintain a constant head over the membrane.

A membrane filtration experiment was carried out using raw rainwater (no pre-treatment) as the feed water (Figure 3.22) from rainwater tank T1 (Table 3.1). Membrane filtration experiments were then repeated using raw rainwater pre-treated with GAC filter as the feed water (Figure 3.22). The raw rainwater was first passed through a pre-treatment of GAC adsorption in filter Column 2 (see Section 3.2.5). During these experiments the filter column was filled with GAC (see Table 3.11). The water level in the filter column was used to maintain a constant head over the membrane. The experimental set-up allowed testing under various pre-set water heads. The water level in the filter column was kept at a constant pre-set level so that the pressure on the membrane was under a constant pressure that in different experiments ranged from 5 to 20 kPa (Table 3.16). The pre-treated water was gravity-fed to the membrane which was positioned horizontally (Figure 3.22).

Water analysis samples were collected at each stage of the process including the influent raw rainwater sample, the pre-treated water sample (in experiments where pre-treatment was applied) and the effluent after passing through the membrane as per the sampling procedures listed in Section 3.2.2.

Table 3.16 Flux decline gravitational head pressures and GAC analysed for Ultra Flo hollow fibre micro-filtration membrane with and without pre-treatment

Ultra Flo Filter Membrane	With GAC	Without GAC
Flux Decline Pressures (kPa)	1.5, 10, 20	5
Flux Decline Head (m of head)	0.15, 1, 2	0.5
GAC Depth (m)	0.3	-

INGE Watertechnologies AG Multibore micro-filtration membrane with GAC adsorption pre-treatment under gravitational head

Membrane filtration experiments were carried out using a polymeric multibore micro-filtration membrane from INGE Watertechnologies AG (Table 3.9). This system was configured to operate as a dead-end mode of filtration and the experimental apparatus set-up is shown in Figure 3.22. Raw rainwater was utilised as the feed water in these experiments; it was firstly pre-treated with GAC filter. During these experiments filter Column 2 (see Section 3.2.5) was filled with GAC (see Table 3.11). The experiments were conducted under a constant pre-set pressure ranging from 3 to 20 kPa (Table 3.17). The remainder of the experimental details were identical to those for the Ultra Flo membrane filtration test described in the preceding section.

Table 3.17 Flux decline gravitational head pressures analysed for INGE Watertechnologies AG Multibore micro-filtration membrane with GAC adsorption pre-treatment

INGE Filter Membrane	With GAC
Flux Decline Pressures (kPa)	3, 10, 20
Flux Decline Head (m of head)	0.3, 1, 2
GAC Depth (m)	0.3

3.3.4. Stormwater harvesting pilot scale plant

Kogarah Council, a local government authority in Sydney, introduced the Carlton Industrial Sustainable Water Program (CISWP), in part to minimise potable water consumption through the Carlton industrial area. One major part of the CISWP was the design and installation of a stormwater harvesting plant at Carlton, Kogarah (Figure 3.24). The development of the stormwater harvesting plant is described in Appendix B.

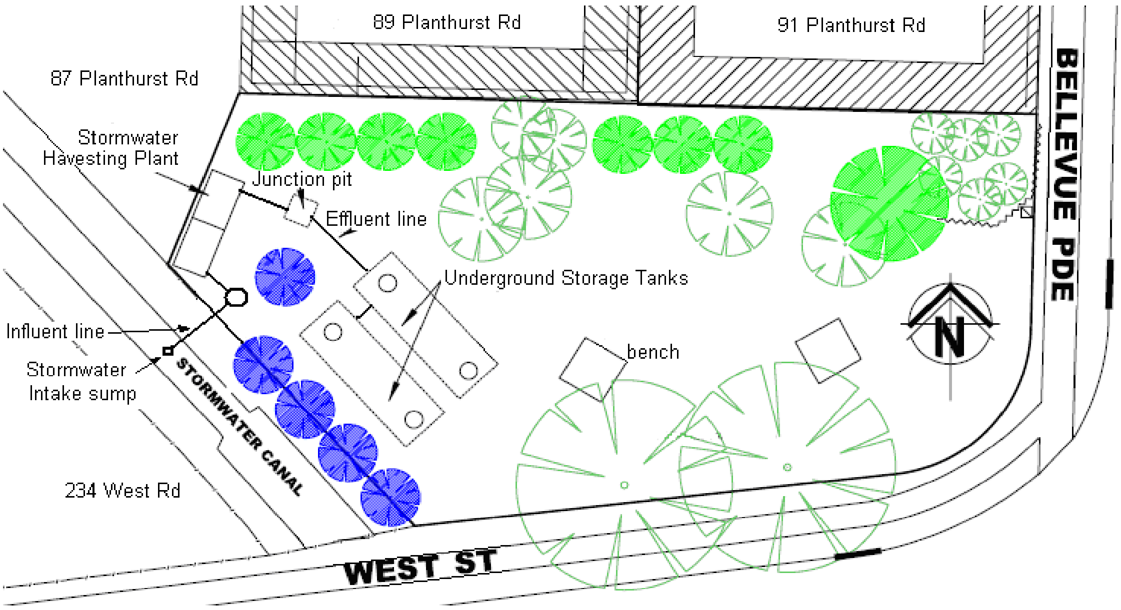


Figure 3.24 Carlton Stormwater Harvesting Plant, Kogarah, Sydney

Raw water samples were collected from the stormwater harvesting plant facility located at the Lower West Street Reserve, Carlton, in Sydney. The stormwater that was harvested originated predominantly from base flow which constantly flows in the stormwater canal between rainfall events (Figure 3.24). The stormwater drains via gravity through a sump pit in the floor of the stormwater canal to an adjacent wet well. It is then pumped through a control valve pit which monitors the turbidity levels for filtration suitability. If the turbidity is greater than 50 NTU, the water is then diverted back through a return pit to the canal. Otherwise it proceeds to the stormwater filtration plant at a rate of 0.7 L/s or 2.5 kL per hour. Water parameters of the raw stormwater were comparable to the quality of water collected in rainwater tanks (Tables 8.1 to 8.4).

Steriflow membrane pilot scale experiments

Experiments were carried out using a membrane filter and granular medium adsorption filtration at Carlton with raw stormwater from the stormwater harvesting plant. The media used was GAC and the membrane was the Steriflow stainless steel membrane. The characteristics of GAC and Steriflow stainless steel membranes are summarised in Tables 3.5 and 3.8, respectively.

The GAC filter system consisted of columns configured in parallel to provide sufficient flow rates and these were operated in two scenarios - pre-treatment and post-treatment. The height of the GAC in the filter column was 1 m with an internal diameter of 100 mm. The flow rate through the columns was 10 m h^{-1}. The filter columns were backwashed at the end of each day’s operation for 60 s which proved to be satisfactory in maintaining less than 1 bar (100 kPa) of pressure across the columns (Figure 3.25). Figure 3.16 depicts the GAC filter column system.

The Steriflow membrane filter was operated by a circulation pressure pump and it functioned in cross-flow mode. The Steriflow filtration system utilised automated cleaning procedures including back-pulsing and back-flushing. The back-pulsing method operated for 0.08 s every 3 s and back-flushing operated for 1 s every 4 min. The clean membrane flux of the system was 250 L/hr/m^2. The circulation bleed valve was partially opened to prevent the retentate’s cross-flow concentration from continuously increasing (Figure 3.26). The bleed rate was regulated at approximately

1.25 L/min. Following the completion of an experiment the entire filtration system was purged of pre-treated feed water and retentate water, then cleaned and backwashed with clean tap water to ensure clean membrane flux starting conditions for each subsequent experiment.

The pre-treatment experiments were conducted using GAC pre-treated stormwater collected from the stormwater canal in Carlton and stored in a pre-treated stormwater tank (Figure 3.25). It was then pumped into the Steriflow feed tank and onto the Steriflow membrane system filtration (Figure 3.26). The post-treatment experiments were conducted utilising raw stormwater collected from the stormwater canal in Carlton and treated with the Steriflow membrane system (Figure 3.26). The Steriflow filtrate water was stored in a stormwater tank which was then pumped to provide the GAC filter with the required feed supply

The pollutants and the water quality parameter concentrations were measured according to standard methods (Eaton et al., 2005) and methods listed in Section 3.2.2. Total organic carbon (TOC) concentration of raw water and treated water was measured using the Multi N/C 2000 analyser (Analytik Jena AG).

The composition of dissolved organic carbon (DOC) matter was measured using Liquid chromatography-organic carbon detection (LC-OCD). LC-OCD categorizes the classes of organic compounds into raw and treated water. All samples were filtered through a 0.45 micro-filtration as a pre-filter before being analysed in the LC-OCD.

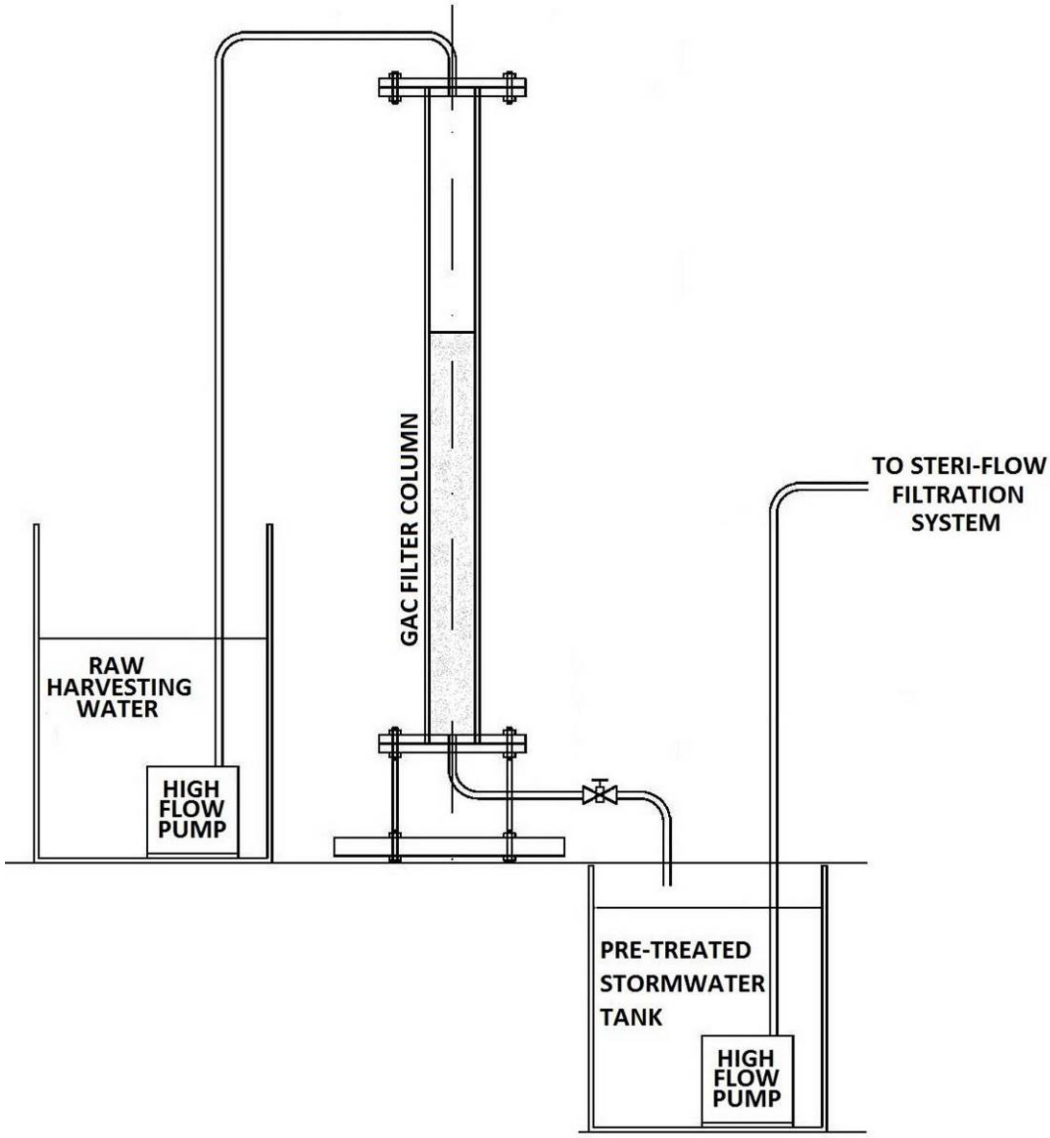


Figure 3.25 Schematic diagram of GAC media filter column stormwater pre-treatment

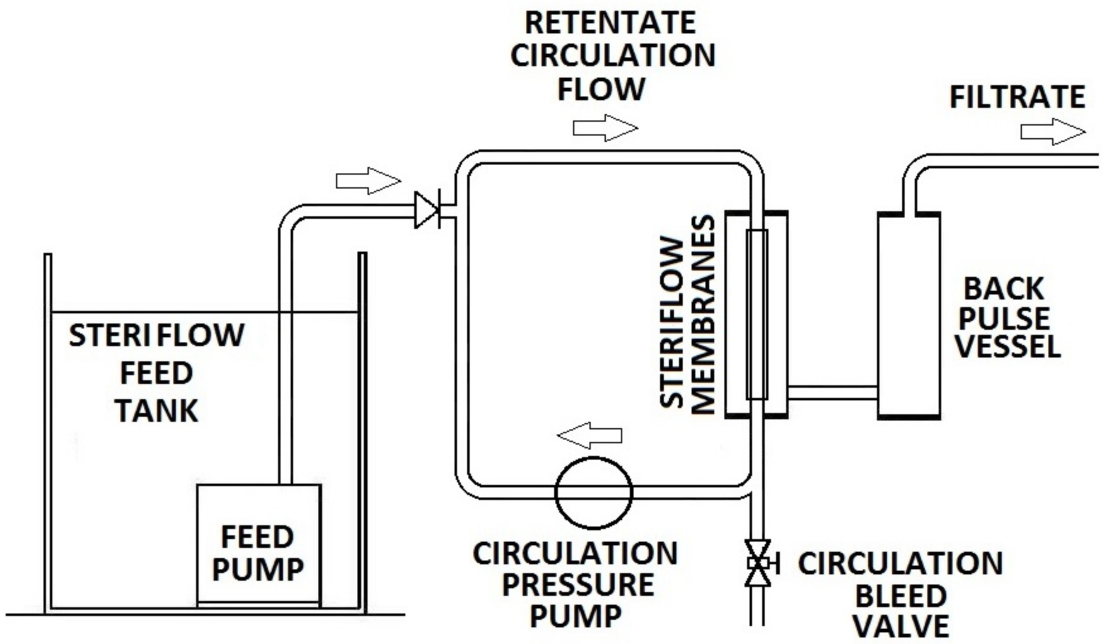


Figure 3.26 Schematic diagram Steriflow filtration system in cross-flow configuration with circulation bleed valve reducing retentate concentration.

Stage media filtration for stormwater treatment

Experiments were carried out using a membrane filter and granular medium adsorption filtration at Carlton, Sydney, with raw stormwater from the stormwater harvesting plant.

Experiments were conducted with granular medium filter packed and a membrane filter (Figure 3.27). The media used were GAC, anthracite and sand (Tables 3.5 to 3.7) and the membrane was the Ultra Flo membrane (Table 3.10). The raw water was pumped to the filter column. The effluent from the filter column was passed through the membrane filter, which was under a 2 m gravity head. The membrane operated under dead-end conditions. Further experiments were conducted with 2 filter columns (anthracite filter and GAC filter in series (Figure 3.28) and 3 columns in series (anthracite filter, sand filter and GAC filter in series (Figure 3.29).

The height of the medium in the column was 1 m. The flow rate through the columns was 10 m/hr. The columns were run continuously for 4 hours per day for three consecutive days. The filter columns were backwashed at the end of each day of operation for 60 seconds which proved to be satisfactory for maintaining less than 1 bar of pressure across the columns. Figure 3.16 depicts the granular medium column filtration system.

The granular medium filter (GAC, anthracite or sand) column can typically operate at a relatively high filtration rate (10 m/hr). By contrast, the flux of the submerged membrane filtration (Ultra Flo membrane) is relatively low. To facilitate the much lower rate of membrane filtration, while maintaining a constant driving head of 2 m, an overflow system was installed as shown in Figures 3.26 to 3.28.

The pollutants and the water quality parameter concentrations were measured according to standard methods (Eaton et al., 2005) and methods listed in Section 3.2.2. Total organic carbon (TOC) concentration of raw water and treated water was measured by using the Multi N/C 2000 analyzer (Analytik Jena AG).

The composition of dissolved organic carbon (DOC) matter was measured using Liquid chromatography-organic carbon detection (LC-OCD). All samples were filtered through a 0.45 micro-filtration as a pre-filter before being analysed in the LC-OCD.

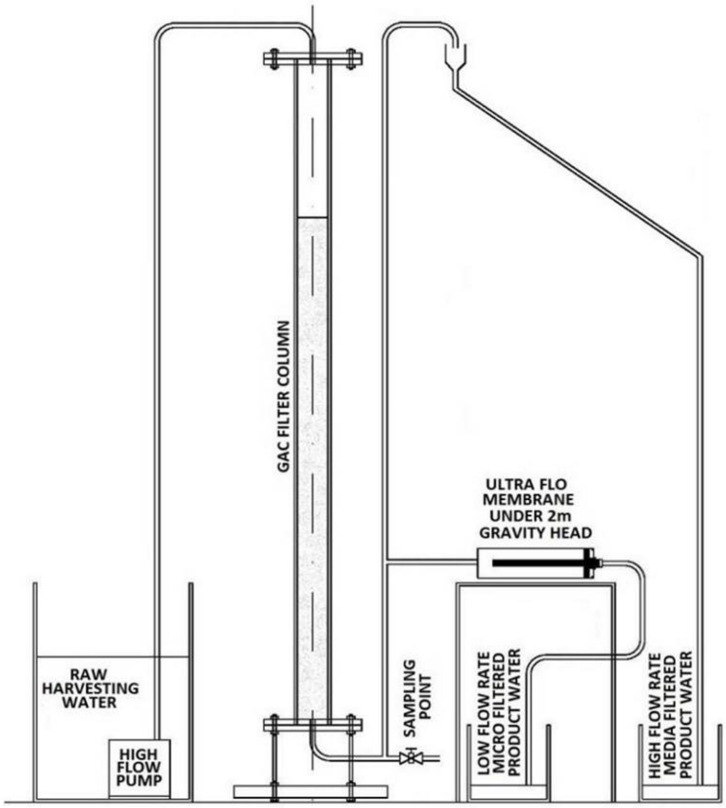


Figure 3.27 Schematic diagram of high flow GAC media filtration and Ultra Flo hollow fibre micro-filtration membrane under gravitational head

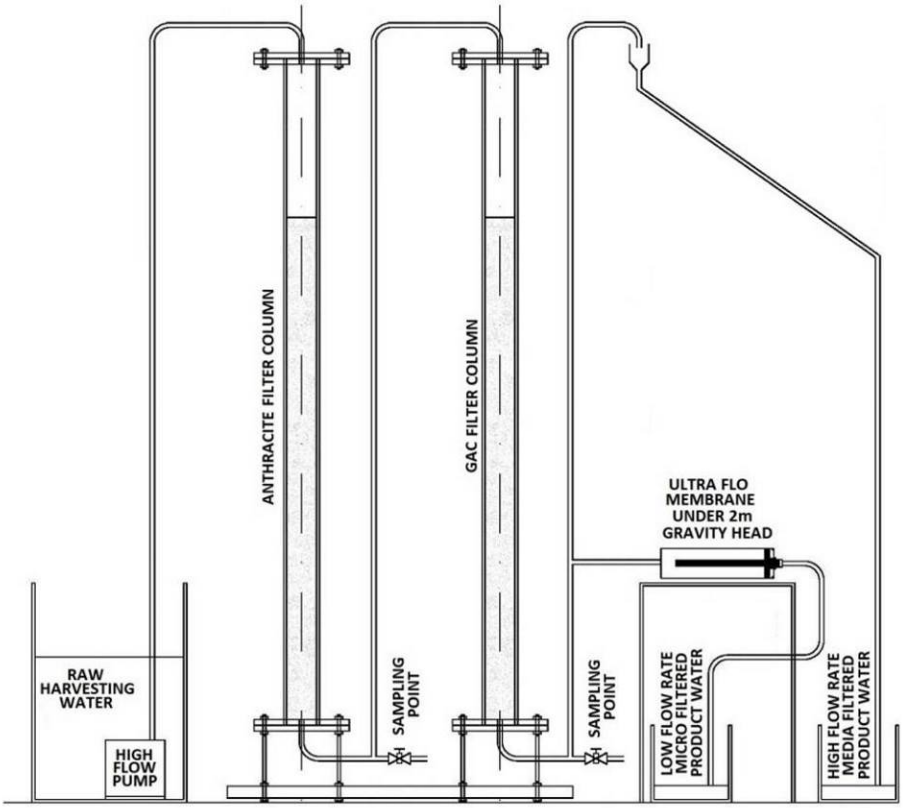


Figure 3.28 Schematic diagram of high flow Anthracite and GAC media filtration and Ultra Flo hollow fibre micro-filtration membrane under gravitational head

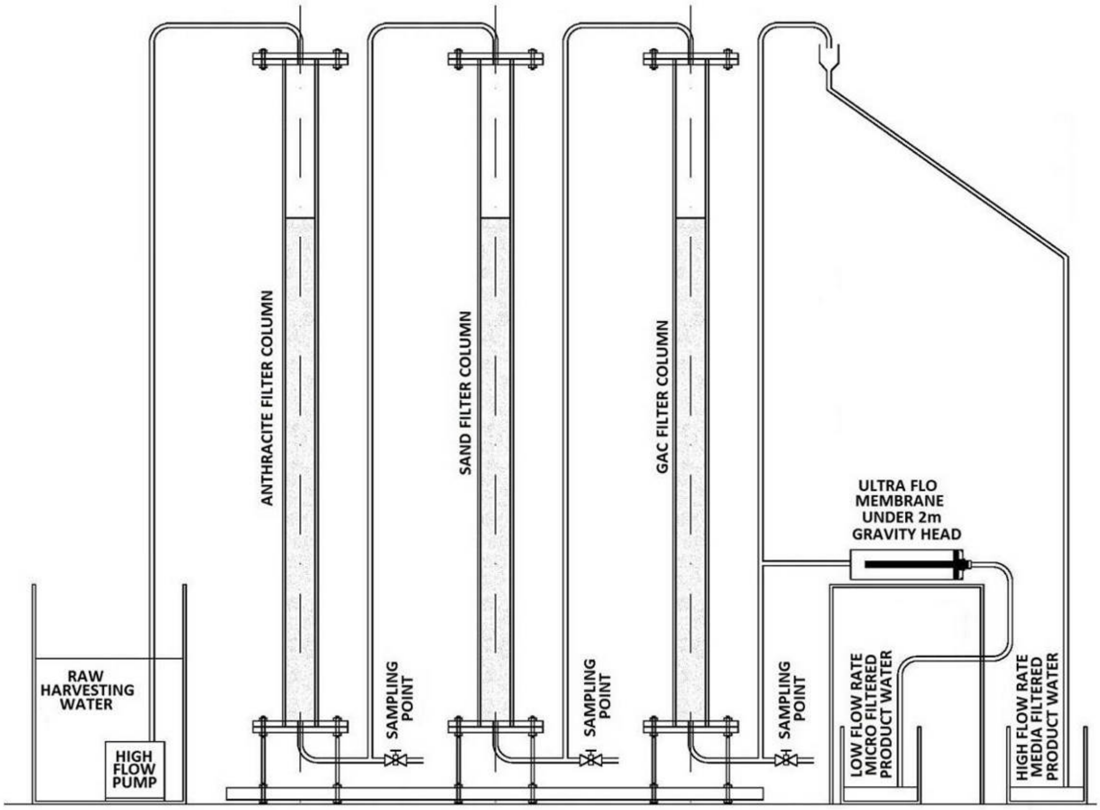


Figure 3.29 Schematic diagram of high flow Anthracite, Sand and GAC media

3.4. Pilot scale rainwater treatment system

A rainwater treatment system consisting of a gravity fed membrane filter (Ultra Flo) and a media (GAC) filter was operated for a period of 120 days and monitored for water quality and flux decline (see Chapter 9). One residential rainwater tank (T17, Table 3.1) located in the Sydney metropolitan basin at Peakhurst was used for the experiment involving the rainwater treatment system. It was located approximately 1 km from a heavily trafficked motorway and 10 km away from Sydney's major domestic and international airport (Figure 3.1). The tank specifications are given in Table 3.1 (listed under T17).

The tank and house were 5 years old with a typical concrete glazed tile roof with aluminium guttering. The tank was a typical PVC tank with PVC plumbing and brass tap fittings and a total volume of 3000 litres. For the duration of this experiment, the tank's normal operation was stopped and the pump was removed to provide an adequate quantity of feed water for the experiments. Potable grade hose lines were installed between the tank and the cartridge filter system.

Filter configuration

The experiment consisted of raw rainwater fed directly from the rainwater tank and passed through the two consecutive filters consisting of pre-treatment with GAC filter (Table 3.5) followed by MF membrane filtration (Ultra Flo). The characteristics of GAC are shown in Table 3.13, while those of the membrane are given in Table 3.14. The allowable pressure according to the manufacturer's specifications was 54 kPa for feed and 95 kPa for back flush. The apparatus set-up is shown in Figure 3.30.

As the system was operated under gravity head, the two filter cartridges were placed horizontally and located at the base of the rainwater tank to take advantage of the full water head available in the rainwater tank. The effluent from the GAC filter cartridge passed through to the membrane filter cartridge. The gravity head in the rainwater tank drives the flow through the media filter and membrane filter. The available water head varied up to 2 m of gravity head. The treatment system was run continuously for 120 days. Over time the flux through the two filters decreased as they became clogged. The GAC and membrane filter cartridges were backwashed for 30 seconds on days 8 and 12. Backwashing was not carried out after day 12 to see how the system will operate without periodical cleaning. During the backwashing process, the flow was passed through the two filters in the reverse direction from normal operation. The influent flow pipe was rerouted from the rainwater tank to a waste drain. Excess free particles were removed from the column filter along with the tap water.

Detailed laboratory analyses were carried out to determine the concentration of individual pollutants as per the sampling methods listed in Section 3.2.2. The primary parameters measured were total organic carbon (TOC) and turbidity.

Data logging equipment (Table 3.2) was utilised to monitor the flow rates (using flow meter, Table 3.12). The flow rate was only controlled by the flux limitation through the treatment system and the available driving head. The available head ranged from the membrane to the top of the water level in the rainwater tank and was up to 2 m. The tank's water level only increased when rainfall occurred.

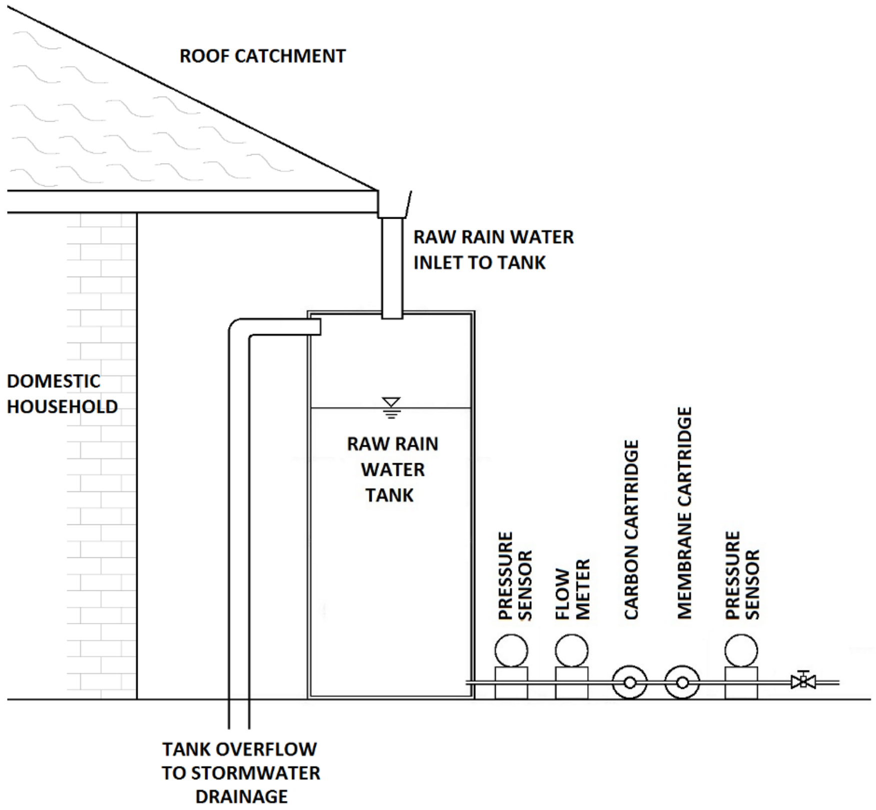


Figure 3.30 Schematic diagram of Watts (Ultra Flo) hollow fibre micro-filtration membrane cartridge system under gravitational head with GAC adsorption pre-treatment

Chapter 4

University of Technology, Sydney

Faculty of Engineering &

Information Technology

4. Characterisation of Rainwater

4.1. Characterisation of metropolitan rainwater tanks

Detailed sampling was carried out on eleven rainwater tanks located in different parts of metropolitan Sydney in New South Wales and one in Wollongong, located south of Sydney, as described in Section 3.3.1. The concentration of pollutants from the samples collected from the metropolitan rainwater tanks (T1 to T11) are described in this chapter and summarised in Table 4.1.

4.1.1. Heavy metals

The water from the majority of rainwater tanks complies with most of the heavy metals tested with the exception of iron and lead. Their average concentration of tanks T1 and T5 were under the ADWG (2011), limit for iron, which was 0.3 mg/L. However each tank contained at least one sample over this limit with individual results of 4.70 mg/L and 4.18 mg/L, respectively. The lead concentration was a concern with individual samples and average samples from most tanks exceeding the ADWG (2011) lead limit of 0.01 mg/L. T1 contained an average of 0.016 mg/L with an upper limit of 0.033 mg/L, T4 contained an average or 0.010 mg/L with an upper limit of 0.029 mg/L, T5 contained an average of 0.049 mg/L with an upper limit of 0.067 mg/L, T8 contained an average of 0.007 mg/L with an upper limit of 0.017 mg/L, and finally, T10 contained an average of 0.013 mg/L with an upper limit of 0.021 mg/L. All other heavy metals were well within the ADWG (2011) standard. The concentration levels of arsenic, cadmium, chromium, mercury, nickel selenium and silver all showed negligible concentrations of less than 0.001 mg/L.

If the sludge that collects in the base of the rainwater tanks were to be disturbed, the concentration could be expected to exceed the ADWG (2011) limit for some of the heavy metals, due to the volume of accumulation between periods of rainwater tank maintenance (Magyar et al., 2007).

4.1.2. Anions and cations, total dissolved salts

Mineral salts are a part of most of people's daily dietary intake. The ADWG (2011) does not provide the maximum concentration limits for these salts in drinking water. Analysis carried out on potable water (supplied by Sydney Water) showed that the rainwater in tanks was generally equivalent to or had lower concentrations of sodium, calcium, magnesium, chloride and sulphate. The only mineral salt that was higher in concentration than the potable water supply (Sydney Water) was potassium.

Medical dietary multi-vitamin and mineral supplements are commonly available. The dose of one tablet contains 100 mg of potassium, 100 mg of calcium, 145 mg of magnesium and 36.3 mg of chloride. To consume these quantities of mineral salts in the typical rainwater tanks sampled in this study (with the exception of T3 and T8), more than 30 litres of water is required to consume the equivalent potassium dosage, more than 8 litres for calcium, more than 50 litres for magnesium and more than 7 litres for chloride. An average person drinks approximately 2-3 L of water per day.

4.1.3. Ammonia, nitrate, nitrite and orthophosphate

With regard to nitrate, nitrite and ammonia, all rainwater tanks complied with ADWG (2011). Orthophosphate is used around the world as a corrosion inhibitor in some potable water supplies, especially where it has been observed that high concentrations of lead or copper exist sourced from potable water pipelines. Concentrations of orthophosphate are dosed at up to 1mg/L (Edwards et al., 2002; Li et al., 2004) to reduce the metal corrosion in the water distribution pipes. All rainwater tanks contained concentrations of orthophosphate of less than 1 mg/L.

4.1.4. pH and water hardness

The pH analysis demonstrated that the rainwater tanks are generally within or close to compliance with the ADWG (2011) guidelines of between pH 6.5 and 8.5 with the exception of T6 which on average was around pH 5.7. The water hardness of T5 and T6 was rather low at an average value of 0.92 mg/L and 1.26 mg/L of $CaCO_3$, respectively, which indicates that there is no water buffer. With the addition of any acidic elements to this system such as animal or humic acids from leaves, the pH of this rainwater tank would be expected to drop rapidly. This seems true for T5 which is actually approaching the minimum limit with a pH 6.52. Rainwater tanks which contain high buffers or water hardness levels (with the exception of T5) drain concrete tiled roofs. The rainwater tanks containing low buffer or water hardness levels drain Colourbond or Zincalume metal roofing.

4.1.5. Turbidity and Suspended Solids

The ADWG (2011) has a recommended limit for turbidity of 5 NTU. Most tanks complied with this limit. Rainwater tanks T1, T5 and T8 on average complied with this limit although at times this limit was exceeded with the highest individual readings of 12 NTU, 8 NTU and 6 NTU, respectively. This was due to a dirty roof on the house of T1 and T5 and the rainfall collected in tank T8 had stirred up sediments within the tank.

The ADWG (2011) does not state a limit for total suspended solids (TSS). As this is somewhat similar to turbidity it could be assumed that if turbidity complies with the recommended limits then the total suspended solids should also be satisfactory. TSS mostly ranged from less than 0.5 mg/l to 3.5 mg/L in most of the metropolitan tanks (except for T1, T5 and T8) when they complied with turbidity of less than 5 NTU. T1, T5 and T8 contained concentrations of 5.5 mg/L and above when their turbidity levels exceeded 5 NTU. Overall, the water collected in the rainwater tanks generally complied with the ADWG (2011) limits for most parameters except for a few individual parameters from individual rainwater tanks. These are shown in bold in Table 4.1. The majority of parameters tested were comparable to potable water.

Table 4.1 Laboratory analysis of rainwater tank samples

Parameter	T1* Ingleburn	T2* Wollongong	T3 * Kogarah	T4 * Mnt Hunter	T5* Kirkham	T6* Narellan	T7* Kemps Crk	T8* Cawdoor	T9* Therisa Park	T10* Glen Alpine	T11* Newtown	Urban, Range and Average*
pH	6.89-7.30 7.13	6.71-7.49 7.12	7.13-7.48 7.28	**6.39**-8.19 7.27	**5.79**-7.09 6.52	**5.41-5.83** **5.70**	7.19-7.38 7.26	6.64-7.45 6.96	6.60-**8.62** 7.54	6.58-7.54 7.10	**6.48**-6.90 6.74	**5.41-8.62** 6.97
Conductivity (EC) (dS/m)	0.06-0.10 0.08	0.04-0.05 0.05	0.12-0.15 0.13	0.06-0.07 0.07	0.01-0.01 0.01	0.02-0.02 0.02	0.07-0.08 0.08	0.12-0.16 0.14	0.04-0.05 0.04	0.07-0.10 0.09	0.04-0.06 0.05	0.01-0.16 0.07
Total dissolved salts (mg/L)	39.44-70.72 55.3	29.92-32.64 31.05	80.24-98.60 86.36	42.84-47.60 45.33	7.48-8.84 8.39	13.60-14.96 14.45	50.32-53.04 51.91	79.56-107.44 91.80	27.20-35.36 30.15	45.56-67.32 59.16	23.80-40.12 34.68	7.48-107.44 46.23
Total suspended solids (mg/L)	0.50-17.00 6.17	1.00-2.00 1.67	1.00-2.00 1.33	0.50-2.50 1.17	1.00-12.50 4.83	1.00-1.50 1.13	1.00-3.50 1.83	2.00-5.50 3.33	1.00-2.50 1.50	0.50-3.50 1.83	0.50-1.00 0.83	0.50-17.00 2.33
Turbidity (NTU)	0.20-**12.00** **5.07**	0.20-2.00 0.87	0.20-0.60 0.33	0.20-4.00 2.07	0.80-**8.00** 3.60	0.40-2.00 1.05	0.60-2.00 1.53	0.20-**6.00** 2.73	0.20-2.00 1.07	1.00-2.00 1.67	0.60-2.00 1.13	0.20-**12.00** 1.92
Water hardness (mg/L$CaCO_3$ equivalent)	17.32-33.72 27.65	6.91-8.37 7.53	26.34-37.31 30.22	17.15-24.37 21.89	0.59-1.32 0.92	0.58-2.05 1.26	26.85-30.54 28.63	32.88-46.78 39.96	8.53-17.33 13.23	21.48-37.79 29.17	5.13-6.53 5.98	0.58-46.78 18.77
Orthophosphate (mg/L P)	0.00-0.03 0.02	0.01-0.01 0.01	0.00-0.00 0.00	0.01-0.36 0.24	0.00-0.00 0.00	0.00-0.01 0.00	0.00-0.01 0.00	0.03-0.03 0.03	0.01-0.02 0.02	0.01-0.02 0.02	0.01-0.01 0.01	0.00-0.36 0.03
Nitrate (mg/L N)	0.10-0.65 0.33	0.08-0.38 0.19	0.20-0.26 0.24	0.05-0.80 0.45	0.18-0.41 0.32	0.43-0.74 0.59	0.78-0.91 0.86	0.52-0.73 0.63	0.38-0.60 0.52	0.06-0.68 0.39	0.19-0.62 0.34	0.05-0.91 0.44
Nitrite (mg/L N)	0.00-0.01 0.00	0.00-0.01 0.00	0.01-0.02 0.02	0.00-0.01 0.00	0.00-0.00 0.00	0.00-0.00 0.00	0.00-0.00 0.00	0.00-0.01 0.00	0.00-0.01 0.00	0.01-0.01 0.01	0.01-0.01 0.01	0.00-0.02 0.00
Ammonia (mg/L N)	0.020-0.119 0.056	0.005-0.012 0.08	0.073-0.186 0.112	0.003-0.023 0.013	0.005-0.115 0.053	0.129-0.250 0.195	0.009-0.036 0.023	0.020-0.069 0.038	0.005-0.061 0.024	0.005-0.072 0.035	0.037-0.208 0.147	0.003-0.250 0.064
Sodium (mg/L)	1.75-4.02 2.62	1.89-3.24 2.76	6.49-15.09 9.42	1.11-2.66 1.88	0.61-1.03 0.82	0.91-1.51 1.25	1.35-1.92 1.65	6.55-8.72 7.64	0.97-1.21 1.11	1.41-4.26 2.54	3.10-8.17 6.45	0.61-15.09 3.47
Potassium (mg/L)	0.98-1.82 1.29	1.83-2.85 2.18	0.94-1.57 1.17	0.48-1.41 0.80	0.16-0.32 0.25	0.01-0.25 0.13	0.10-0.44 0.27	0.85-1.38 1.14	0.46-1.01 0.78	0.03-1.15 0.57	0.13-0.34 0.21	0.01-2.85 0.80
Calcium (mg/L)	6.44-12.40 10.30	2.19-2.84 2.43	6.96-8.15 7.67	6.06-9.28 8.16	0.17-0.38 0.25	0.14-0.44 0.28	10.06-11.67 10.81	10.94-15.88 13.64	3.20-6.71 5.06	8.01-14.31 10.79	1.03-1.23 1.11	0.14-15.88 6.41

Table 4.1 (contd.) Laboratory analysis of rainwater tank samples

Parameter	T1* Ingleburn	T2* Wollongong	T3 * Kogarah	T4 * Mnt Hunter	T5* Kirkham	T6* Narellan	T7* Kemps Crk	T8* Cawdoor	T9* Therisa Park	T10* Glen Alpine	T11* Newtown	Urban, Range and Average*
Magnesium (mg/L)	0.30-0.67 0.47	0.31-0.41 0.36	1.60-4.84 2.69	0.29-0.49 0.37	0.04-0.09 0.07	0.05-0.23 0.14	0.34-0.43 0.40	1.22-1.73 1.43	0.13-0.16 0.14	0.36-0.77 0.54	0.50-0.93 0.78	0.04-4.84 0.67
Chloride (mg/L)	1.91-7.66 4.63	3.94-5.67 4.84	11.59-22.81 15.35	5.37-7.34 6.07	3.86-4.98 4.50	4.45-5.11 4.70	4.56-6.08 5.09	16.03-22.96 19.39	4.26-4.90 4.62	4.53-12.93 7.44	11.19-12.94 12.28	1.91-22.96 8.08
Sulphate (mg/L SO_4^{2-})	1.23-1.59 1.42	1.53-3.12 2.10	4.08-9.81 6.02	1.92-2.37 2.12	0.81-1.56 1.21	1.98-2.13 2.03	2.73-3.12 2.92	3.30-4.38 4.00	1.41-1.80 1.58	2.22-2.85 2.54	2.55-5.37 4.33	0.81-9.81 2.75
Aluminium (mg/L)	0.06-0.09 0.08	0.00-0.01 0.01	0.01-0.03 0.02	0.01-0.03 0.02	0.02-0.03 0.03	0.02-0.03 0.02	0.02-0.03 0.02	0.02-0.07 0.04	0.08-0.16 0.11	0.01-0.07 0.03	0.03-0.04 0.03	0.00-0.16 0.04
Copper (mg/L)	0.01-0.06 0.03	0.00-0.00 0.00	0.11-0.64 0.41	0.00-0.10 0.06	0.00-0.02 0.01	0.05-0.13 0.07	0.01-0.34 0.12	0.03-**2.37** 1.03	0.00-0.01 0.00	0.02-0.07 0.05	0.00-0.01 0.01	0.00-**2.37** 0.16
Iron (mg/L)	0.05-**4.70** **1.69**	0.01-0.02 0.02	0.02-0.02 0.02	0.01-0.03 0.02	0.07-**4.18** **1.51**	0.00-0.08 0.03	0.01-0.06 0.04	0.03-0.11 0.07	0.00-0.01 0.00	0.01-0.02 0.01	0.01-0.02 0.01	0.00-**4.70** **0.31**
Manganese (mg/L)	0.00-0.06 0.02	0.00-0.00 0.00	0.00-0.00 0.00	0.00-0.00 0.00	0.01-0.03 0.02	0.01-0.01 0.01	0.00-0.01 0.00	0.00-0.02 0.01	0.00-0.00 0.00	0.00-0.00 0.00	0.00-0.01 0.00	0.00-0.06 0.01
Lead (mg/L)	0.006-**0.033** **0.016**	0.000-0.01 0.00	0.001-0.006 0.003	0.001-**0.029** **0.010**	**0.038-0.067** **0.049**	0.004-0.007 0.006	0.000-0.001 0.001	0.001-**0.017** 0.007	0.001-0.001 0.001	0.008-**0.021** **0.013**	0.001-0.008 0.003	0.000-**0.067** **0.010**
Zinc (mg/L)	0.04-0.31 0.13	0.01-0.02 0.03	0.03-0.56 0.26	0.07-1.41 0.56	0.04-0.05 0.04	0.18-1.16 0.45	0.01-0.04 0.03	0.09-0.29 0.17	0.01-0.03 0.02	0.09-0.63 0.28	0.38-0.77 0.52	0.01-1.41 0.23

*1st row-range of value, 2nd row – mean average value, Values exceeding the ADWG (2011) limit are shown in bold,

4.1.6. Molecular weight distribution (MWD) of rainwater organic matter (RWOM)

Detailed molecular weight distribution (MWD) of rainwater organic matter (RWOM) was conducted to identify the components of organic contamination. The MWD of RWOM was monitored to determine the effects of: i) contamination of rainwater by contact with the roof, ii) the ageing of rainwater in the tank for the duration between storm events, and iii) the ageing of rainwater in the tank over a long period during which rainfall occurred.

MWD of RWOM before/after roof contact of rainwater

Figure 4.1 shows the MWD of RWOM in rainwater before it came into contact with the roof (rainwater itself), rainwater after roof contact, commercially available bottled water and tap water supplied by Sydney Water. Rainwater itself included the MWD of RWOM ranging from 850 Da to 220 Da. The origin of RWOM may be due to contact with air pollutants dissolved in the rainwater. However, when the rainwater came into contact with the roof, the MWD of RWOM indicated a different trend compared to rainwater itself. The MWD of RWOM after contact with the roof consisted of 37500 Da, 850 Da, 500 Da and 220 Da. A new MW of 37500 Da appeared and the MW of 850 Da showed the highest peak intensity. Overall, the intensity of UV responses significantly increased. According to one study (Shon et al., 2006), the MW of 37500 Da may be due to biopolymers, 850 Da to humic substances, 500 Da to building blocks, 220 Da to low MW acids, and less than 220 Da to amphiphilics. This suggests that after contact with the roof, the rainwater was significantly contaminated especially by biopolymers and humic substances. However there is some uncertainty concerning the origins of the RWOM.

The MWD of RWOM with rainwater before/after contact with the roof was compared with commercially available bottled water and Sydney tap water (Figure 4.1). The MWD of organic matter from bottled and tap water showed low UV intensity compared to rainwater. Tap water mostly included humic substances and low MW acids, while bottled water only consisted of low MW acids. The results show that concentration of the RWOM before/after is higher than that of tap and bottled water.

MWD of RWOM in terms of the ageing effects for durations between storm events

Figure 4.2 illustrates the MWD of RWOM which shows the effect of ageing of rainwater for the duration between storm events in a residential rainwater tank. From day 1 to day 17, the intensity of the MW of 37500 Da and 850 Da increased with time, showing that biopolymers (37500 Da) and humic substances (850 Da) increased during the storage period in the tank (Figure 4.2). The increase of the former could be the effect of microbial communities increasing the concentration of biopolymers and humics.

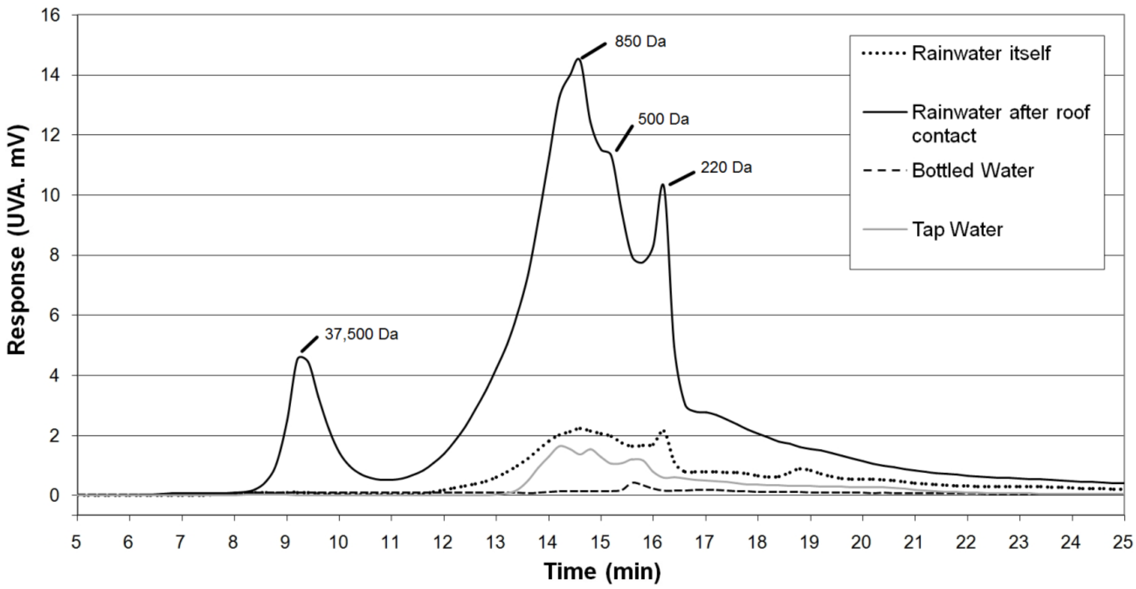


Figure 4.1 MWD of RWOM before/after roof contact of rainwater

MWD of RWOM in terms of the ageing effects for long durations during which rainfall occurred

Figure 4.3 shows the MWD of RWOM as the rainwater in the tanks ages during the normal operation of a residential rainwater tank over a 180-day period. During this period the rainwater tank was used for general purposes and allowed to fill, principally during periods of rainfall, and empty, as rainwater water was consumed. Here, it should be noted that the rainwater samples in the rainwater tank were collected at set times and during the intervening periods rain may have fallen and filled the tank.

Figure 4.3 shows that from day 90 to day 180, a generalised trend of the MWD was not found. MWD for the major peaks of 850 Da, 500 Da and 220 Da showed a decline over the period up to 180 days. Interestingly, the intensity of the MW of 37500 Da significantly increased after 120 days in an inconsistent manner during the period of sampling. This could be due to: i) uncontrollable seasonable changes which include surrounding trees not having leaves to shed on the roof in winter, ii) the dilution effect of the frequent variation in rainfall, and iii) the effect of microbial communities increasing the concentration of biopolymers. Further detailed characterisation of RWOM is needed to investigate this issue.

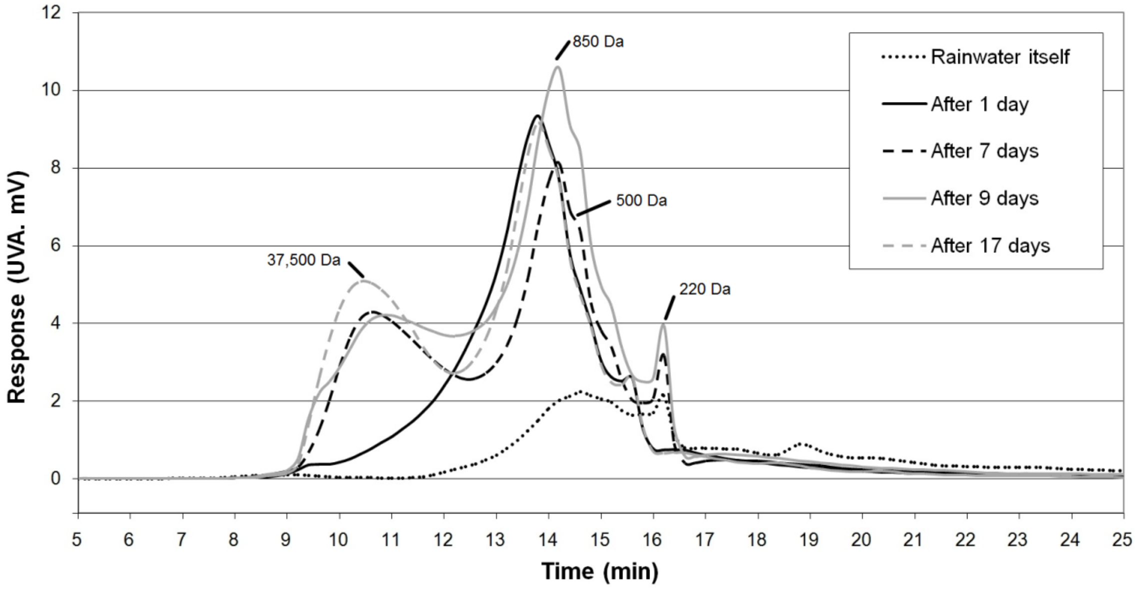


Figure 4.2 MWD of RWOM showing the effects of ageing of rainwater in tank over 1-17 days

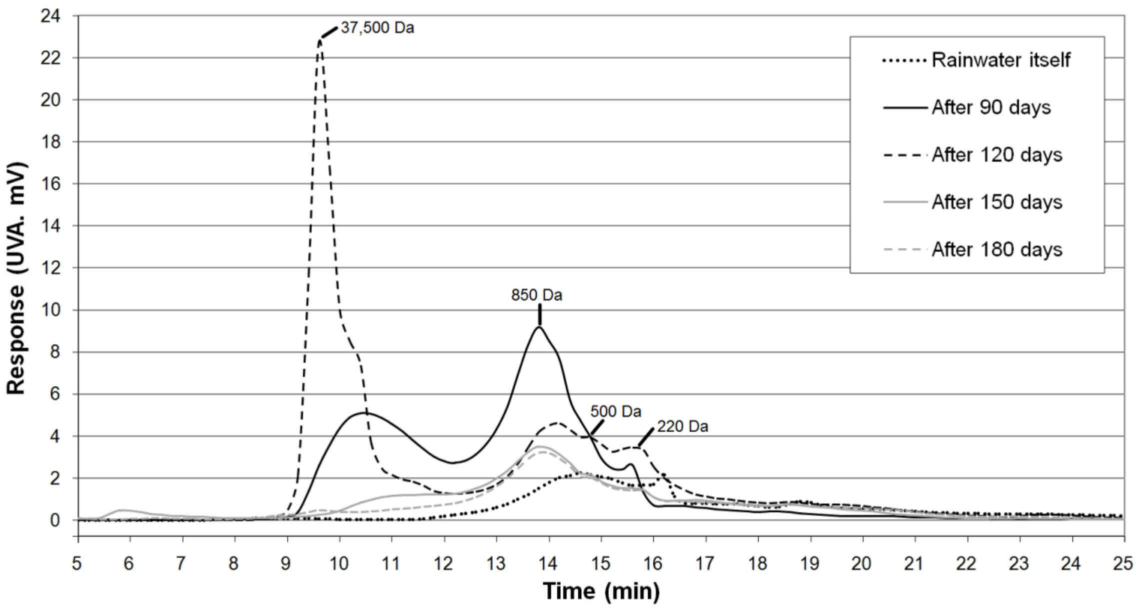


Figure 4.3 MWD of RWOM showing the ageing effects for long durations during which rainfall occurred

4.2. Characterisation of outer Sydney rural raintank water

Detailed sampling was carried out on rural rainwater tanks (T12 to T16) located in the Kangaroo Valley, approximately 160 km south-west of Sydney as described in Section 3.3.1. The concentrations of pollutants in the samples collected from rural rainwater tanks (T12 to T16) are described below and summarised in Table 4.2.

4.2.1. Anions and cations and total dissolved salts

ADWG (2011) does not provide recommended limits for these parameters. A comparison with the metropolitan potable water supply (Sydney Water Corporation) shows that rural rainwater tanks generally had low concentrations of sodium, calcium, magnesium, chloride and sulphate. The only parameter that was marginally higher in concentration than the potable water supply was potassium in the rural rainwater tanks. Total dissolved salts (TDS) is the combined measurement of all anions and cations in the water. All of the rural rainwater tanks had considerably lower levels of TDS at one fifth or less.

4.2.2. pH and water hardness

Water hardness is another parameter that follows TDS closely as they are to some extent related parameters. Water hardness acts as a buffer to the addition of any acidic elements to this system such as animal or humic acids from leaves to prevent the pH from resulting in an acidic range. All of the rural rainwater tanks had considerably lower water hardness at one third or less than the metropolitan potable water supply. The rural rainwater tanks all drained from galvanised Colourbond roofs and rain tanks were constructed from PVC with the exception of T15, which was a concrete rainwater tank. This resulted in the highest level of water hardness for all rural rainwater tanks.

With all rural rainwater tanks having considerably low water hardness, it is not surprising that all rainwater tanks except T15, do not comply with pH measurements being below pH 6.5, thus resulting in acidic rainwater conditions.

4.2.3. Ammonia, nitrate, nitrite and orthophosphate

All rural rainwater tanks complied with the ADWG (2011) limits for ammonia, nitrate, nitrite and orthophosphate.

4.2.4. Turbidity and total suspended solids

The ADWG (2011) stated that 5 NTU is the recommended limit of turbidity. All rural rainwater tanks complied well below the 5 NTU limit. In general, sampling shows that the bigger the rainwater tank volume, the lower the turbidity was. This was true with T1 (Tables 3.1 and 3.2 in Section 3.2.1) being the smallest rainwater tank, which also contained the highest turbidity levels. With the rural rainwater tanks being larger than most metropolitan rainwater tanks, this again is true for the rural rainwater tanks resulting in low turbidity levels with an overall average of 0.71 NTU across the 5 rainwater tanks.

Larger tanks afford longer settling times for the rainwater contained within them resulting in better turbidity levels. Furthermore a larger rainwater tank captures more water from a rainfall event, which in turn produces cleaner runoff the longer that rain falls on the roof. The small rainwater tanks would collect relatively more of the first portion of the rainfall event which typically contains more polluted wash-off from the roof.

The ADWG (2011) does not state a limit for total suspended solids. TSS mostly ranged from less than 0.5 mg/l to 3.5 mg/L in most of the metropolitan tanks (except for T1, T5 and T8) when they complied with the turbidity limit of less than 5 NTU. All samples from the rural rainwater tanks were at 1.0 mg/l or less.

4.2.5. Total organic carbon

The ADWG (2011) does not recommend a limit for total organic carbon. The influent rainwater samples for the rural rainwater tanks contained an average value of 0.393 mg/L within a range of 0.20 mg/L to 0.58 mg/L. By comparison metropolitan rainwater tanks contained an average value of 9.44 mg/L ranging from 2.17 mg/L to 13.26 mg/L.

4.2.6. Heavy metals

The sampling shows that rural rainwater tanks did comply with the heavy metals tested. The lead concentration was a concern in that most of the metropolitan tanks sampled with average concentrations exceeded the ADWG (2011) lead limit of 0.01 mg/L. The rural rainwater tanks, however, all complied with ADWG (2011); all samples of lead concentrations were below the detectable limit of 0.001 mg/L. All other heavy metals were well within the guideline's recommended limits. The concentration levels of arsenic, cadmium, chromium, mercury, nickel selenium and silver all indicated negligible concentrations of less than 0.001mg/L.

4.2.7. Total Coliform and Faecal Coliform

The ADWG (2011) recommends a limit of <1 CFU/100 mL for faecal and total coliform counts. All rural rainwater samples had counts exceeding the recommended limits for faecal and total coliform counts. Overall, the water collected in both the rural rainwater tanks generally complied with the ADWG (2011) for most parameters except for a few such as the pH and total and faecal coli. These are shown in bold in Table 4.2. To comply with the ADWG (2011), minimal water treatment is required to produce potable water.

Table 4.2 Laboratory analysis of rural rainwater tank samples

	ADWG (2011)	T12	T13	T14	T15	T16	Rural Average
pH*	6.5-8.5	**6.07 - 6.11** **6.09**	**5.69 - 5.81** **5.75**	**6.02 - 6.03** **6.03**	6.93 - 7.23 7.08	**5.95 - 6.05** **6.00**	**5.69** - 7.23 **6.19**
Total dissolved salts (mg/L)*	-	12 - 15 13.5	12 12	12 - 13 12.5	24 24	20 - 21 20.5	12 - 24 16.5
Total suspended solids (mg/L)*	-	<0.5 - 1 0.75	<0.5 - <0.5 <0.5	<0.5 - <0.5 <0.5	<0.5 - <0.5 <0.5	<0.5 - <0.5 <0.5	<0.5 - 1 <0.5
Turbidity (NTU)*	<5	0.7 - 1.0 0.85	0.6 - 0.8 0.7	0.6 - 0.8 0.7	0.9 - 1.0 0.95	0.3 - 0.4 0.35	0.3 - 1.0 0.71
Water hardness (mg/L $CaCO_3$ equivalent)**	<200	2 - 2 2	2 - 3 2.5	3 - 3 3	12 - 12 12	4 - 5 4.5	2 - 12 4.8
Orthophosphate (mg/L P)	-	0.014-0.019 0.017	0.023-0.024 0.024	0.010-0.012 0.011	0.009-0.010 0.010	0.009-0.009 0.009	0.009-0.024 0.014
Nitrate (mg/L N)	<50	0.046-0.068 0.057	0.248-0.254 0.251	0.198-0.208 0.203	0.128-0.130 0.129	0.386-0.400 0.393	0.046-0.400 0.207
Nitrite (mg/L N)	<3	<0.005 - <0.005 <0.005	<0.005 - <0.005 <0.005	<0.005 - <0.005 <0.005	<0.005 - <0.005 <0.005	<0.005 - <0.005 <0.005	<0.005 - <0.005 <0.005
Ammonia (mg/L)	<0.5	0.006-0.006 0.006	0.005-0.006 0.006	0.005-0.007 0.006	0.007-0.009 0.008	0.004-0.007 0.006	0.004-0.009 0.006
Sodium (mg/L)	-	1.92-1.99 1.96	1.98-4.86 3.42	2.15-2.29 2.22	2.17-2.20 2.19	3.78-6.17 4.98	1.92-6.17 2.951
Potassium (mg/L)	-	0.76-2.47 1.615	0.26-0.42 0.34	0.34-0.34 0.34	0.89-1.02 0.96	0.34-0.45 0.40	0.26-2.47 0.729

	ADWG (2011)	T12	T13	T14	T15	T16	Rural Average
Calcium (mg/L)	-	0.25-0.29 0.27	0.37-0.64 0.51	0.38-0.40 0.39	4.29-4.43 4.36	0.72-0.99 0.86	0.25-4.43 1.276
Magnesium (mg/L)	-	0.31-0.31 0.31	0.31-0.33 0.32	0.39-0.40 0.40	0.28-0.29 0.29	0.59-0.63 0.61	0.28-0.63 0.384
Chloride (mg/L)	<400	3.4-5.3 4.35	2.6-2.9 2.75	2.9-3.1 3.0	2.9-3.0 2.95	5.5-5.6 5.55	2.6-5.6 3.72
Sulphate (mg/L SO42-)	<400	4-6 5	4-7 5.5	3-4 3.5	5-5 5	5-10 7.5	3-10 5.3
Total coliforms (cfu/100 ml)*	<1	**20 - 40 30**	**40 - 70 55**	**30 - 40 35**	**<10 - 10 10**	**<10 - 10 10**	**<10 - 70 28**
Faecal coliforms (cfu/100 ml)*	<1	<10 - <10 <10	**10 - 10 10**	<10 - <10 <10	<10 - <10 <10	<10 - <10 <10	<10 -10 <10
Total organic carbon (mg/L)*	-	0.20 - 0.27 0.235	0.33 - 0.37 0.35	0.49 - 0.58 0.535	0.47 - 0.50 0.485	0.36 - 0.36 0.36	0.20 - 0.58 0.393
Aluminium (mg/L)	<0.2	0.016-0.017 0.0165	0.010-0.014 0.012	0.013-0.023 0.018	0.022-0.023 0.0225	0.009-0.010 0.0095	0.009-0.023 0.016
Copper (mg/L)	<2	0.004-0.004 0.004	0.013-0.022 0.0175	0.011-0.028 0.0195	0.024-0.032 0.028	0.010-0.013 0.0115	0.004-0.032 0.018
Iron (mg/L)*	<0.3	<0.01 - <0.01 <0.01	<0.01 - <0.01 <0.01	<0.01 - <0.01 <0.01	<0.01 - <0.01 <0.01	<0.01 - <0.01 <0.01	<0.01 - <0.01 <0.01
Manganese (mg/L)	<0.1	0.002-0.003 0.0025	0.004-0.008 0.006	0.003-0.005 0.004	0.001-0.001 0.001	0.016-0.022 0.019	0.001-0.022 0.0115
Lead (mg/L)*	<0.01	<0.001 - <0.001 <0.001	<0.001 - <0.001 <0.001	<0.001 - <0.001 <0.001	<0.001 - <0.001 <0.001	<0.001 - <0.001 <0.001	<0.001 - <0.001 <0.001
Zinc (mg/L)	<3	0.514-0.586 0.550	0.514-0.586 0.550	0.539-0.586 0.5625	0.085-0.122 0.1035	0.125-0.415 0.270	0.009-0.586 0.2975

*1st row-range of value, 2nd row – mean average value, Values exceeding the ADWG (2011) limit are shown in bold,
** $CaCO_3$ equivalent

4.3. Characterisation of the first flush in Sydney metropolitan rainwater tanks

In this study first flush samples of roof runoff from an urban residential roof (T1) located in the Sydney's metropolitan area were collected and analysed. The analysis of the first flush was conducted to determine whether a first flush rainfall runoff exists from the roof of a residential house and whether bypassing it from the tank could help improve rainwater tank quality. The methodology is described in Section 3.3.1.

The results of pollutant concentration over the depth of rainfall are given in Figure 4.4 (a-h) and Tables 4.3 to 4.5 for all 3 rainfall events that were monitored. Also given are their applicable concentration limits provided by the ADWG (2011) and their corresponding concentration in potable tap water and rainwater. The measurements for concentration of pollutants in rainfall (before coming into contact with the roof) are also provided.

4.3.1. Turbidity and total suspended solids

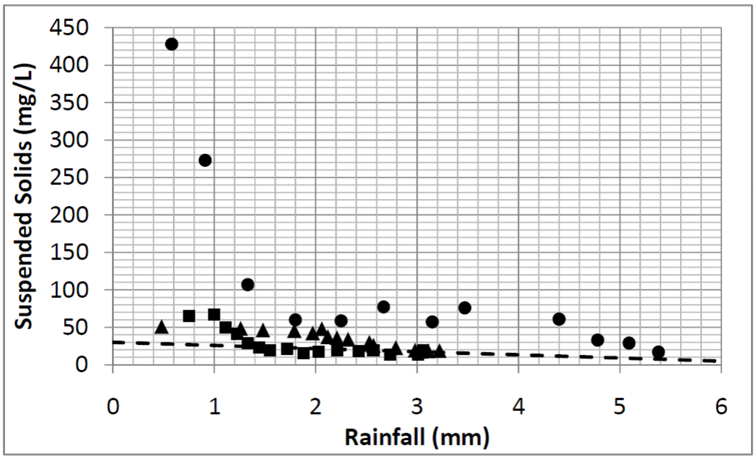
Figure 4.4 (a) shows that bypassing the first 0.5 mm to 1 mm of roof runoff will improve the rain tank water by reducing TSS during a filtration process.

Figure 4.4 (b) demonstrates that for the turbidity to be equal to the 5 NTU as specified in ADWG (2011), the first approximately 5 mm of rainfall should be bypassed. The concentration of turbidity in rain (before it came into contact with the roof) was more than the ADWG (2011) limit for the first 1 mm of rainfall. Although turbidity is not necessarily a health hazard, it may constitute a health risk if the suspended particles harbour microorganisms able to cause disease in humans, or if the particles have adsorbed toxic organic or inorganic compounds. Bypassing the first flush substantially improves the visual aesthetics of the water, which is also an important step in achieving public acceptance based on viewing a glass of the tank water. 5 NTU would appear slightly milky-looking in a drinking glass (ADWG, 2011).

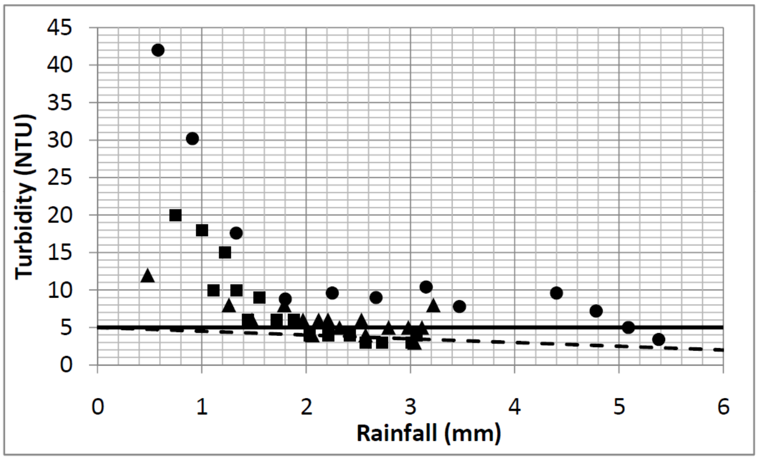
According to previous studies conducted on rainwater tanks in Australia, some tanks did not comply with ADWG (2011) limits of turbidity (Magyar et al., 2007, 2008). The specific tanks that did not comply were smaller ones that the user did not rely on for everyday potable needs. Generally the larger the rainwater tank, the less the effects are of the first flush as it is diluted into a much larger body of stored water. Furthermore a larger tank capacity was able to capture more water from a longer-lasting storm event.

4.3.2. Water hardness and conductivity

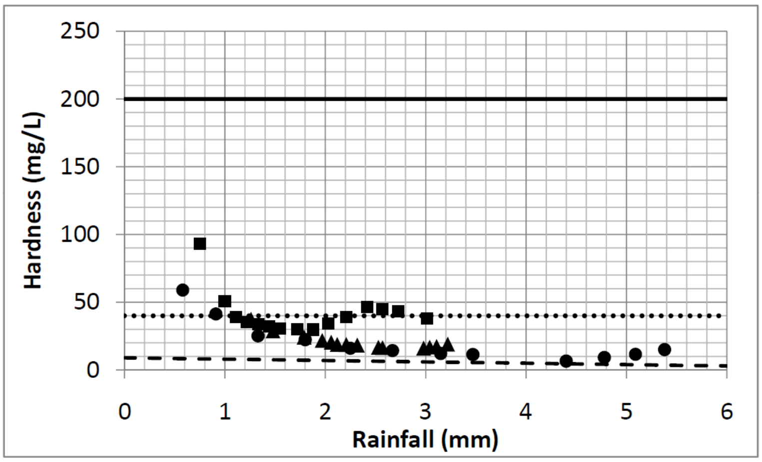
Figure 4.4 (c) shows that all samples of water hardness were below the limit of 200 mg/L ($CaCO_3$ equivalent). Tables 4.3 to 4.5 indicate that conductivity is well below the ADWG (2011) limit of 0.8 dS/m.



(a)

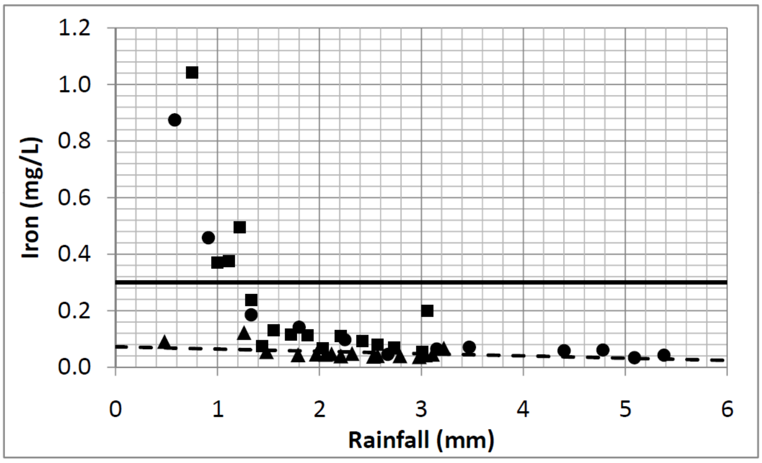


(b)

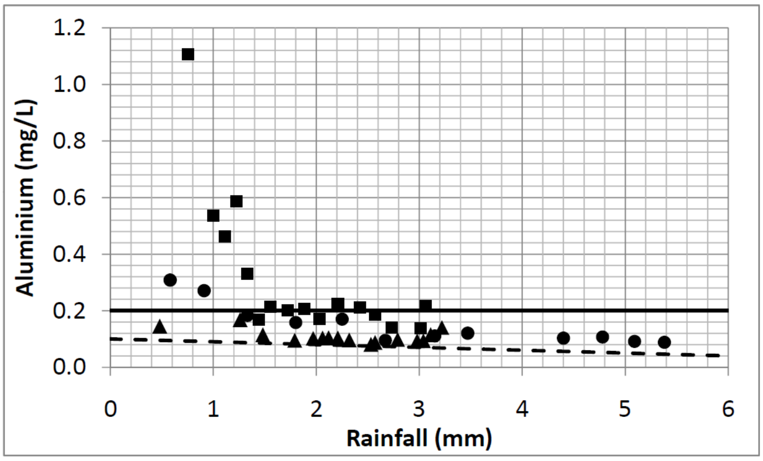


(c)

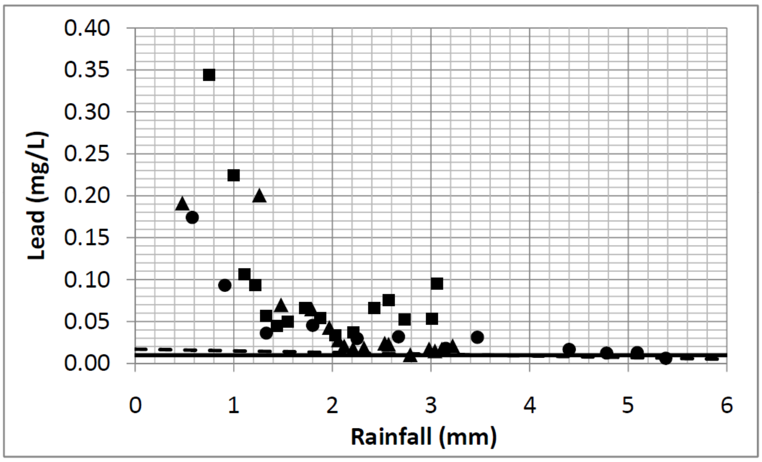
Figure 4.4 (a-h) Physical and chemical characteristics of wash-off values from a concrete tiled roof of an urban house compared with ADWG (2011) limit values



(d)

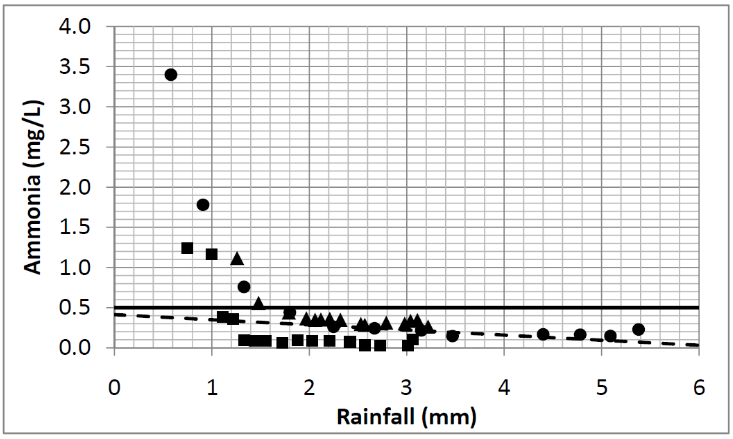


(e)

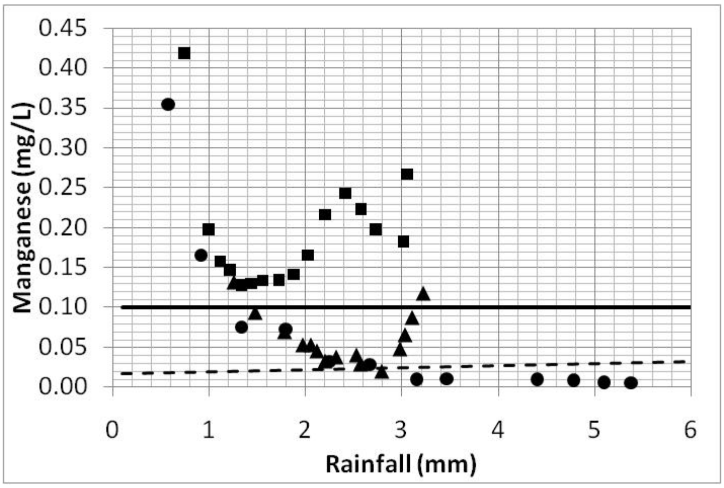


(f)

Figure 4.4 (a-h) contd.



(g)



(h)

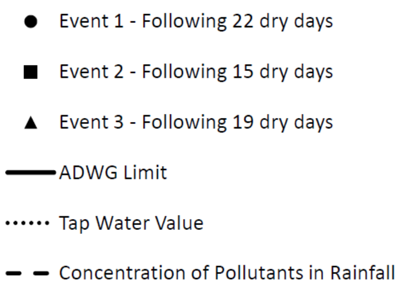


Figure 4.4 (a-h) contd.

4.3.3. Heavy metals

Figure 4.4 (d-f) and Tables 4.3 to 4.5 show the data for metals. Figure 4.4 (d and h) shows that the iron and manganese ADWG (2011) limits of 0.3 and 0.1 mg/L, respectively, are exceeded in runoff from the first 1 to 1.5 mm of rainfall. Iron is an essential trace element for humans. The limit concerning iron is more an aesthetic one due to iron's tendency to stain objects it comes in contact with or because it can lead to the water having a rust-brown colour (ADWG, 2011). Figure 4.4 (e) demonstrates that the ADWG (2011) aluminium limit of 0.2 mg/L is exceeded in the first 1.5 mm of rainfall coming off the roof.

Figure 4.4 (f) shows lead is of most concern with the levels exceeding the ADWG (2011) limit of 0.01 mg/L until runoff from the first 5 mm to 6 mm of rainfall is bypassed. Figure 4.4 (f) indicates the concentration of lead in rainwater (before contact with the roof) is 0.02 mg/l and is above the ADWG (2011) limit for the first 3 mm to 4 mm of rainfall. The lead concentration in roof runoff was probably due to atmospheric deposition on the roof. In the intervening dry days the concentration of lead on the roof builds up from atmospheric deposition. Lead is a health hazard to humans (ADWG, 2011). There was no lead flashing on the roof of the house used for this analysis.

Other metals such as arsenic, cadmium, chromium, copper, mercury, nickel, selenium and silver were either not detected or less than 0.002 mg/L.

4.3.4. Ammonia, nitrate, nitrite and orthophosphate

Figure 4.4 (g) and Tables 4.3 to 4.5 demonstrate that levels of ammonia were above the ADWG (2011) limit of 0.5mg/L in runoff from the roof during the first 1.5 mm of rainfall. The other nutrients, i.e. nitrate, nitrite and orthophosphate concentrations, did not show any consistent pattern and randomly varied with ongoing rainfall (Tables 4.3 to 4.5). Nitrate concentrations were low ranging between 0 and 2.0 mg/L with two outlying samples at 3.5 and 7 mg/L. The ADWG (2011) limit of nitrate is 50 mg/L. Nitrite concentrations were also low ranging between 0 and 0.35 mg/L with 2 outlying samples at 0.9 and 1.95 mg/L. The ADWG (2011) limit of nitrite is 3mg/L. Orthophosphate concentration ranged between 0 and 0.2 mg/L (Tables 4.3 to 4.5).

4.3.5. Anions and cations and total dissolved salts

ADWG (2011) does not provide recommended limits for these parameters. Similar to the other tested parameters, the first flush parameters of anions and cations were typically also more concentrated in the first 3 to 5 mm of runoff compared to the typical T1 tank water.

Table 4.3 First flush rainwater samples from Event 1

Parameter	pH	TSS (mg/L)	Turbidity (NTU)	Conductivity (EC)	TDS (mg/L)	Water hardness (mg/L$CaCO_3$ equivalent)	Orthophosphate (mg/L P)	Nitrate (mg/L N)	Nitrite (mg/L N)	Ammonia (mg/L N)	Sodium (mg/L)	Chloride (mg/L)	Sulphate (mg/L SO_4^{2-})	Potassium (mg/L)	Calcium (mg/L)	Magnesium (mg/L)	Aluminium (mg/L)	Iron (mg/L)	Manganese (mg/L)	Lead (mg/L)	Zn (mg/L)
ADWG (2011) Limit	6.5-8.5	-	<5	<0.8	-	<200	-	<50	<3	<0.5	<300	<400	<400	-	-	-	<0.2	<0.3	<0.1	<0.01	<3
0.58 mm	6.51	428	**42**	0.24	160	59	0.003	0.022	0.005	**3.401**	9.4	16.6	18.7	4.6	21.1	0.355	**0.308**	**0.875**	**0.355**	**0.174**	0.190
0.91 mm	6.53	273	**30**	0.17	117	41	0.005	0.016	0.003	**1.782**	7.6	12.4	15.4	3.0	14.9	0.167	**0.271**	**0.458**	**0.167**	**0.093**	0.123
1.33 mm	6.55	107	**18**	0.12	84	25	0.001	0.014	0.002	**0.759**	5.8	6.5	10.5	2.1	9.1	0.077	0.183	0.185	0.077	**0.036**	0.057
1.8 mm	6.57	60	**9**	0.11	72	22	0.001	0.016	0.003	0.442	5.2	5.1	9.2	1.9	8.0	0.074	0.158	0.142	0.074	**0.046**	0.056
2.25 mm	6.62	59	**10**	0.08	56	16	0.005	0.361	0.063	0.262	4.0	3.3	6.7	1.2	5.7	0.033	0.170	0.097	0.033	**0.030**	0.042
2.67 mm	6.55	77	**9**	0.08	53	14	0.005	0.260	0.021	0.243	3.4	2.5	5.0	1.0	5.2	0.029	0.095	0.045	0.029	**0.032**	0.049
3.15 mm	6.61	57	**10**	0.06	44	12	0.004	0.389	0.020	0.217	2.9	2.0	3.7	0.8	4.4	0.010	0.111	0.065	0.010	**0.018**	0.033
3.47 mm	6.58	76	**8**	0.06	41	11	0.004	0.333	0.018	0.146	2.6	1.7	3.4	0.6	4.1	0.012	0.121	0.071	0.012	**0.031**	0.035
4.4 mm	6.66	61	**10**	0.04	29	7	0.006	0.248	0.012	0.168	1.7	1.2	2.5	0.3	2.4	0.009	0.103	0.059	0.009	**0.017**	0.019
4.78 mm	6.64	33	**7**	0.05	36	9	0.008	0.175	0.004	0.165	2.3	1.8	3.0	0.6	3.3	0.008	0.107	0.061	0.008	**0.012**	0.039
5.09 mm	6.59	29	**5**	0.06	41	12	0.005	0.186	0.005	0.147	2.6	1.5	3.5	0.8	4.2	0.006	0.092	0.034	0.006	**0.013**	0.021
5.38 mm	6.59	17	3	0.08	52	15	0.023	0.114	0.005	0.229	3.2	3.1	4.2	1.2	5.4	0.007	0.088	0.043	0.007	0.006	0.021
Rain Water (0-3 mm)	6.70	54	3	0.05	30	6	0.037	0.334	0.011	0.222	2.0	2.3	4.8	0.3	2.1	0.026	0.078	0.048	0.026	0.009	0.047
Rain Water (3-6 mm)	6.75	21	3	0.03	16	4	0.003	0.086	0.005	0.147	2.2	2.2	1.7	0.2	1.3	0.008	0.061	0.038	0.008	0.005	0.019
Tap Water	6.93	0.5	0.2	0.18	122	36	0.000	0.193	0.051	0.205	10.3	26.1	3.6	0.8	11.6	1.600	0.013	0.007	0.000	0.000	0.024

Notes:
Values exceeding the ADWG (2011) limit are shown in bold
Arsenic, Cadmium, Chromium, Copper, Mercury, Nickel, Selenium and Silver were either not detected or less than 0.002 mg/L
Rain Water – rainwater before contact with roof

Table 4.4 First flush rainwater samples from Event 2

Parameter	**pH**	**TSS (mg/L)**	**Turbidity (NTU)**	**Conductivity (EC)**	**TDS (mg/L)**	**Water hardness (mg/LCaCO$_3$ equivalent)**	**Orthophosphate (mg/L P)**	**Nitrate (mg/L N)**	**Nitrite (mg/L N)**	**Ammonia (mg/L N)**	**TOC (mg/L)**	**Sodium (mg/L)**	**Chloride (mg/L)**	**Sulphate (mg/L SO$_4^{2-}$)**	**Potassium (mg/L)**	**Calcium (mg/L)**	**Magnesium (mg/L)**	**Aluminium (mg/L)**	**Iron (mg/L)**	**Manganese (mg/L)**	**Lead (mg/L)**	**Zn (mg/L)**
ADWG (2011) Limit	6.5-8.5	-	<5	<0.8	-	<200	-	<50	<3	<0.5	-	<300	<400	<400	-	-	-	<0.2	<0.3	<0.1	<0.01	<3
0.75mm	6.34	65	**20**	0.35	240	93	0.026	6.926	1.954	**1.244**	39.6	16.2	58.4	28.1	7.2	33.0	0.418	**1.107**	**1.043**	**0.418**	**0.344**	0.151
1.00 mm	6.38	67	**18**	0.21	142	51	0.005	3.579	0.900	**1.170**	22.8	10.1	27.4	15.9	4.3	18.0	0.198	**0.536**	**0.369**	**0.198**	**0.225**	0.107
1.11 mm	6.32	50	**10**	0.17	112	39	<0.005	1.894	0.258	0.389	21.1	8.6	21.1	12.2	3.8	13.7	0.159	**0.464**	**0.375**	**0.159**	**0.106**	0.058
1.22 mm	6.27	41	**15**	0.15	104	35	0.019	1.855	0.138	0.358	20.9	8.5	19.5	10.7	3.8	12.2	0.147	**0.586**	**0.496**	**0.147**	**0.093**	0.056
1.33 mm	6.24	29	**10**	0.14	95	34	0.015	1.650	0.290	0.097	19.6	8.3	17.3	9.4	3.9	11.8	0.128	**0.330**	0.239	**0.128**	**0.057**	0.043
1.44 mm	6.24	23	**6**	0.14	92	32	0.015	1.311	0.274	0.085	20.3	8.4	17.3	9.1	4.1	11.2	0.131	0.167	0.075	**0.131**	**0.045**	0.039
1.55 mm	6.28	20	**9**	0.13	90	31	0.023	1.285	0.208	0.086	22.3	8.1	16.6	8.4	4.0	10.5	0.135	**0.214**	0.132	**0.135**	**0.050**	0.042
1.72 mm	6.28	21	**6**	0.14	97	30	0.020	0.854	0.057	0.060	21.0	7.9	15.9	8.0	3.9	10.3	0.134	**0.201**	0.116	**0.134**	**0.066**	0.045
1.88 mm	6.29	16	**6**	0.13	91	30	0.032	1.366	0.348	0.093	21.2	7.8	15.9	7.9	3.9	10.2	0.143	**0.206**	0.114	**0.143**	**0.054**	0.042
2.03 mm	6.32	17	4	0.15	101	34	0.053	1.210	0.352	0.088	25.6	9.0	18.4	8.8	4.8	11.7	0.167	0.171	0.067	**0.167**	**0.033**	0.048
2.21 mm	6.34	19	4	0.17	112	39	0.085	0.366	0.103	0.089	31.6	10.4	23.0	10.1	5.5	13.4	0.217	0.224	0.111	**0.217**	**0.037**	0.065
2.42 mm	6.45	18	4	0.19	130	46	0.096	0.418	0.116	0.074	34.1	11.5	25.4	10.9	6.4	15.8	0.244	0.212	0.094	**0.244**	**0.066**	0.076
2.57 mm	6.53	19	3	0.18	123	45	0.074	0.693	0.023	0.030	26.6	10.4	21.2	9.8	5.5	15.4	0.223	0.186	0.081	**0.223**	**0.076**	0.065
2.73 mm	6.11	14	3	0.19	128	43	0.070	0.585	0.018	0.025	24.8	9.8	19.9	8.8	5.2	15.0	0.198	0.141	0.069	**0.198**	**0.053**	0.054
3.01 mm	6.50	14	3	0.16	108	38	0.071	0.447	0.018	0.025	20.1	8.4	16.2	7.8	4.8	13.1	0.182	0.137	0.056	**0.182**	**0.053**	0.051
3.06 mm	6.48	19	4	0.21	143	51	0.025	1.188	0.078	0.108	27.7	10.6	26.8	11.6	5.5	17.8	0.268	0.217	0.058	**0.268**	**0.095**	0.066
Rain Water (0-3 mm)	5.88	30	5	0.05	35	6	<0.005	0.604	<0.005	0.465	7.5	4.2	6.9	4.1	0.9	1.2	0.032	0.123	0.054	0.032	0.005	0.032
Tap Water	6.93	0.5	0.2	0.18	122	36	0.000	0.193	0.051	0.205	2.7	10.3	26.1	3.6	0.7	11.7	0.004	0.013	0.007	0.000	0.000	0.024

Notes:Values exceeding the ADWG (2011) limit are shown in bold; Arsenic, Cadmium, Chromium, Copper, Mercury, Nickel, Selenium and Silver were either not detected or less than 0.002 mg/L; Rain Water – rainwater before contact with roof

Table 4.5 First flush rainwater samples from Event 3

Parameter	pH	TSS (mg/L)	Turbidity (NTU)	Conductivity (EC)	TDS (mg/L)	Water hardness (mg/L $CaCO_3$ equivalent)	Orthophosphate (mg/L P)	Nitrate (mg/L N)	Nitrite (mg/L N)	Ammonia (mg/L N)	Sodium (mg/L)	Chloride (mg/L)	Sulphate (mg/L SO_4^{2-})	Potassium (mg/L)	Calcium (mg/L)	Magnesium (mg/L)	Aluminium (mg/L)	Iron (mg/L)	Manganese (mg/L)	Lead (mg/L)	Zn (mg/L)
ADWG (2011)	6.5-8.5	-	<5	<0.8	-	<200	-	<50	<3	<0.5	<300	<400	<400	-	-	-	<0.2	<0.3	<0.1	<0.01	<3
0.48 mm	6.49	51	**12**	0.22	150	47	0.13	1.182	0.013	0.435	9.8	27	19	9.1	15.7	1.8	0.146	0.091	**0.170**	**0.191**	0.126
1.26 mm	6.53	49	**8**	0.17	116	37	0.12	0.986	0.021	**1.117**	6.9	17	15	5.2	12.8	1.3	0.168	0.123	**0.132**	**0.201**	0.111
1.48 mm	6.52	47	**6**	0.13	86	29	0.05	1.161	0.023	**0.559**	4.8	10	11	3.2	10.0	0.9	0.115	0.054	0.094	**0.070**	0.058
1.79 mm	6.51	46	**8**	0.11	73	24	0.03	1.056	0.016	0.438	4.2	8	9	2.6	8.4	0.8	0.095	0.044	0.070	**0.065**	0.046
1.97 mm	6.48	43	**6**	0.10	67	22	0.02	0.870	0.010	0.367	4.0	8	7	2.7	7.6	0.7	0.102	0.047	0.053	**0.043**	0.036
2.06 mm	6.46	48	**4**	0.09	64	20	0.02	0.805	0.010	0.349	4.0	7	7	2.6	7.0	0.7	0.104	0.045	0.053	**0.028**	0.029
2.12 mm	6.50	38	**6**	0.09	61	19	0.03	0.768	0.011	0.352	3.8	7	6	2.6	6.5	0.6	0.104	0.047	0.046	**0.021**	0.026
2.21 mm	6.45	36	**6**	0.08	57	19	0.03	0.770	0.010	0.362	3.7	6	6	2.5	6.5	0.6	0.104	0.040	0.033	**0.018**	0.024
2.32 mm	6.41	34	**5**	0.08	52	18	0.03	0.739	0.010	0.350	3.8	6	6	2.4	6.3	0.6	0.096	0.049	0.038	**0.018**	0.024
2.53 mm	6.41	30	**6**	0.08	52	17	0.03	0.649	0.010	0.298	3.2	5	5	2.1	5.9	0.5	0.081	0.039	0.040	**0.024**	0.022
2.57 mm	6.40	26	4	0.07	50	16	0.03	0.642	0.009	0.286	3.1	5	5	2.1	5.8	0.5	0.088	0.041	0.029	**0.023**	0.020
2.79 mm	6.56	23	5	0.07	50	16	0.03	0.565	0.009	0.313	3.3	5	5	2.4	5.6	0.5	0.098	0.040	0.020	**0.010**	0.017
2.98 mm	6.35	20	5	0.07	53	16	0.04	0.522	0.008	0.301	3.5	5	5	2.5	5.8	0.6	0.094	0.037	0.049	**0.017**	0.021
3.04 mm	6.31	18	3	0.08	56	17	0.05	0.491	0.011	0.335	3.7	6	5	2.7	6.0	0.6	0.093	0.043	0.066	**0.015**	0.023
3.11 mm	6.30	19	5	0.08	62	17	0.06	0.449	0.014	0.346	4.2	7	5	3.2	6.5	0.7	0.115	0.046	0.087	**0.017**	0.027
3.22 mm	6.29	19	8	0.09	72	19	0.09	0.195	0.016	0.265	5.1	9	6	4.1	7.2	0.9	0.140	0.067	0.118	**0.021**	0.036
Rain Water (0-3 mm)	6.49	22	3	0.14	18	30	0.01	0.535	0.019	0.401	1.5	2	2	0.4	0.8	0.2	0.068	0.030	0.030	**0.017**	0.039
Tap Water	6.93	0.5	0.2	0.18	122	36	0.000	0.193	0.051	0.205	10.3	26.1	3.6	0.8	11.6	1.600	0.013	0.007	0.000	0.000	0.024

Notes:

Values exceeding the ADWG (2011) limit are shown in bold

Arsenic, Cadmium, Chromium, Copper, Mercury, Nickel, Selenium and Silver were either not detected or less than 0.002 mg/L

Rain Water – rainwater before contact with roof

4.3.6. Molecular weight distribution of RWOM for the effect of continuous rainfall after the roof contact

Understanding how RWOM varies in the first flush of the roof runoff can provide valuable information for the design, maintenance, and operation of a rainwater tank. A detailed variation of molecular weight distribution (MWD) of RWOM was monitored during a period of continuous rainfall (rainfall event 15/11/08; see Table 3.15 from Section 3.3.1) falling on the roof (Figure 4.5). The first flush generated from up to 2 mm rainfall after contact with the roof was collected and analysed.

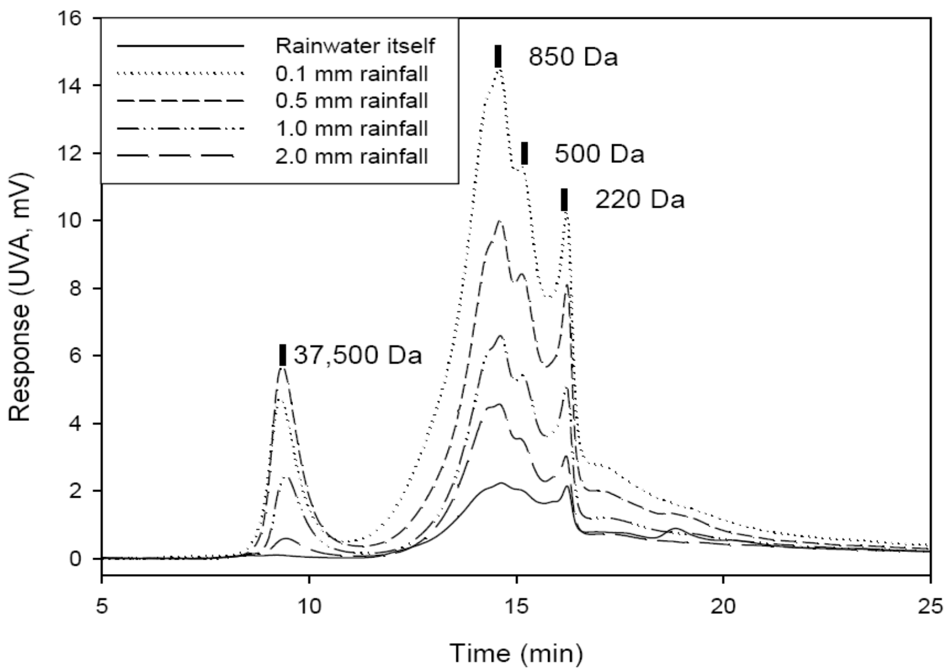


Figure 4.5 MWD of RWOM with the effects of continuous rainfall after the roof contact.

As rainfall falls on the roof and washes the pollutants off it, the diversion of the first part of the runoff from the roof also diverts organic contaminants away from the rainwater tank. The first part of the runoff generated is commonly referred to as the first flush. The knowledge of how RWOM varies with increasing amounts of rainfall and in the first flush can provide valuable information for the maintenance and operation of a rainwater tank. A detailed variation of MWD of RWOM was monitored during a period of continuous rainfall on the roof (Figure 4.5). The first flush generated from up to 2 mm rainfall after contact with the roof was investigated.

Overall, the intensity of UVA responses significantly increased following rain contact with the roof. At 0.1 mm of roof runoff, the MWD of RWOM included five major peaks, namely 37500 Da, 850 Da, 500 Da and 220 Da. According to one study, the MW of 37500 Da may be due to biopolymers, 850 Da to humic substances, 500 Da to building blocks, 220 Da to low MW acids, and less than 220 Da to amphiphilics, (Shon et al., 2006). As the roof runoff increased up to 2 mm, the intensity of the MWD of RWOM generally decreased. However, a preferential removal of specific MW was not detected, suggesting that the initial flushing carries the majority of organic contaminants. The concentration of organic matter decreases with further runoff. After 2 mm of continuous roof runoff, the intensity of the MWD peaks from the start of the runoff reduced to the extent that it began to resemble results for rainwater that did not come into contact with the roof. This implies that diverting the initial 2 mm of rainfall from the roof can reduce the need for, or can simplify, the treatment process to remove RWOM.

4.4. Comparison of the various tested water sources

Table 4.6 compares the water quality collected in raintank T1, first flush flowing to T1, rainfall at T1 before it came into contact with the roof, rural raintank water and tap water (at Ingleburn where T1 is located and at UTS). As described in Sections 4.2 to 4.4, the quality of the water in rainwater tanks is comparable with potable supply water with the exception of heavy metals (lead and iron), and the turbidity level in the metropolitan tanks and pH in the rural tanks. In Table 4.6, non-compliance with ADWG (2011) is shown in bold. The first flush that by-passes the tank can have a significant influence on improving the water quality in the rainwater tank. However, minimal forms of treatment will still be required especially for the metropolitan rainwater tanks.

Table 4.6 Comparison of Water Quality at T1 with ADWG, Rainfall and Potable Water

Parameter	ADWG (2011)	T1 RWT *	T1 1st flush	Rain fall at T1	Rural Average	Ingleburn tap water (T1)	University of Technology tap water
pH	6.5 -8.5	7.13	6.51	6.66	**6.19**	6.93	6.66
Conductivity (EC) (dS/m)	< 0.8	0.08	0.24	0.06	-	0.179	0.181
Total dissolved salts (mg/L)	-	55.31	160	39	16.5	122	123
Total suspended solids (mg/L)	-	6.17	428	6.7	<0.5	0.5	0.5
Turbidity (NTU)	<5	**5.07**	**42**	**5**	0.71	0.2	1
Water hardness (mg/L $CaCO_3$ equivalent)	<200	27.65	59	9	4.8	36	29
Orthophosphate (mg/L P)	-	0.02	0	0.07	0.014	0	0.005
Nitrate (mg/L N)	<50	0.33	0.022	0.352	0.207	0.193	0.383
Nitrite (mg/L N)	<3	0.00	0.005	0.020	<0.005	0.051	0.005
Ammonia (mg/L N)	<0.5	0.056	**3.401**	0.413	0.006	0.205	0.01
Sodium (mg/L)	<300	2.62	9.4	1.7	2.951	10.3	11.3
Potassium (mg/L)	-	1.29	4.6	0.5	0.729	0.8	0.6
Calcium (mg/L)	-	10.30	21.1	3.4	1.276	11.6	7.3
Magnesium (mg/L)	-	0.47	1.5	0.2	0.384	1.6	2.6
Chloride (mg/L)	<400	4.63	16.6	1.5	3.72	26.1	31.7
Sulphate (mg/L SO_4^{2-})	<400	1.42	18.7	7.3	5.3	3.6	7.5
Aluminium (mg/L)	<0.2	0.08	**0.308**	0.108	0.016	0.013	0.01
Copper (mg/L)	<2	0.03	0.009	0.006	0.018	0.247	1.474
Iron (mg/L)	<0.3	**1.69**	**0.875**	0.072	<0.01	0.007	0.171
Manganese (mg/L)	<0.1	0.02	**0.355**	0.017	0.0115	0	0.01
Lead (mg/L)	<0.01	**0.016**	**0.174**	**0.017**	<0.001	0	0
Zinc (mg/L)	<3	0.13	0.190	0.077	0.2975	0.024	0.076

Notes:
* Mean average value shown, the range is given in Tables 4.1 and 4.2
Values exceeding the ADWG (2011) limit are shown in bold

4.5. Conclusion

4.5.1. Characterisation of Sydney metropolitan rainwater tanks

Based on the results for testing the domestic roof-collected rainwater supplies in the Sydney metropolitan area, the water quality is predominantly suitable for drinking when compared to the ADWG (2011) requirements. The pollutants that did not comply in a few rainwater tanks were the heavy metals, in particular the concentrations of iron and lead, pH and turbidity (see Table 4.1). All samples were within acceptable levels for salts and minerals analysed. The MWD of samples indicating prominent peaks at 37500 Da may be due to biopolymers, 850 Da to humic substances, 500 Da to building blocks, 220 Da to low MW acids, and less than 220 Da to amphiphilics.

4.5.2. Characterisation of outer Sydney rural rainwater tanks compared to Sydney metropolitan rainwater tanks

Detailed sampling and analysis was conducted in rural rainwater tanks located 160 kilometres south-west of Sydney. Overall, the water collected in rural rainwater tanks generally complied with the standards for most parameters in the ADWG (2011) except for pH and faecal and total coliforms (see Table 4.2).

4.5.3. Characterisation of the first flush in Sydney metropolitan rainwater tanks

The analysis of the first flush of the roof runoff from an urban residential roof (T1) located in Sydney's metropolitan area, shows that quality of the water could improve significantly after bypassing the first 2 mm of rainfall. The pollutants (excluding faecal and total coliforms) that did not comply were turbidity and lead, which required the about the first 5 mm of rainfall to be bypassed in order to meet the limits specified by ADWG (2011). An increased annual rainwater tank yield can be realised from bypassing the first 2 mm of rainfall if lead and turbidity are reduced to acceptable levels by treatment.

MWD analysis shows that the concentration of RWOM declined when roof runoff increased. After 2 mm of continuous roof runoff from when the rain started, the intensity of the peaks of the MWD distribution of RWOM decreased. In fact the peaks began to resemble the peaks observed in rainwater itself.

The findings demonstrate that diverting the first flush off a roof that is heavily polluted, could significantly improve the water quality of the rainwater collected in the tank. Furthermore it has the potential to reduce treatment requirements and associated energy consumption.

Chapter 5

University of Technology, Sydney

Faculty of Engineering & Information Technology

5. Water Demand

5.1. Introduction

In Australia, potable water demand is expected to increase above the current available water supplies due to: firstly, the predicted population increases in capital cities; and secondly, the recurring drought exacerbated by climate change. In this chapter, the water consumption pattern in Sydney's metropolitan area was analysed based on the metered potable water usage of all residential properties between January 2002 and October 2009, an important factor to determine the size of the rainwater tank for potable water usage. Additionally this chapter compares the potable water consumption of all residential properties in the Sydney metropolitan area against the residential properties that installed a rainwater tank and received a rebate from Sydney Water Corporation (SWC). The water usage consumption before and after rainwater tanks were installed was analysed to quantify the amount by which these tanks reduced water consumption. The results were compared against socio-demographic factors and analysed in detail. For high resolution figures refer to Appendix A.

5.2. Background and data

5.2.1. Climate

The average rainfall in Sydney over the period 1913 to 1998 was 1203 mm/yr. The year is sub-divided into quarters that approximately follow the seasons: Q1 from November to January; Q2 from February to April; Q3 from May to July; and Q4 from August to October. The wettest period of the year is generally in the second quarter (Q2) while the driest part of the year is the fourth quarter (Q4). The average quarterly and yearly rainfall patterns over the study period (2002 to 2009) indicate a large variance from year to year. The driest year was 2005 with a total annual rainfall of only 808 mm while the wettest year was 2007 with 1325 mm of rainfall. The year 2005 was the third hottest summer recorded in NSW (Randolph and Troy, 2007). Some notable periods during the study period include the fourth quarter of 2002 (Q4-2002) and first quarter of 2003 (Q1-2003) where a total of only 47 mm and 120 mm of rainfall occurred respectively; and the third quarter of 2007 (Q3-2007) where a total of 588 mm of rainfall occurred, which was almost three quarters of the total rainfall occurring in 2005 and well above the quarterly average.

High average temperatures can be a leading factor for increased water consumption due to the need for additional watering of gardens and lawns and higher overall consumption. The quarterly and annual average temperatures were 25.6 (Q1), 25.2 (Q2), 19.1 (Q3), 21.7 (Q4), and the annual average was 22.9. The hottest period of the year is generally in the first and second quarter (November through April) while the coldest part of the year is the third quarter (May through July).

5.2.2. Water Restrictions

SWC implemented various water restrictions over the past decade due to the declining water levels in dams that supply the Sydney metropolitan area (SWC 2012a; and Table 5.1). Voluntary restriction commenced in Sydney in October 2002. SWC introduced Level 1 restrictions in October 2003 when dam levels dropped below 60% (Sydney Water, 2012a). Dam levels continued to decline due to the lack of rainfall in the dams' catchment and by June 2004 were below 50% when SWC implemented Level 2 restrictions (Sydney Water, 2012a). In June 2005 dam levels had dropped further to below 40% resulting in Level 3 restrictions (Sydney Water, 2012a). Level 3 restrictions remained in place for a number of years. In June 2008, SWC eased Level 3 restrictions to permit its residential customers to wash cars, boats and caravans at home as well as to clean the windows and wall of their house with a hose as long as a trigger nozzle was fitted. By June 2009, water levels in the dams remained steady at around 60% for 12 months. Level 3 restrictions ceased and SWC brought in new 'Water Wise Rules' (Sydney Water, 2012a) which are still in effect despite dam water levels being currently at 100%.

5.2.3. SWC Rainwater Tank Rebate

In October 2002, SWC introduced a rainwater tank rebate scheme in an effort to promote the adoption of rainwater tanks as a measure to reduce water consumption. The scheme provided various rebates (monetary refunds) for the installation of rainwater tanks. $150 was provided for the installation of rainwater tanks with a capacity of between 2,000 to 3,999 L, $400 for 4,000 to 6,999 L and $500 for more than 7,000 L. In addition to this, further monetary incentives ($150) were provided to connect the rainwater tank to the household toilet system and/or the laundry system. This was increased to $300 from October 2006. In July 2007, $500 was given for a rainwater tank connected to the household toilet system and/or $500 for a connection to the laundry system. This scheme ended in June 2011.

Table 5.1 SWC Water restrictions (Sydney Water, 2012a)

Level	Restriction
Level 1 **October 2003**	No hosing of hard surfaces; and No sprinklers or watering systems.
Level 2 **June 2004**	No hosing of hard surfaces; No sprinklers or watering systems; No hosing of lawns and gardens except hand-held hosing before 9 am and after 5 pm on Wednesdays, Fridays and Sundays; and No filling of new or renovated pools over 10,000 L except with a permit from Sydney Water.
Level 3 **June 2005**	No hosing of hard surfaces; No sprinklers or watering systems; Hosing of lawns and gardens only allowed on Wednesdays and Sundays before 10 am and after 4 pm; No filling of new or renovated pools over 10,000 L except with a permit from Sydney Water; No hoses or taps to be left running unattended, except when filling pools or containers; and Fire hoses used only for firefighting purposes - not for cleaning.
Water Wise Rules **June 2008**	All hoses must now have a trigger nozzle; To avoid the heat of the day, watering is allowed only before 10 am and after 4 pm; No hosing of hard surfaces such as paths and driveways. Washing vehicles is allowed; and Fire hoses must only be used for firefighting activities only.

5.2.4. SWC Data Base

SWC supplies water to properties within the Sydney metropolitan area. A database of water bills, which are issued quarterly and include the amount of water used at a property, which belongs to SWC were used to undertake a statistical analysis on residential water usage in Sydney. The database covered the period November 2001 (Q1-02) to October 2009 (Q4-09). The quarters approximately follow the seasons: Q1 from November to January; Q2 from February to April; Q3 from May to July; and Q4 from August to October. The first quarter in the dataset (Q1) was for the period November 2001 to February 2002. The total number of properties in the database totalled 1,207,359, of which 962,697 were categorised as 'Residential - Single Dwelling'. By 2009, there were a total of 52,576 households registered for a rainwater tank rebate with SWC which represented 5.5% of the households supplied by SWC. The data made available to this study related to single dwelling residential properties. The average quarterly consumption for these properties with and without rainwater tanks for each local government authority (LGA) was provided. Data was released in this restricted form to comply with Sydney Water privacy restrictions which prohibit releasing any information concerning their individual customers.

The SWC database covers a large data set of various property types, land sizes and water usages. A subset was created - Conditional Data Set – Residential Properties (CDSR) - to remove data not used in this study. The CDSR was separated into individual LGAs to provide a location-based investigation and further isolated properties to the following conditions: single dwelling residential properties only; properties with a lot area greater then 100m² and no larger than 2000 m² as larger properties are not typically urban residential; and properties with typical water usage between 10 kL to 500 kL per quarter (typically 100 L to 5000 L per day). Subsequent to creating the CDSR, a second data set was created with these same conditions in addition to including only properties that had a registered rainwater tank rebate (CDSRT).

5.2.5. Standardisation of Data Set

To ensure that the data set contains consistent data to compare the water usage before and after the installation of a rainwater tank, the CDSRT needed to be standardised. To standardise the data, individual LGA CDSR were graphed to observe the typical water usage trend over the study period (Q1-02 to Q4-09) to determine the impact of factors that led to changes in water usage other than the rainwater tank. These factors include the effects of Sydney Water's restrictions on water usage, seasonal and climate variations, and the changes in the consumer's water usage pattern (e.g. as a result of public education campaigns).

The CDSR data was grouped by LGAs. For each LGA a factor was determined for each quarter based on the average of the respective quarterly water usage compared to the average of the LGAs water usage over the study period (Q1-02 to Q4-09). Subsequently, these factors were applied to the respective individual quarterly water bills of the CDSRT. The data set with factors applied is called the Standardised Conditional Data Set with rainwater tanks (S-CDSRT). The outcome of applying the factors to the CDSRT allowed a comparison between water usage, before and after the rainwater tanks were installed with any of the other effects discussed above removed. The reduction in water consumption was calculated for rainwater tanks over a period of at least two years (8 quarters) before and at least two years after the installation. There were 32,276 properties with rainwater tanks or 61.4% of the total number of properties with rainwater tanks that met this criterion. The portion of rainwater tanks meeting this criterion was over 50% in all but 6 LGAs. Any decline in the average water usage trend after the installation of the rainwater tank would confirm that installing a rainwater tank had resulted in a reduction of potable water usage.

5.3. Results

5.3.1. Water consumption for residential single dwellings

Figure 5.1 shows the reduction in water consumption in Sydney and in various LGAs over the period leading up to and including the Sydney water restrictions. The overall trend in the data in terms of the times when water consumption rises, falls and plateaus is similar. The consumption sharply fell during the period of voluntary restriction and reflects the community education and publicity campaign carried out to save water and the decline in dam storage to critical levels over that period. The saving was so large that the reduction in water consumption level during periods of level 1, 2 and 3 actually leveled off even as restrictions became increasingly more severe. Savings in water consumption became harder to achieve once practices and habits in the community that were easier to change were accomplished. The average reduction in water consumption during the period 2002-2009 was about 25%. Not all the saving in water consumption can be attributed to the program of water restrictions. Concurrently, during this period other measures were implemented to reduce water consumption. This included the installation of water savings devices such as water efficient shower heads and dual flush toilets, the implementation of the Building Sustainability Index (BASIX) for new and refurbished buildings which aimed at delivering effective water reductions in Sydney and across New South Wales, Australia, and the continual replacement of water appliances with new ones that were typically more efficient.

Figure 5.1 shows the variation in water demand in Campbelltown and Hornsby which have the lowest and highest rainfall, respectively. The annual rainfall in Hornsby is approximately double that of Campbelltown. Both suburbs are similar in that they are typically middle class with predominantly single dwelling houses and large gardens. While the difference in water consumption between the two LGAs is not large (<5%) Hornsby had a slightly lower consumption. The plot also shows the LGAs with the largest (Ku-ring-gai) and smallest (Auburn) reduction in water consumption over the period of the water restrictions. The former is a wealthy leafy suburb with predominantly single dwelling houses with large gardens while the former is an inner city suburb with terrace type houses and very small outdoor/garden areas. While there was tangible reduction in both suburbs, the reduction in Ku-ring-gai was substantial. In Perth, studies with smart metering showed a significant reduction came from outdoor water usage (Loh and Coghlan, 2003; Water Corporation, 2009). The result in Ku-ring-gai reflects a similar characteristic. Both Ku-ring-gai and Auburn had similar water consumption at the end of the monitoring period and this was about 10% larger than the Sydney average.

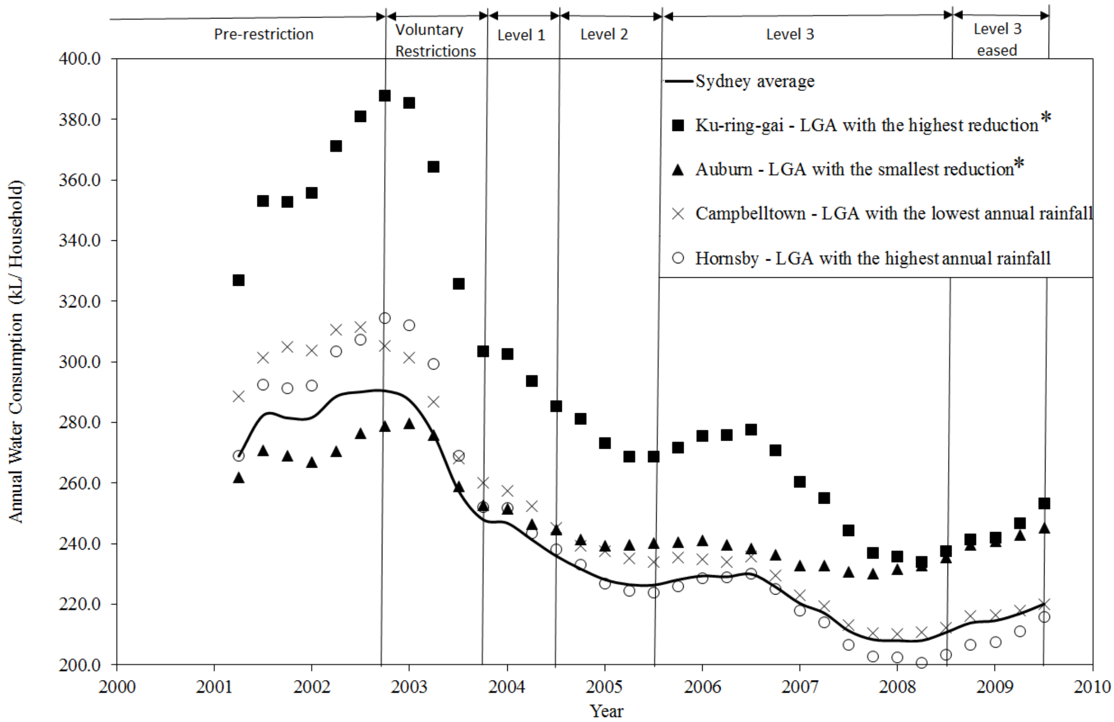


Figure 5.1 Variation in average water consumption in Sydney and its LGAs.

Figures 5.2 to 5.5 presents an overview of water consumption in Sydney together with various socio-economic indicators (Figure 5.3) which were prepared from the SWC database together with data from the Australian Bureau of Statistic (ABS, 2006a), and Bureau of Meteorology (BOM, 2012). The data across Sydney is presented by LGA. For clarity, the overarching map containing most of the LGAs is referred to as the 'outer Sydney area' and the smaller insert is referred to as the 'inner Sydney area'.

The SWC data shows that the average annual water consumption per household in Sydney metropolitan areas during the study period (Q1-02 to Q4-09) declined from 282 kL/annum in 2002 to 200 KL/annum in 2009. Even without including the impact of rainwater tanks (which is discussed in the next section) the average water consumption fell by 24% over the study period Table 5.2 details the reduction in water savings in all 44 local government areas (LGAs). These reductions can be attributed to effective demand managing techniques such as the Sydney-wide water restrictions between 2003-2009 (Sydney Water, 2012a and Table 5.1) and the introduction of water efficient fixtures like taps, dual flush toilets, efficient shower heads, etc. SWC installed these free of charge or at subsidised prices. Other factors included the implementation of BASIX for new and refurbished buildings, and the installation of new water efficient water appliances. It may also be due to lot sizes becoming smaller in part due to the sub-division of existing residential lots which led to smaller gardens (Troy et al., 2005).

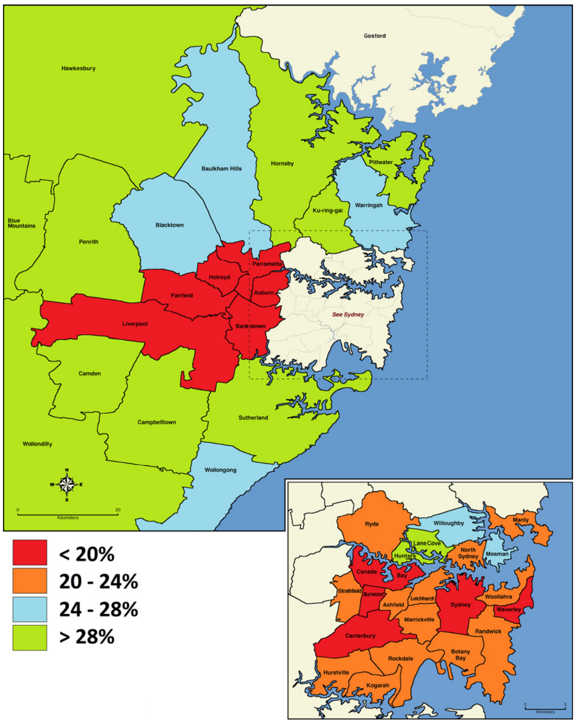
Table 5.2 Water savings by installing a rainwater tank in the Sydney Metropolitan Area

LGA	Average* Annual Water Usage +/- Standard Deviation (kL/Household)	Reduction in Water Consumption (%)	Further reduction due to Rainwater Tanks (%)	Percentage of rain water tanks installed (%)
Ashfield	233 +/- 24	22	12	3.7
Auburn	246 +/- 19	11	11	1.9
Bankstown	237 +/- 22	19	10	2.8
Blacktown	233 +/- 28	24	10	2.8
Blue Mountains	181 +/- 31	32	9	12.2
Botany Bay	254 +/- 24	22	6	15.2
Burwood	250 +/- 22	19	8	2.1
Camden	248 +/- 40	29	7	6.8
Campbelltown	242 +/- 34	29	9	3.6
Canada Bay	225 +/- 22	19	11	3.6
Canterbury	243 +/- 21	19	10	2.3
Fairfield	253 +/- 24	19	9	3.1
Hawkesbury	251 +/- 47	32	8	7.0
Holroyd	226 +/- 19	17	9	4.1
Hornsby	237 +/- 39	30	11	7.0
Hunters Hill	284 +/- 47	31	12	7.9
Hurstville	230 +/- 24	21	12	4.0
Kiama	172 +/- 21	25	10	15.7
Kogarah	244 +/- 27	22	10	4.2
Ku-ring gai	284 +/- 53	34	11	7.5
Lane Cove	249 +/- 37	30	12	5.6
Leichhardt	175 +/- 17	20	9	2.0
Liverpool	249 +/- 25	19	7	2.8
Manly	239 +/- 31	22	10	4.7
Marrickville	196 +/- 19	22	11	1.8
Mosman	284 +/- 38	26	16	5.3
North Sydney	215 +/- 27	24	12	3.2
Parramatta	230 +/- 25	19	11	3.6
Penrith	244 +/- 37	29	7	5.2
Pittwater	237 +/- 36	30	12	7.5
Randwick	242 +/- 26	24	10	3.5
Rockdale	242 +/- 22	24	10	2.0
Ryde	229 +/- 25	22	11	4.2
Shellharbour	206 +/- 22	24	7	17.0
Strathfield	279 +/- 35	21	7	3.0
Sutherland	242 +/- 16	29	9	7.4
Sydney	184 +/- 101	20	9	0.7
The Hills Shire	276 +/- 46	27	12	5.2
Warringah	237 +/- 30	24	11	5.5
Waverly	232 +/- 20	18	8	2.3
Willoughby	239 +/- 30	26	8	5.9
Wollondilly	233 +/- 40	29	9	12.0
Wollongong	196 +/- 22	25	8	16.4
Woollahra	280 +/- 35	23	9	2.3
Average	236 +/- 31	24	9 +/1 1.9%	5.5

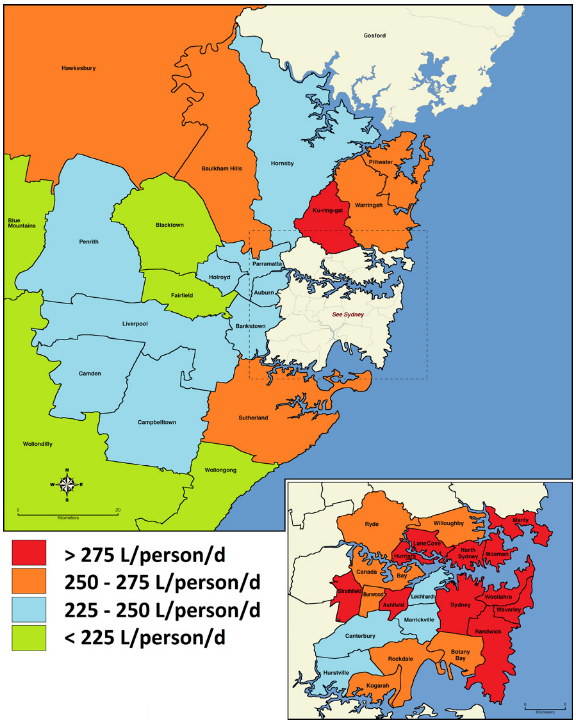
Note: Average of the period 2002 to 2009

When comparing the reduction in the level of water consumption between the various LGAs (Figure 5.2a) it is evident that water reductions are smaller for properties with smaller lot areas (Figure 5.3a). A majority of the LGAs within inner Sydney which have small lot areas had less than 20% reduction of water usage. Most of the LGAs in the outer Sydney area with some exceptions (Liverpool, Fairfield, Holroyd, Parramatta, Auburn and Blacktown), experienced significant reductions most of which were greater than 28%. This may be attributed to the significant reduction available from outdoor water usage.

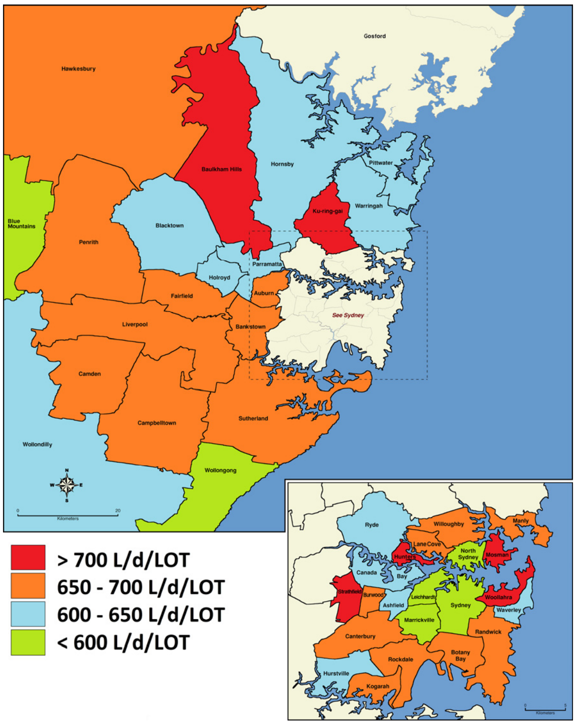
A relationship exists between water consumption in terms of per capita daily demand (L/p/d) compared to the number of people per household (ABS, 2005a; SWC, 2012b). Figure 5.3b shows that as the number of people per household increases, water consumption (in per capita terms) (Figure 5.2b) generally reduces. This is true for most LGAs within the inner Sydney area (except Leichardt and Marrickville) which have lower numbers of people per household and higher per capita usage compared to most LGAs in the outer Sydney area (except Pittwater, Baulkham Hills, Hawkesbury, Wollongong and Sutherland) which have higher number of people per household and lower per capita usage. The trend between water consumption is not completely explained by the house lot area and number of people per household (Figure 5.3c and Figure 5.3d). Socio-economic factors are explored in more detail below to assess how they affect water consumption.



(a) Reduction of water consumption in each LGA over the study period

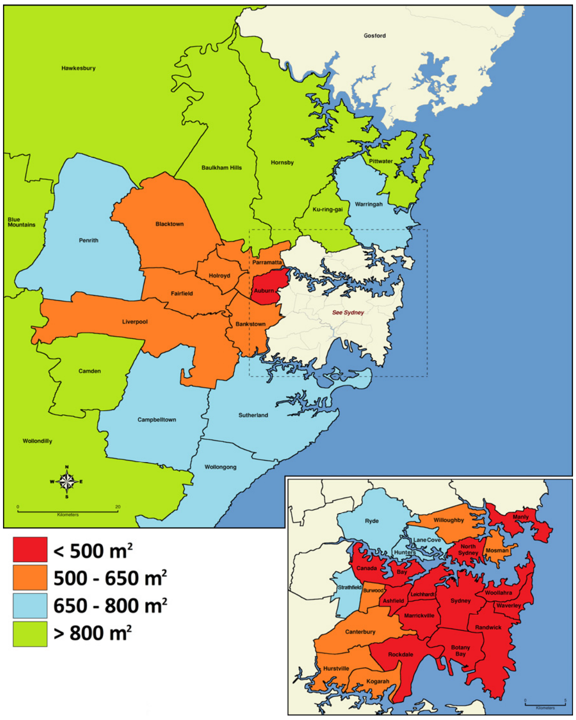


(b) Average daily per capita water usage

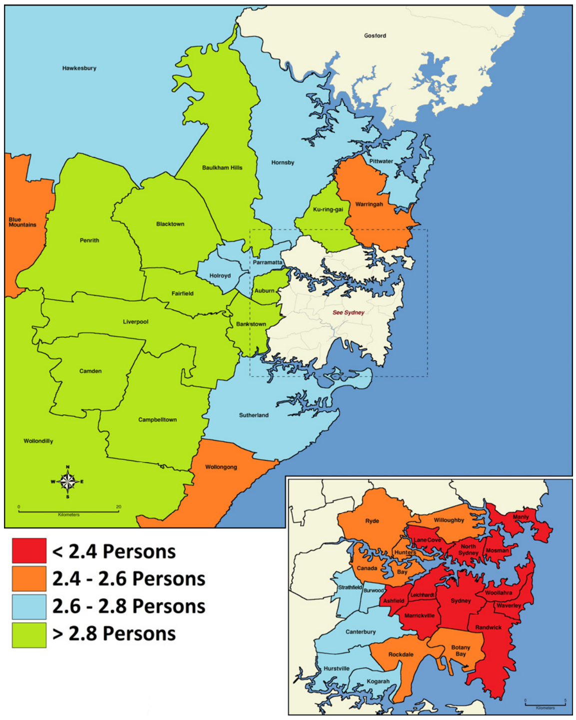


(c) Average daily household water usage

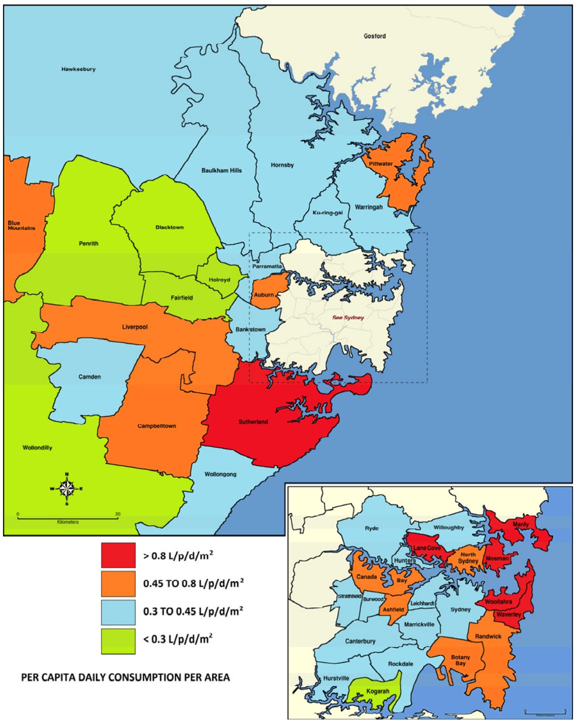
Figure 5.2 Water consumption for single residential houses in the Sydney metropolitan area



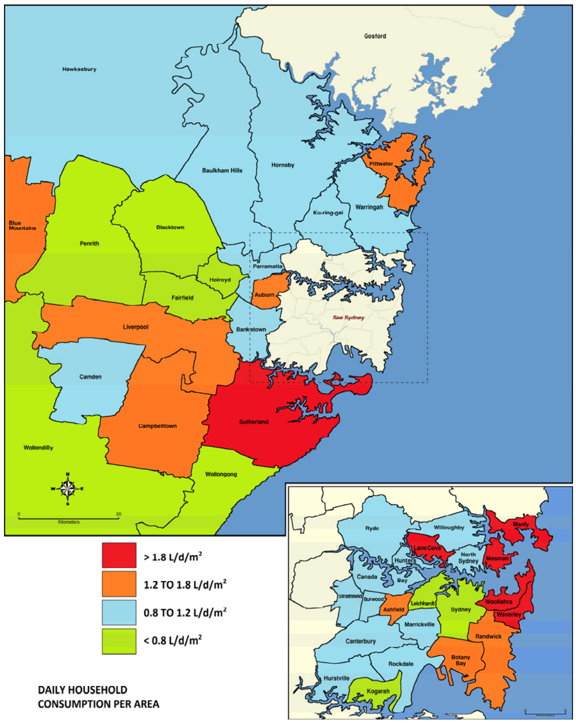
(a) Average lot sizes



(b) Average number of persons per household



(c) Average daily per capita water usage per square meter of lot area



(d) Average daily household water usage per square meter of lot area

Figure 5.3 Average area, number of persons and water usage for single residential houses in Sydney

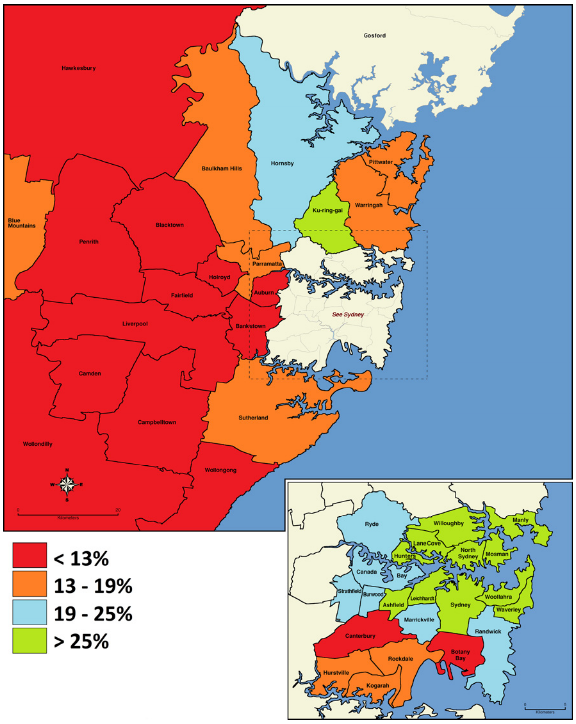
5.3.2. Socio-economic aspects

Figure 5.4(a-d) shows the distribution of socio-economic aspects such as levels of educational qualifications, mean taxable incomes, proportion of rental properties and proportion of residents born in non-English speaking countries (NESC).

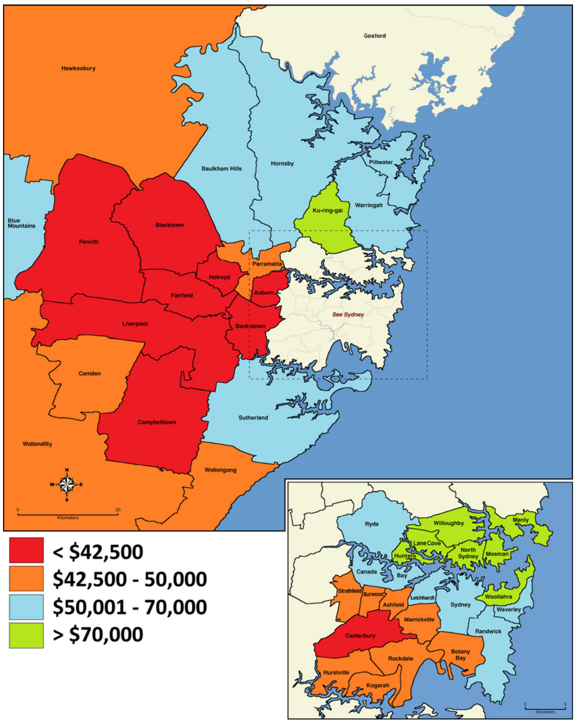
Educational qualification and mean taxable income: Troy et al. (2005) suggested that people with higher education qualifications and wealthier people have lifestyles associated with higher consumption including water usage. Figure 5.4a shows that education qualifications of people are much higher in the inner Sydney LGAs (ABS, 2007). More than 1 in 4 people that live in harbour side LGAs (Sydney, Woollahra, Waverley, Leichhardt, Ashfield, Hunters Hill, Lane Cove, North Sydney, Mosman, Willoughby and Manly) have a Bachelor degree or higher. This ratio is less than 1 in 7 in most of the LGAs west of the inner Sydney area. A higher portion of people with higher education qualifications in an LGA (Figure 5.4a) also corresponds with higher than average individual tax incomes (Figure 5.4b) (ABS, 2007). Further, it appears most of the LGAs with higher levels of educational qualifications (Figure 5.4a) generally had households with fewer occupants (Figure 5.3b). These suburbs generally had a higher per-capita water consumption pattern (Figure 5.2b). This is true for most LGAs in the Sydney metropolitan area except for Sutherland, Baulkham Hills and the Blue Mountains where the portion of people with higher educational qualifications is lower but they still have a high average of individual taxable income. On the other hand, Ashfield which has more than 1 in 4 people with a Bachelor degree or higher, has an average individual taxable income similar to those LGAs with people of lower levels of educational qualifications. This may be due to the high number of students and recently arrived migrants.

LGAs that are an exception to this trend are Leichhardt and Marrickville. Based on election returns, both LGAs contain a higher number of supporters of environment aligned political parties compared to other LGAs. They have high ratios of people with Bachelor degrees or higher qualifications. Leichhardt also has reasonably high income levels in-line with nearby suburbs (CBD Sydney, Randwick and Waverley). However, these LGAs have low water consumption levels in terms of daily household consumption and daily per-capita household consumption.

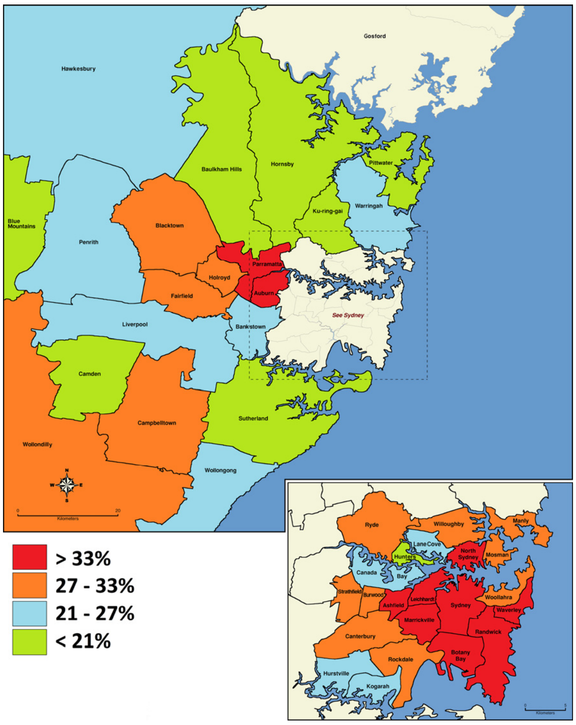
Rented properties: Figure 5.4c shows that in inner Sydney LGAs 1 in 3 properties were rented while in most of the remaining LGAs the ratio was at least 1 in 4 properties, (Figure 5.4c). This is very different to most outer Sydney LGAs where less than 1 in 4 properties were rented. There seems to be no relationship between per-capita daily household water consumption to the portion of rental properties in an LGA (Figure 5.4c and Figure 5.2b). Although some inner Sydney LGAs with higher numbers of rental properties have higher per-capita water consumption levels (Figure 5.2b), others with a high per-capita water consumption level have less rental properties.



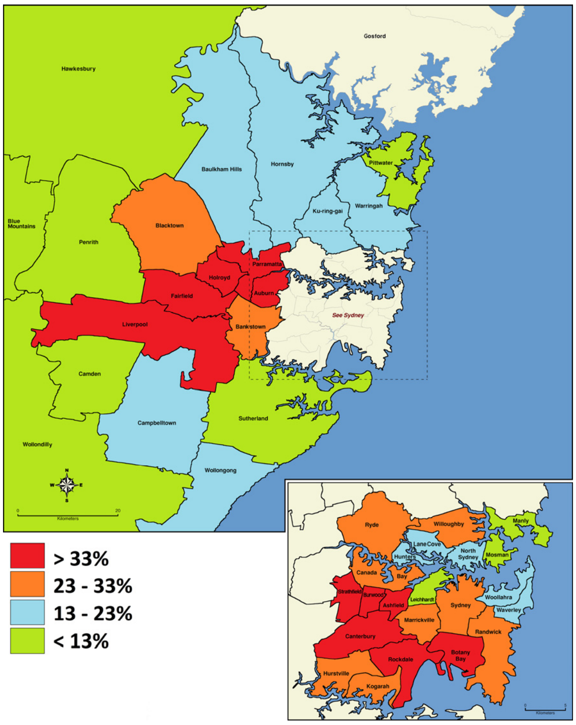
(a) Average number of people with Bachelor qualifications or higher



(b) Mean taxable income



(c) Percentage of houses that are not owner occupied (rented)



(d) Percentage of residents born in non-English speaking countries (NESC)

Figure 5.4 Socio-economic indicators in the Sydney metropolitan area

People born in non-English speaking countries (NESC): There appears to be a relationship between the level of water consumption between the various LGAs (Figure 5.2b) in the inner or outer Sydney areas with a high number of people born in NESC, (Figure 5.4d). LGAs that contained a higher proportion of people born in NESC were LGAs south of Sydney (Botany Bay and Rockdale) and LGAs west of Sydney (Canterbury, Ashfield, Burwood, Strathfield, Auburn, Parramatta, Holroyd, Fairfield and Liverpool) (Figure 5.4d). These areas are generally the same areas which have less people having at least a Bachelor degree and less than average individual tax incomes (Figure 5.4a and 5.4b). With the exception of Strathfield and Ashfield, these areas generally have average or below average levels of per-capita water consumption (Figure 5.2b). Figure 5.2c shows that Strathfield had a very high level of household consumption whereas Ashfield is slightly below average. The rest of the LGAs with higher numbers of people born in NESC generally had average levels of household consumption.

The LGAs with a higher portion of people born in NESC generally had less reductions in water consumption between 2001 and 2009 (Figure 5.2a). Other LGAs such as Campbelltown, Camden, Penrith and Hornby, which have much lower poritons of people born in NESC, had similar low average per-capita water consumption levels (Figure 5.2b) yet all had the biggest reductions for water consumption between 2001 and 2009 of more than 28% (Figure 5.2a). This could be due to the communication/education barriers faced by people born in NESC who did not fully benefit from the education programs in water restriction regulations and water saving incentives that were run when these programs were implemented.

5.3.3. Rainwater Tanks

Table 5.2 also provides the percentage of water savings by installing rainwater tanks. The distribution of reduction in water savings across all 44 LGAs fits a normal distribution with an average reduction of 9% +/- 1.9%. The statistical significances in the reduction of the mean water usage (P) is extremely small (1.9E-143) due to the very large data set of properties from the SWC database.

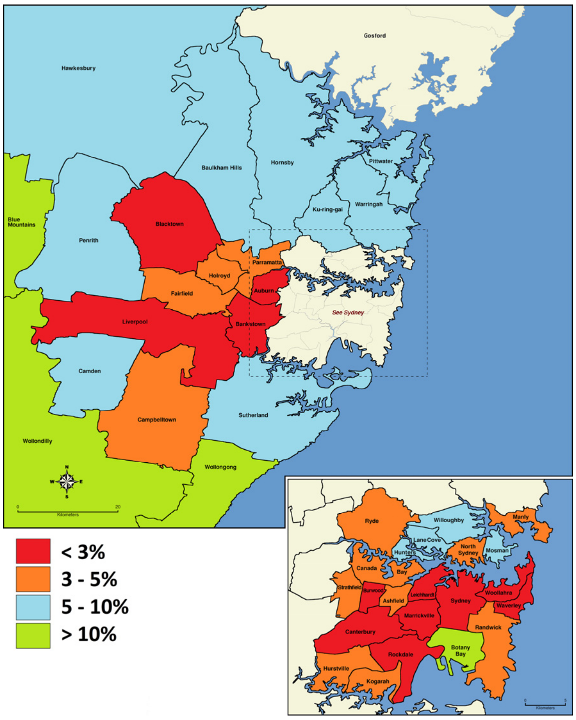
On average, a household could be able to save around 24 kL of water annually by installing a rainwater tank. This level of savings was compared with the findings of other studies undertaken on individual LGAs within the Sydney metropolitan and surrounding LGAs supplied water by SWC. Moy (2011) revealed that households that installed rainwater tanks in the Wollongong and Shellharbour LGAs reduced their water consumption by approximately 10.3% although this data was not adjusted for the overall reduction of water consumption that occurred in the wider community as households without rainwater tanks were also shown to reduce their water consumption over the same period by 10.8%. Further the reduction in water consumption was not individually reported for Wollongong LGA and Shellharbour LGA. Table 5.1 shows the reduction in water consumption due to rainwater tanks for Wollongong LGA and Shellharbour LGA was 7% and 8% respectively in this study. Knights et al (2012) presents data from the rainwater tank incentive scheme in Marrickville LGA which showed that rainwater tanks can reduce residential water consumption on average by 25%. Again this data was not adjusted for the overall reduction of water consumption that occurred in the wider community. A comparison of the methods of analysis between the studies is difficult without knowing the full extent of the datasets selected and whether specific conditions were placed on removing any data, (eg. as was carried out in this study, and outlined in the 'SWC Data Base' section of this paper).

Figure 5.5a shows the level of uptake of the rainwater tank rebate in LGAs, and demonstrates that the largest adopters were Wollongong, Wollondilly and the Blue Mountains in the outer Sydney area, and Botany Bay in the inner Sydney area, with more than 10% of houses having received a rainwater rebate. Other outer Sydney area LGAs with high levels (5-10%) were Sutherland, Camden and Penrith, along with all northern LGAs from the Hawkesbury through to Pittwater and the inner Sydney LGAs of Hunters Hill, Lane Cove, Willoughby and Mosman. The remainder of the inner Sydney areas was evenly split between lower uptakes of 3-5% or less than 3%. This could be due to the general lack of space for a rainwater tank in residential backyards in the inner Sydney locations (Figure 5.3a) and that a higher portion of the houses were rented compared to those in the outer Sydney LGAs (Figure 5.4c). LGAs located in the outer Sydney area with more people born in NESC, had lower levels of rainwater rebate uptake (between 3-5% or less than 3%).

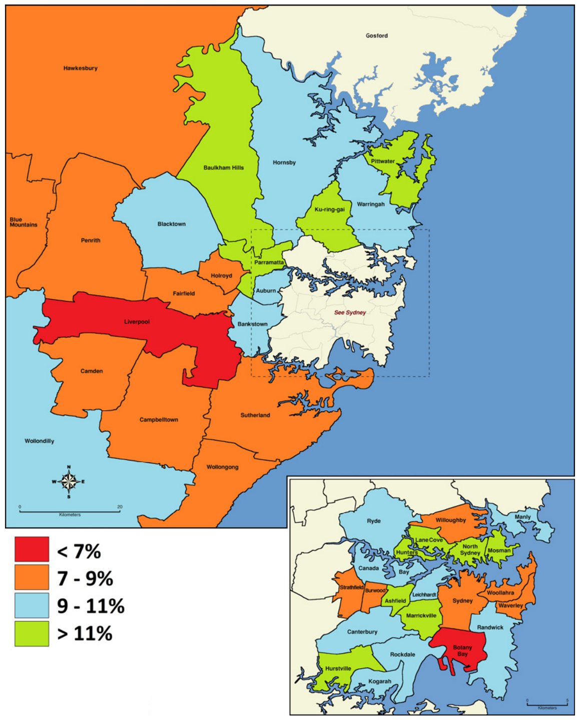
Figure 5.5b shows the level of reduction in water consumption that is attributed only to the installed rainwater tanks. The reduction in water consumption was calculated for rainwater tanks over a period of at least two years before and at least two years after the installation. The results indicate that most properties within inner Sydney with a rainwater tank achieved at least a 9-11% additional reduction in water usage, with more than half of those LGAs achieving more than 11% additional reductions. These same levels of water usage reductions were also observed for most of the northern and central LGAs in the outer Sydney area.

A reason for this large reduction in water consumption in the inner Sydney LGAs as compared to southern and western outer Sydney LGAs (Figure 5.5b), could be explained by the smaller lot areas (Figure 5.3a) in the former. It is estimated that the sizes of the rainwater tanks would not differ all too much in size in inner and outer Sydney LGAs, when compared to the difference in lot sizes. In inner Sydney LGAs, there would be a higher yield of collected rainwater relative to the area of garden. A rainwater tank in the inner Sydney LGAs would likely contain enough yield for most if not all outdoor watering requirements, and perhaps completely replacing the potable water requirement for outdoor/garden use. In outer Sydney LGAs, with much larger outdoor and/or garden areas, rainwater can only supplement potable water supplies rather than replacing it.

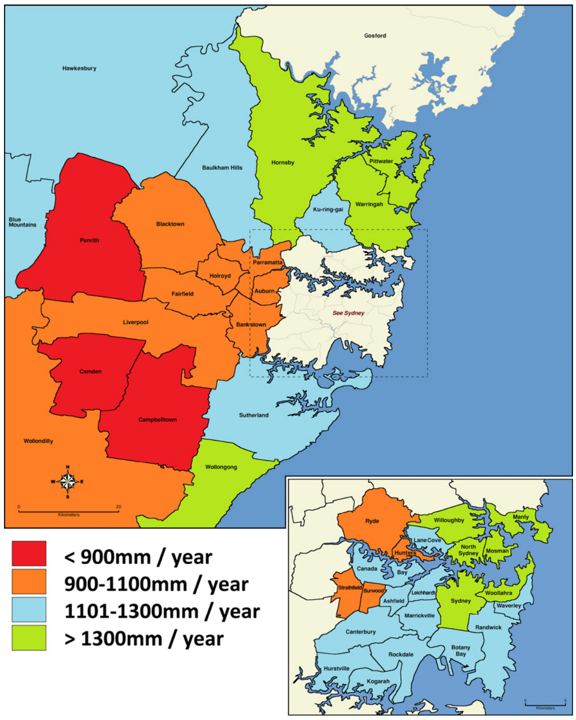
People born in NESC: Those central LGAs with the higher number of people born in NESC (Figure 5.4d) also have lower level reductions for water consumption from 2001 to 2009 and generally a lower uptake of the rainwater rebate. They actually achieved high water usage reduction following the installation of the rainwater tanks (Figure 5.5b). This could be due to the fact that the smaller number of people who received the grant in these LGAs were strongly motivated and had a high potential for saving water for gardening and other outdoor requirements.



(a) Percentage of rebated rainwater tanks installed



(b) Average percentage of water savings from rebated rainwater tanks



(c) Sydney Metropolitan area long term average rainfall by LGA from 1961 to 1990

Figure 5.5 Water savings from rebated rainwater tanks in the Sydney metropolitan area

Lot size: An analysis of the average lot size (Figure 5.3a) of properties that received rainwater tank rebates shows a bias to large properties. This is likely due to people with larger properties having larger gardens and being able to better utilise and warrant a rainwater tank.

Influence of rainfall: The typical rainfall patterns show that the coastal areas and the elevated areas of the Sydney Basin (Blue Mountains, Hawkesbury and Baulkham Hills) generally have higher levels of rainfall compared to other LGAs located away from the coastline. There seems to be some relationship between the average total annual rainfall (Figure 5.5c) for each LGA compared to the level of water usage reductions from installing a rainwater tank (Figure 5.5b). A few of the LGAs (Sydney, Marrickville, North Sydney, Mosman, Baulkham Hills and Hornsby) that had high levels of rainfall resulted in higher levels of water usage reductions. Western and south-western LGAs of outer Sydney area (from Parramatta to Penrith down to Wollondilly) that received lower levels of rainfall, achieved lower levels of water usage reductions. While this is true for some LGAs, there are also a significant number of LGAs (Botany Bay, Waverley, Woollahra, Leichhardt, Lane Cove, and Willoughby) who received high levels of rainfall but did not achieve large reductions in water usage reductions and vice versa (Ryde, Strathfield and Burwood).

5.4. Conclusions

The SWC data shows that the average annual water consumption per household in Sydney's metropolitan areas during 2002 to 2009 declined from 282 kL/annum to 200 kL/annum. Even without including the impact of rainwater tanks the average water consumption fell by 24% over the study period. In many localities (LGAs) the reduction in water consumption was over 28% and up to 33.5%. These reductions were due to the effective 'demand managing' techniques such as the Sydney-wide water restrictions and the introduction of water efficient fixtures like taps, dual flush toilets, and efficient shower heads.

With respect to Socio-economic aspects, a higher ratio of people with higher education qualifications in a LGA typically corresponded with higher average individual tax incomes and generally had higher per-capita water consumption patterns. People with higher education qualifications and wealthier people typically have a lifestyle associated with higher consumption including water usage (Troy et al., 2005).

LGAs with high percentages of People born in NESC had lower reductions for water consumption and generally a lower uptake of the rainwater rebate. LGAs with lower ratios of people born in NESC, had similar low average per-capita water consumption levels yet all had the biggest reductions for water consumption. This could be due to the communication/education barriers faced by people born in NESC who did not fully benefit from the education programs in water restriction regulations and water saving incentives being run at the time.

There seems to be no relationship between per-capita daily household water consumption to the portion of rental properties in an LGA.

There also seems to be only a partial relationship between the average total annual rainfall for each LGA compared to the level of water usage reductions from installing a rainwater tank. Some of the LGAs received high levels of rainfall resulting in higher levels of water usage reductions though this was contradicted with a significant number of LGAs receiving high levels of rainfall and not showing any significant reductions in water usage.

The average percentage of water savings by installing rainwater tanks across all 44 LGAs witnessed a further reduction of 9%. In some LGAs in Sydney the reduction in water consumption due to rainwater tanks was up to 15%. On average, a household was able to save around 24 kL of water annually by installing a rainwater tank excluding the effect of other factors that affected water usage.

Chapter 6

University of Technology, Sydney

Faculty of Engineering & Information Technology

6. Granular Activated Carbon (GAC) Filter for Rainwater Treatment

6.1. Introduction

This study investigated how well a GAC adsorption filter performed in the long-term as a pre-treatment strategy concerning membrane filtration for treating raw rainwater.

From 22nd to 24th September 2009, an extreme dust storm swept across the Australian states of New South Wales and Queensland and was described as the worst in New South Wales in nearly 70 years. The dust originated from the far west of New South Wales and north-east South Australia, principally dry remote areas. The storm was measured at more than 500 km wide and 1000 km in length and estimated to have carried 16 million tonnes of dust. At the storm's peak it was estimated that Australia had lost 75,000 tonnes of dust per hour off the north coast, north of Sydney.

During the dust storm, thousands of tonnes of dirt and soil were carried through Sydney. The day after the dust storm had passed through, a rainfall event occurred over the local catchment in which the rainwater tank (T1) of this experiment was located. GAC adoption experiments were conducted using this rain water.

6.2. Granular Activated Carbon (GAC) adsorption filter – first experiment

A filtration experiment was conducted with an acrylic column packed with GAC. The experimental set up was described in Chapter 3 (Section 3.3.2 and Figure 3.20).

The rainfall event that occurred over the local catchment, in which the rainwater tank (T1) of this experiment was located, washed the increased concentration of dust in the atmosphere from the storm of the 22nd to 24th September 2009 and the dust that settled on the roof into the rainwater tank. This rainwater tank (T1) contained the raw rainwater used to supply the laboratory feed tank. A GAC adsorption experiment was conducted immediately after the rain and dust storm event, first using existing rainwater in the laboratory feed tank which was then topped up by the dust-contaminated rainwater initially on day 6 with subsequent incremental top-ups on days 22 and 40.

The concentrations of various water quality parameters in the influent and effluent of the GAC adsorption filter were monitored for 50 days. However, after this the feed pump malfunctioned and the experiment had to be subsequently stopped. The experiment was repeated and the results for the second experiment are reported in Section 6.3.

Total organic carbon

The TOC concentration in the influent and effluent of the GAC adsorption filter was monitored for 50 days. In Figures 6.1 and 6.2, during the initial 32 days of the monitoring there was a steady decline in TOC removal from 79% down to 18%. On day 25 the flow rate through the column declined to less than the desired 1m/hr and so a backwash cycle was initiated. The backwashing was conducted by using tap water in an upward flow direction from the bottom of the column. During the backwashing process, the overflow pipe was rerouted from the feed tank to a waste tank. The GAC adsorption bed was expanded up to approximately 30% for 2 minutes. Backwashed contaminants were removed from the column through the overflow pipe along with the tap water. Following the backwash cycle, there was a short improvement in TOC removal for 4 days before the filter reverted back to the original steady decline until day 32.

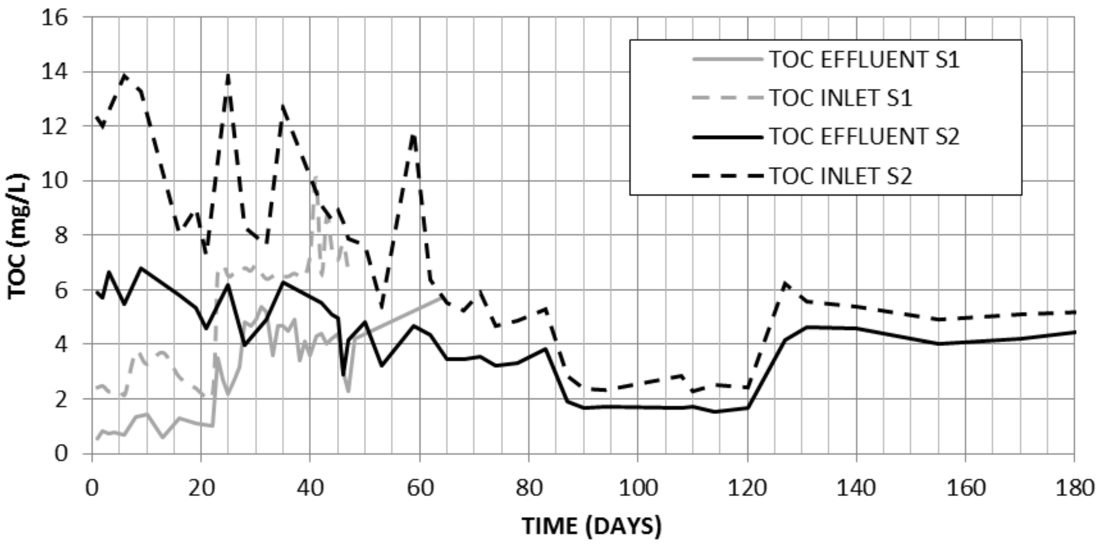


Figure 6.1 TOC influent and effluent of GAC adsorption filter experiments. S1 is data from the first experiment, S2 is from the second experiment. Backwash carried out on Day 25 (for S1) and Days 35, 60, 100, and 140 (for S2).

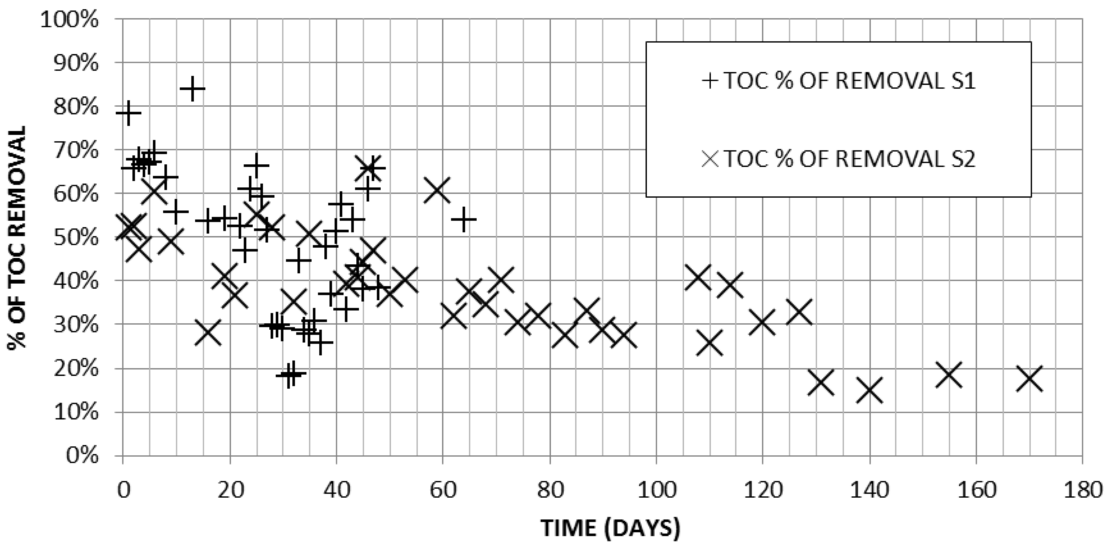


Figure 6.2 Graph combining TOC removal rate from both GAC adsorption filter experiments. S1 is data from the first experiment; S2 is from the second experiment. Backwash carried out on Day 25 (for S1) and Days 35, 60, 100, and 140 (for S2).

Over the next 16 days the TOC removal rate of the GAC adsorption filter started to increase in an oscillating fashion with a general rise from 25% to 50%. The oscillating pattern in TOC removal rate is typically seen in biofilters. It should be also noted that the microbial colonies are affected by the concentration of influent TOC which in raw rainwater can be variable. After refilling the feed tanks on day 22, the influent TOC level increased from an average of 2.6 mg/L to an average of 7.0 mg/L for the rest of the experiment (Figure 6.1). This increase in the influent TOC could explain the possible formation of microbial colonies, which may not have been able to establish at lower concentrations of influent TOC. This resulted in an improved TOC removal rate. Furthermore detailed analysis of the parameters was carried out for the first 21 days of GAC adsorption filter's operation (Table 6.1).

Turbidity, true colour and suspended solids

The GAC adsorption filter performed effectively in removing turbidity, TSS and true colour as shown in Table 6.1 and Figures 6.3 to 6.5.

Although the removal rate of turbidity ranged between 44-97% over the first 21 days, the influent concentrations were rather low , ranging between 0.5-4 NTU which was below the ADWG (2011) limit (5 NTU) (Figure 6.3). The feed water contaminated by the dust storm was added to the feed tank on day 6. The subsequent sample taken on day 7 contained a higher level of turbidity. Although the influent level increased substantially from less than 1 NTU up to 4 NTU over this period, the effluent concentration still remained at 1 NTU or less. The turbidity declined in subsequent days (but remained higher than pre-dust storm values) due to settling in the laboratory feed tank although this effect is more evident with TSS (Figure 6.4).

The feed water contaminated by the dust storm caused an increase in the concentration of TSS from less than 1 mg/L to 5 mg/L on day 7. The concentration of TSS in the effluent during this period fell to below the detection limit (0.5 mg/L). The removal of TSS was good and an average removal rate of 53% (Table 6.1; Figure 6.4) was recorded.

The feed water contaminated by the dust storm caused increased levels of true colour on day 7 and remained at elevated levels until day 15. On day 7 and 9 (when the influent concentrations were elevated due to the dust storm) the removal rate of true colour was an average of 68% with the average effluent value being 4 PtCo which is nearly colourless (Figure 6.5, Table 6.1). The removal rate of true colour removal was not as effective as that for turbidity and TSS although it was still over 50%.

Heavy metal analysis

The influent concentration of lead ranged between 0.006 mg/L down to detectable limits of 0.001 mg/L and all samples complied with the ADWG (2011) limit (0.01 mg/L) (Figure 6.6). Following the GAC adsorption, the removal rate of lead started at a high of 58.4% and then slowly decreased to 20% removal after 21 days (Figure 6.6). Despite this it should be noted that the performance of the GAC adsorption filter managed to reduce most of the effluent samples to detectible limits of 0.001 mg/L. The feed water contaminated by the dust storm event was added to the feed tank on day 6. Subsequent samples taken on days 7 and 9 contained a higher concentration of lead. During this period the performance of the GAC adsorption filter in terms of lead removal rates improved, followed by a return to lower removal rates once the influent concentrations declined in subsequent days when the influent concentration reduced.

The influent concentration of iron ranged between 0.062-0.026 mg/L with the concentration in all samples complying with the ADWG (2011) limit (0.3 mg/L) (Figure 6.7 and Table 6.1). The removal rate of iron with GAC adsorption achieved an average of 45%. The feed water contaminated by the dust storm did not show any noticeable elevation in iron concentration which was more or less steady at 0.05 mg/L. The concentration of iron in all effluent samples was less than 0.3 mg/L which complied with the ADWG (2011) limit.

The influent concentration of manganese in samples ranged between 0.013-0.001 mg/L and was below the ADWG (2011) limit (0.1 mg/L) (Figure 6.8). The average removal rate of manganese with GAC adsorption was 20% (Table 6.1). The concentration of manganese in all effluent samples, however, was less than 0.1 mg/L which complied with the ADWG (2011) limit. The feed water contaminated by the dust storm feed water did not show any noticeable increase in the influent manganese concentration.

The removal rate of zinc varied from an initial high of 95.2% down to 30% after 21 days (Figure 6.9 and Table 6.1).The average removal rate of zinc was approximately 70%. The concentration of zinc in all influent samples complied with the ADWG (2011) limit (3 mg/L). The feed water contaminated by the dust storm did not demonstrate any noticeable effects.

The influent concentration of aluminium ranged between 0.016-0.029 mg/L which complied with the ADWG (2011) limit (0.2 mg/L) (Table 6.1). The GAC adsorption pre-treatment did not improve upon the influent water samples leading to an average concentration of 0.029 mg/L, which was higher than the influent concentration. This could be due to the initial removal by the GAC filter and its subsequent leaching out. The concentration of aluminium in all effluent samples, however, was less than 0.2

mg/L, below the ADWG (2011) limit. There were no traces of copper, arsenic or cadmium detected in the influent rainwater samples.

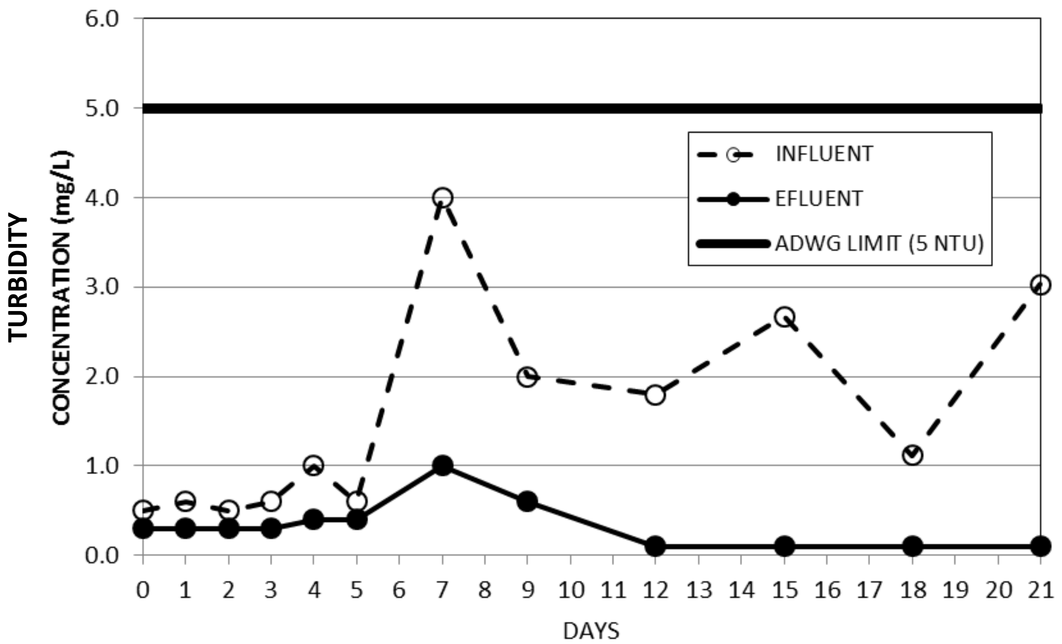


Figure 6.3 Graph of **turbidity** removal with GAC adsorption filter

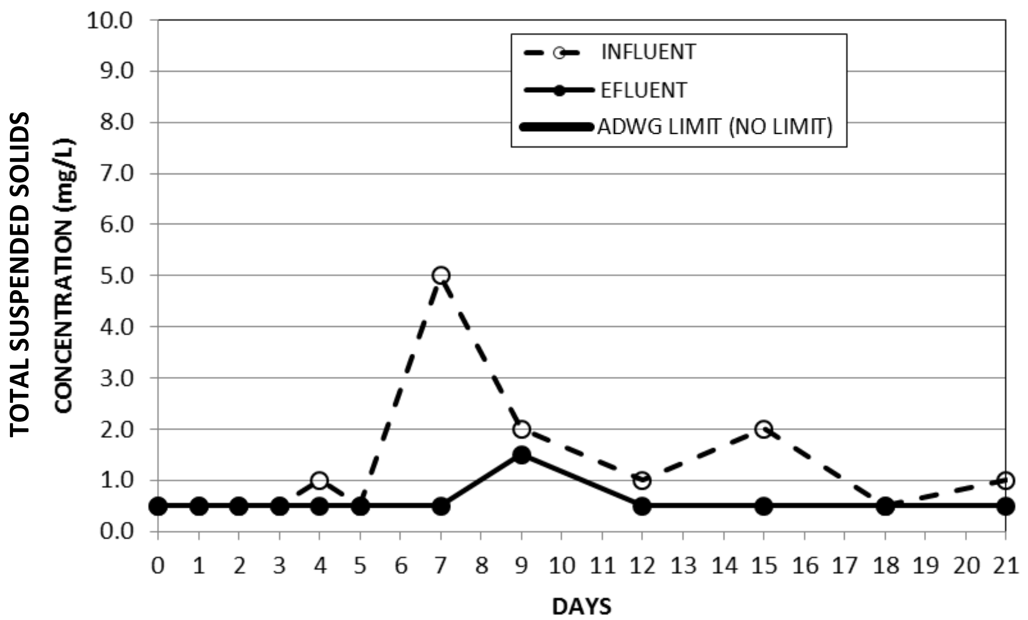


Figure 6.4 Graph of **total suspended solids** removal with GAC adsorption filter

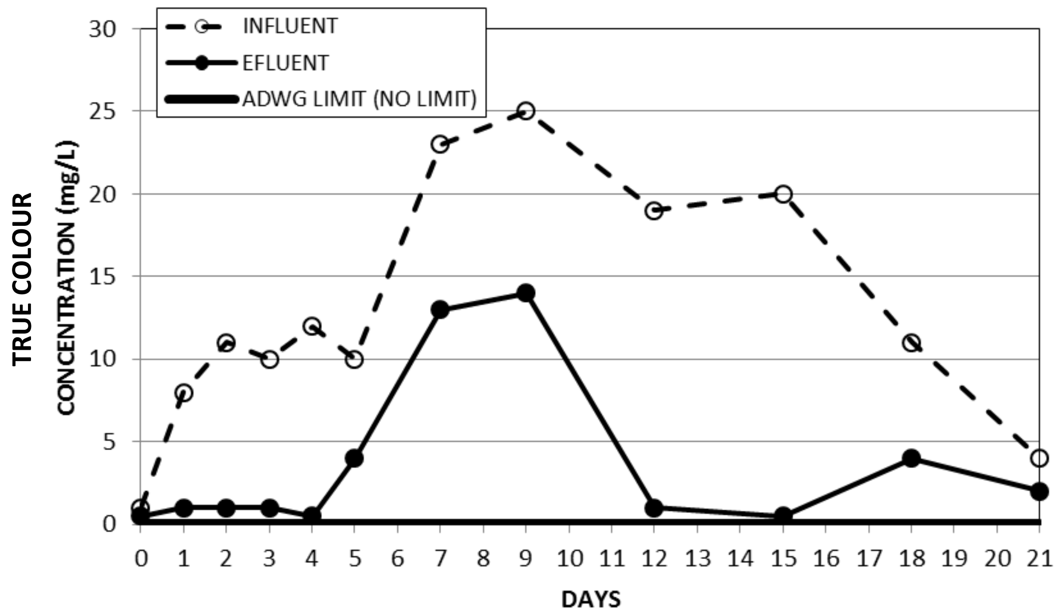


Figure 6.5 Graph of **true colour** removal with GAC adsorption filter

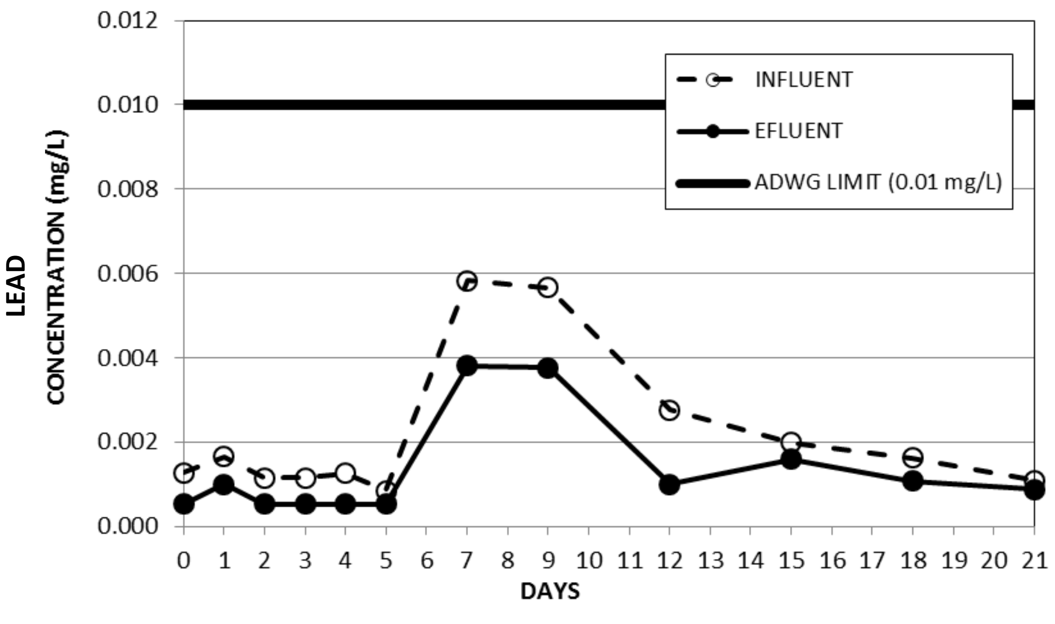


Figure 6.6 Graph of **lead** removal with GAC adsorption filter

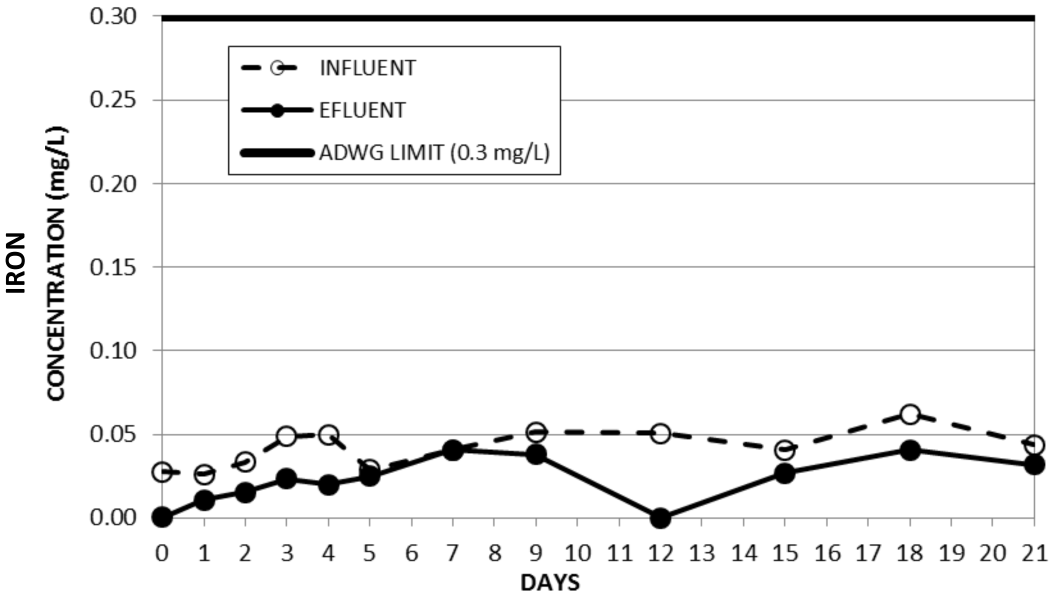


Figure 6.7 Graph of **iron** removal with GAC adsorption filter

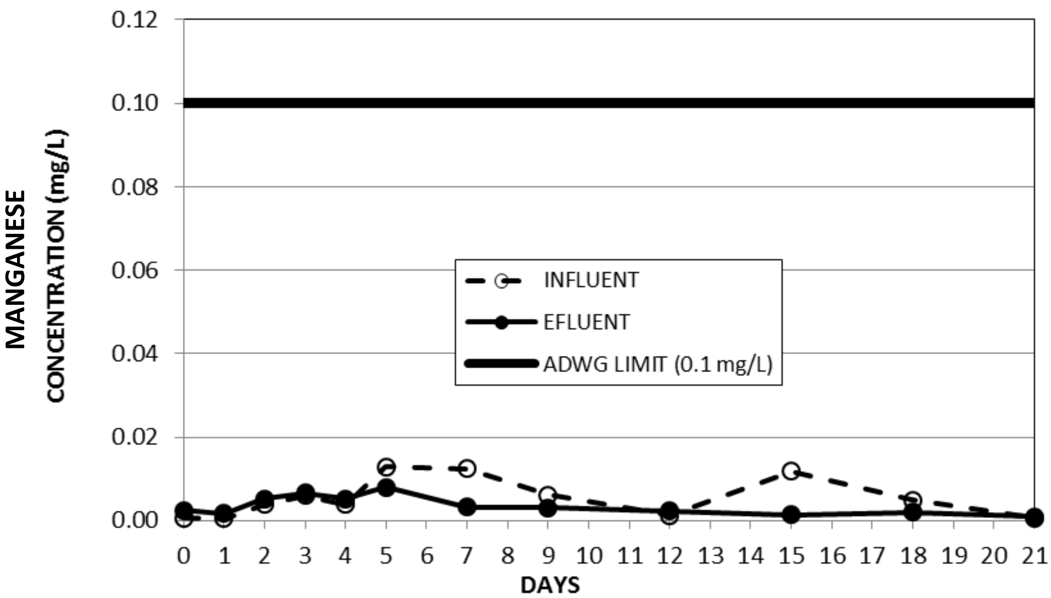


Figure 6.8 Graph of **manganese** removal with GAC adsorption filter

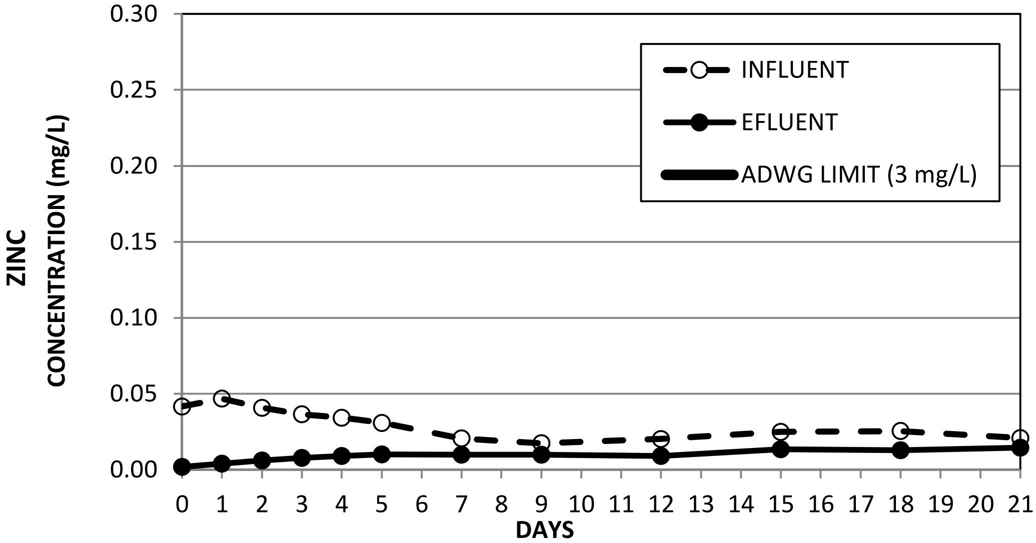


Figure 6.9 Graph of **zinc** removal with GAC adsorption filter

Anions and cations, total dissolved salts, water hardness and pH analysis

The GAC adsorption filter had a small to negligible effect in removing anions, cations, or in reducing the total dissolved salts and water hardness. There was no quantifiable change in pH (Table 6.1).

Orthophosphate, nitrate, nitrite and ammonia

Generally the effectiveness of the GAC adsorption filter for these parameters was negligible. The average concentration of orthophosphate, nitrate, nitrite and ammonia in the influent (Table 6.1) were 0.016, 0.612, 0.009 and 0.088 mg/L, respectively, while in the effluent they were 0.019, 0.059 0.004 and 0.092 mg/L (Table 6.1). Although these parameters, with the exception of nitrite, did not benefit from GAC adsorption filtration, all influent and effluent samples were below the ADWG (2011) limit.

Total coliforms and faecal coliforms

The total and faecal coliform counts indicated that the GAC adsorption filter caused some reduction in these parameters (Table 6.1). The total and faecal coliforms counts of the influent and effluent samples did not comply with the ADWG (2011) recommendation and for this reason further treatment is required.

Table 6.1 Laboratory analysis of GAC adsorption filter - first experiment

Day / Parameter	ADWG (2011)	Detectable limit	Days 0 - 21			
			# Samples	Influent	Effluent	% Removal
pH	6.5 - 8.5	3.0 - 10	11	7.0 6.8 - 7.2	7.0 6.87 - 7.13	NQ
Total Suspended Solids (mg/L)	-	0.5	11	1.3 0.5 – 5.0	0.6 0.5 – 1.5	53%
Turbidity (NTU)	<5	0.2	11	1.5 0.5 – 4.0	0.3 0.1 – 1.0	80%
True Colour (PtCo)	NA	1	11	12.8 1 – 25	4 1 – 14	68%
Total Dissolved Salts (mg/L)	NA	0.1	11	54 44 - 67	54 46 - 61	NQ
Water hardness (mg/L $CaCO_3$)	200	0.1	11	19 14 - 36	19 12 - 35	NQ
Sodium (mg/L)	NA	0.05	11	9.9 2.3 - 15.9	9.5 2.4 - 13.9	4%
Potassium (mg/L)	NA	0.001	11	0.6 0.4 - 1.0	0.7 0.4 - 1.0	NQ
Calcium (mg/L)	NA	0.05	11	3.9 1.0 - 13.3	3.7 0.9 - 12.9	5%
Magnesium (mg/L)	NA	0.05	11	2.4 0.5 - 3.3	2.5 0.5 - 4.4	NQ

Notes:
- NQ – not quantified, this parameter is not reduced by adsorption
- 1^{st} row – Mean average value, 2^{nd} row -range of value

Table 6.1 Laboratory analysis of GAC adsorption filter-first experiment (cont'd.)

Day / Parameter	ADWG (2011)	Detectable limit	Days 0 - 21 # Samples	Influent	Effluent	% Removal
Orthophosphate (mg/L)	NA	0.005	11	0.016 0.004 – 0.029	0.019 0.011 – 0.029	NQ
Nitrate (mg/L)	<50	0.005	11	0.612 0.110 – 1.022	0.590 0.183 – 1.056	3.6%
Nitrite (mg/L)	<3	0.005	11	0.009 0.002 – 0.038	0.004 0.001 – 0.017	55%
Ammonia (mg/L)	<0.5	0.005	11	0.088 0.015 – 0.339	0.092 0.006 – 0.299	NQ
Total Coliforms (cfu/100mL)	< 1	1	11	**186** **20 – 1000**	**94** 1 – **310**	49%
Faecal Coliforms (cfu/100mL)	< 1	1	11	**63** 1 – **250**	**40** 1 – **200**	36%
Aluminium (mg/L)	<0.2	0.001	11	0.022 0.016 – 0.029	0.029 0.009 – 0.048	NQ
Iron (mg/L)	<0.3	0.005	11	0.042 0.026 – 0.062	0.023 0.005 – 0.041	45%
Manganese (mg/L)	<0.1	0.001	11	0.005 0.001 – 0.013	0.004 0.001 – 0.008	20%
Lead (mg/L)	<0.01	0.001	11	0.002 0.001 – 0.006	0.001 0.001 – 0.004	50%
Zinc (mg/L)	<3	0.001	11	0.030 0.018 – 0.047	0.009 0.002 – 0.015	70%

Notes:

- NQ – not quantified, this parameter is not reduced by adsorption
- Arsenic, Copper and Cadmium were not detected
- 1st row – Mean average value, 2nd row -range of value
- Values exceeding the ADWG (2011) limit are shown in bold

6.3. GAC adsorption filter – second experiment

As the first GAC adsorption experiment ended prematurely at day 50 due to the failure of the feed pump, an experiment was repeated for 180 days with the laboratory feed tank being refilled on days 24, 55, 86, 105, 125, 145 and 165. It should be noted that the same rainwater tank (T1) affected by the dust storm was utilised for this second experiment. The dust particles had settled in the tank as minimal rain had fallen since the dust storm and the rain event that followed soon after. However, large intense storm events occurred just prior to the top-up of the feed tank on days 55, 86 and 125 which re-suspended some of the accumulated sediments from the base of the rainwater tank. The experimental set-up was discussed in Chapter 3 (Section 3.3.2 and Figure 3.20).

Total organic carbon

The GAC adsorption filter was monitored for a period of 180 days (Figures 6.1 and 6.2). On days 35, 60, 100, and 140 the flow rate of the column declined to less than the desired 1 m/hr and so backwash cycles were initiated. The backwashing was conducted by using tap water in an upward flow direction from the bottom of the column. During the backwashing process, the overflow pipe was rerouted from the feed tank to a waste tank. The GAC adsorption bed was expanded up to approximately 30% for 2 minutes. Backwashed contaminants were removed from the column through the overflow pipe along with the tap water.

In Figure 6.2, during the 180 days of monitoring, a steady decline in TOC removal from 50-80% to 20% was observed. A higher removal rate could be due to a higher influent TOC. After day 60 the influent TOC dropped below 4 mg/L (Figure 6.1).

Turbidity, suspended solids and true colour

Turbidity sampling was carried out during the first 90 days of the GAC adsorption filtration. It emerged that the filter performed effectively in removing turbidity (Figure 6.10 and Table 6.2). The influent concentration of turbidity ranged between 2.0-6.0 NTU. The turbidity started at a low of 2.0 NTU and there was little removal. The top-up water added to the feed tank on days 55 and 86 contained re-suspended sediments from the rainwater tank's base following storm events, leading to an increase in influent turbidity and TSS levels. Although the influent turbidity doubled on day 60 from 3 NTU to 6 NTU and remained high until day 90, the effluent only increased from 2 NTU to 3 NTU. Table 6.2 indicates an average 42% removal during the course of the experiment. The concentration of turbidity in the effluent samples was always below the ADWG (2011) limit.

The influent concentration of all TSS influent samples were less than 5.5 mg/L (Figure 6.11). The average removal rate following GAC filtration was 50% with the TSS concentration of all effluent samples being less than 3.5 mg/L (Figure 6.11 and Table 6.2). The average influent concentration of total colour was 244 PtCo, which was higher than the first experiment (Tables 6.1 and 6.2). The GAC adsorption filter achieved an average total colour removal of 26% (Table 6.2). The ADWG (2011) does not state a limit for true colour.

Heavy metal

Detailed analysis was also carried out on the first 90 days for the GAC adsorption filter (Table 6.2). The influent contained lead concentration (Figure 6.12) ranging between 0.019-0.025 mg/L with all samples exceeding the ADWG (2011) limit (0.010 mg/L).

The removal rate of lead occurred at an average rate of 59% with a maximum removal rate of 63%. The concentration of lead in all effluent samples complied with the ADWG (2011) limit with the exception of one sample at 0.011 mg/L.

The influent concentration of iron ranged between 0.427-0.225 mg/L with 3 of 7 samples not complying with the ADWG (2011) limit (0.3 mg/L) (Figure 6.13). The top-up of the feed tank on day 55 with rainwater containing re-suspended sediments caused a noticeable increase in iron influent levels from day 60. The removal rate of iron using the GAC adsorption filtration treatment was marginal at 9%. The concentration of iron in 5 of the 7 effluent samples was less than 0.3 mg/L as prescribed in the ADWG (2011) limit.

The influent concentration of manganese ranged from 0.140-0.040 mg/L with most samples not complying with the ADWG (2011) limit (0.1 mg/L) (Figure 6.14). These influent concentrations were quite high and the GAC filtration removal rate was better than the first experiment which evidenced low influent concentration. The average removal rate following GAC filtration was 46%. However, the concentration of manganese in all effluent samples was less than 0.1 mg/L which complied with the ADWG (2011) limit.

The influent concentrations of zinc ranged from 0.070-0.038 mg/L and all complied with the ADWG (2011) limit (3 mg/L) (Figure 6.15). Following the GAC filtration process, on average a further 45% of zinc was removed.

The influent concentration of aluminium ranged from 0.090 mg/L down to 0.069 mg/L and all samples complied with the ADWG (2011) limit (0.2 mg/L) (Table 6.2). The

GAC filtration removal rate of aluminium was negligible resulting in the effluent concentration of all samples still complying with the ADWG (2011).

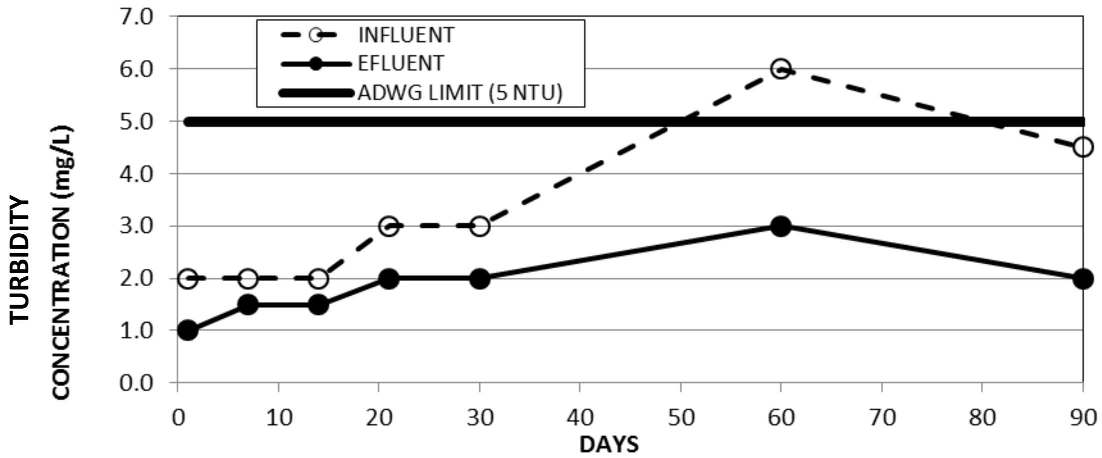


Figure 6.10 Graph of **turbidity** removal with GAC adsorption filter

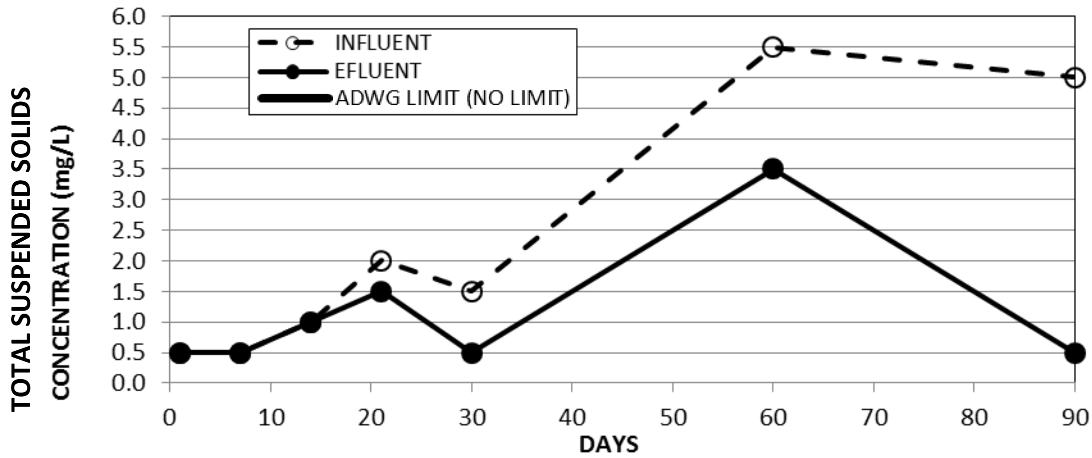


Figure 6.11 Graph of **total suspended solids** removal with GAC adsorption filter.

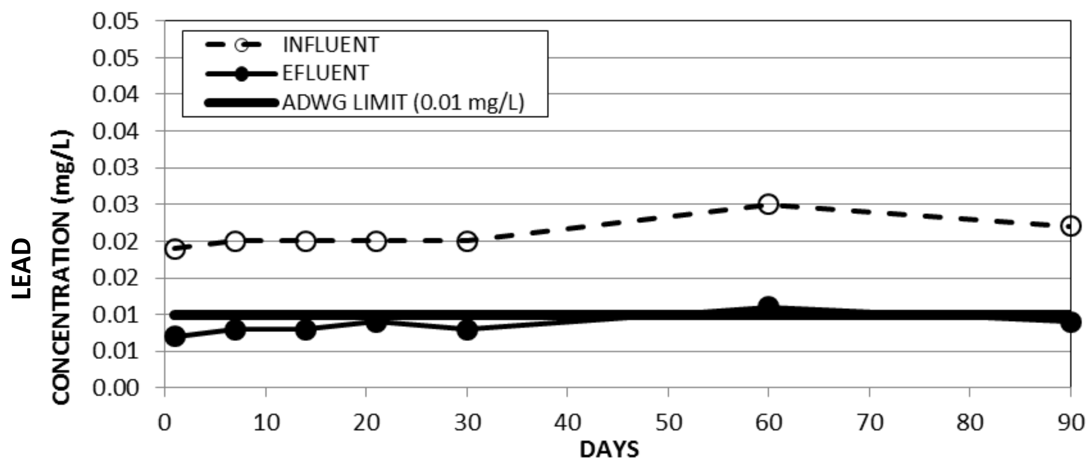


Figure 6.12 Graph of **lead** removal with GAC adsorption filter

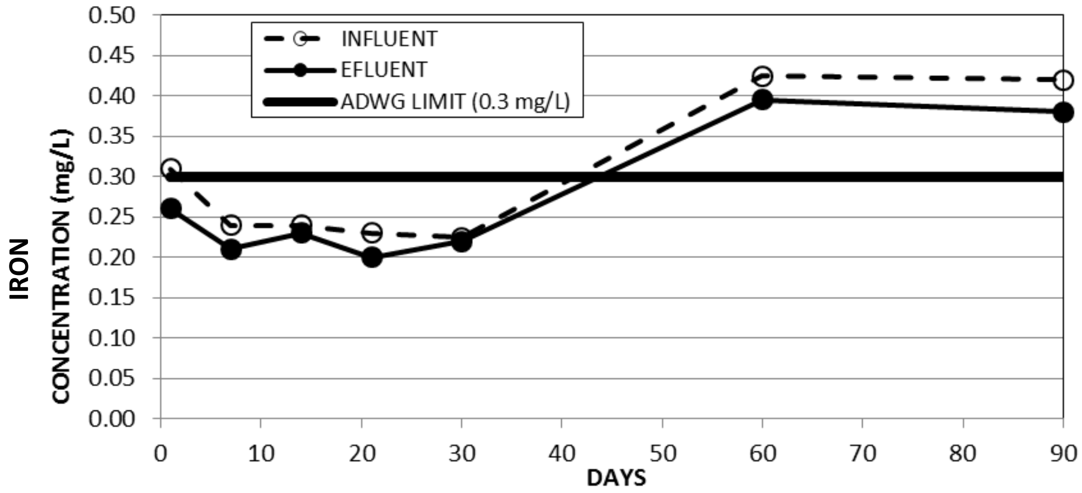


Figure 6.13 Graph of **iron** removal with GAC adsorption filter

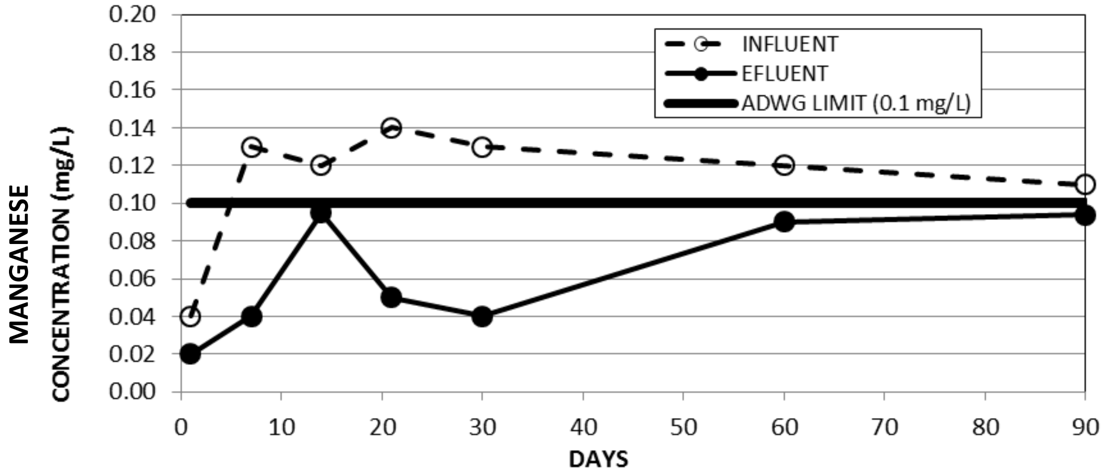


Figure 6.14 Graph of **manganese** removal with GAC adsorption filter

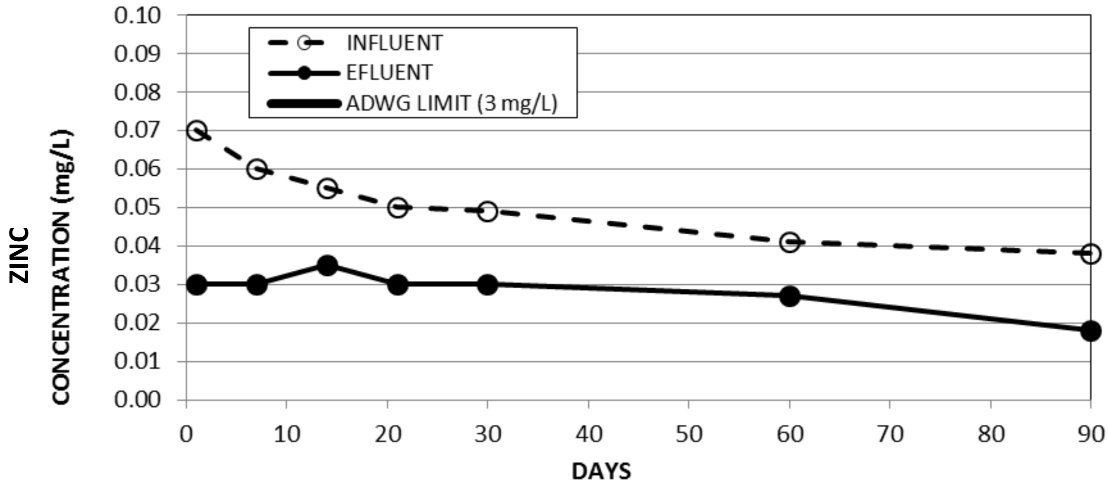


Figure 6.15 Graph of **zinc** removal with GAC adsorption filter

Anions and cations, total dissolved salts, water hardness and pH

The GAC adsorption filter had a small effect in removing anions, cations, or reducing the total dissolved salts and water hardness. The removal rate of total dissolved salts, sodium, potassium and magnesium was less than 10% (Table 6.2). The removal rate of water hardness and calcium is 15% and 16%, respectively. There was no quantifiable change in pH (Table 6.2).

Orthophosphate, nitrate, nitrite and ammonia

Orthophosphate, nitrate, nitrite and ammonia were all detected in the influent and effluent samples (Table 6.2). The effectiveness of the GAC adsorption filtration on these parameters is generally thought to be small. The average concentrations of orthophosphate, nitrate, nitrite and ammonia in the influent were 0.120, 0.226, 0.011 and 0.042 mg/L, respectively. These concentrations were all below what ADWG (2011) recommended. The concentrations of orthophosphate, nitrate, nitrite and ammonia and in the effluent were 0.060, 0.082, 0.009 and 0.035 mg/L, respectively, giving an average removal rate of 48%, 64%, 21% and 18%, also respectively. The high removal rates for orthophosphate, nitrate could be due to the high concentrations of these two parameters in the influent.

Total coliforms and faecal coliforms

The GAC adsorption filtration reduced the total and faecal coliform count by 24% and 66%, respectively (Table 6.2). Despite this the total and faecal coliform counts in the influent and effluent samples did not comply with the ADWG (2011) limit of less than 1 coliform per 100 mL and for this reason further treatment is required.

Table 6.2 Laboratory analysis of GAC adsorption filter - second experiment

Day / **Parameter**	**ADWG (2011)**	**Detectable limit**	Days 1 - 90			
			# Samples	Influent	Effluent	% Removal
pH	6.5 - 8.5	3.0 - 10	7	6.87 6.71 - 7.05	6.95 6.77 - 7.22	NQ
Total Suspended Solids	-	0.5	7	2.29 <0.5 - 5.5	1.14 <0.5 - 3.5	50%
Turbidity (NTU)	<5	0.2	7	3.21 2.0 - 6.0	1.86 1.0 - 3.0	42%
True Colour (PtCo)	NA	1	7	244 185 - 276	181 129 - 238	26%
Total Dissolved Salts (mg/L)	NA	0.1	7	107.9 103 - 123	99.3 80 - 109	8%
Water hardness (mg/L CaCO3)	200	0.1	7	43.2 41 - 49	36.9 24 - 41	15%
Sodium (mg/L)	NA	0.05	7	13.43 10.3 - 16.5	13.14 10.2 - 15.5	2%
Potassium (mg/L)	NA	0.001	7	6.95 5.7 - 7.5	6.87 5.5 - 7.5	1%
Calcium (mg/L)	NA	0.05	7	14.28 13.1 - 16.8	12.01 7.9 - 14.9	16%
Magnesium (mg/L)	NA	0.05	7	1.68 1.6 - 1.7	1.51 1.1 - 1.7	10%

Notes:

- NQ – not quantified, this parameter is not reduced by adsorption
- Arsenic, Copper and Cadmium were not detected
- 1st row- Mean average value, 2nd row - range of value
- Values exceeding the ADWG (2011) limit are shown in bold

Table 6.2 Laboratory analysis of GAC adsorption filter-second experiment (cont'd.)

Day / Parameter	ADWG (2011)	Detectable limit	Days 1 - 90 # Samples	Influent	Effluent	% Removal
Orthophosphate (mg/L)	NA	0.005	7	0.12 0.09 - 0.15	0.06 0.04 - 0.10	48%
Nitrate (mg/L)	<50	0.005	7	0.226 0.010 - 0.944	0.082 0.050 - 0.177	64%
Nitrite (mg/L)	<3	0.005	7	0.011 0.003 - 0.025	0.009 <0.005 - 0.022	21%
Ammonia (mg/L)	<0.5	0.005	7	0.042 0.015 - 0.071	0.035 0.007 - 0.065	18%
Total Coliforms (cfu/100mL)	< 1	1	7	**72.9** **50 - 120**	**55.7** **10 - 90**	24%
Faecal Coliforms (cfu/100mL)	< 1	1	7	**45.7** **10 - 90**	**15.7** **10 - 50**	66%
Aluminium (mg/L)	<0.2	0.001	7	0.076 0.069 - 0.090	0.104 0.082 - 0.130	NQ
Iron (mg/L)	<0.3	0.005	7	0.298 0.225 - **0.427**	0.272 0.200 - **0.397**	9%
Manganese (mg/L)	<0.1	0.001	7	**0.113** 0.040 - **0.140**	0.061 0.020 - 0.095	46%
Lead (mg/L)	<0.01	0.001	7	**0.021** **0.019 - 0.025**	0.009 0.007 - **0.011**	59%
Zinc (mg/L)	<3	0.001	7	0.052 0.038 - 0.070	0.029 0.018 - 0.035	45%

Notes:
- NQ – not quantified, this parameter is not reduced by adsorption
- Arsenic, Copper and Cadmium were not detected
- 1st row- Mean average value, 2nd row - range of value
- Values exceeding the ADWG (2011) limit are shown in bold

6.4. Pilot scale GAC adsorption filter

A pilot scale GAC adsorption filter column was connected to the downpipe of the residential dwelling to intercept and treat the roof runoff before it entered the rainwater tank (T1). The experimental set-up was discussed in Chapter 3 (Section 3.3.2 and Figure 3.21). The GAC filter operates during actual rainfall events when water flowed in the roof gutter and downpipe. The GAC adsorption filter was monitored for a period of 62 days; during this time six rainfall events occurred as shown in Table 6.3. Sampling of the influent to the GAC adsorption filter and its effluent was carried out during three rainfall events on days 0, 30 and 60.

Table 6.3 Details of rainfall events captured by the pre-rainwater tank inline GAC adsorption filter

Day	Rainfall Event	Rainfall Duration (hrs)	Rainfall Depth (mm)	Maximum Intensity (mm/hr)
0	1	15	59	6
11	2	<0.5	3	6
15	3	<0.5	8	16
30	4	4	5	2
60	5	3	5	2
62	6	6	11	3.5

Total organic carbon, turbidity, true colour and suspended solids

The GAC filter performed effectively in removing TOC, turbidity, true colour and TSS during its 60 days of operation (Table 6.4). The removal rate of TOC was greater than 87% in all three samples (day 0, 30 and 60) from an influent TOC concentration range between 7.5-8.9 mg/L. The removal rate of turbidity ranged from 50-75% in the three samples where the influent concentrations were relatively low, ranging between 2-8 NTU (Table 6.4). The removal rate of TSS was greater than 96% for all three samples. TSS was reduced to detectible limits from influent concentrations ranging from 13-30 mg/L. The removal rate of true colour was high with removal rates of greater than 84% (Table 6.4). The concentration of the above parameters in the effluent samples was below the ADWG (2011) limit. The higher removal rates of the GAC adsorption pilot filter compared to the laboratory column experiments (Sections 6.1 and 6.2) are probably due to the smaller amount of influent rainwater that passed through the former.

Heavy metals

The GAC adsorption filter performed effectively in removing heavy metals (Table 6.4). The concentration of lead in the influent varied between 0.005-0.021 mg/L. The ADWG (2011) limit for lead is 0.01 mg/L and as such the concentration of lead in two out of the three influent samples exceeded this limit. The removal rate of lead was greater than 72% with all three samples (Day 0, 30, 60) and the concentration fell to detectible limits (0.001 mg/L).

The concentration of iron in the influent was between 0.054-0.067 mg/L and was below the ADWG (2011) limit of 0.3 mg/L. The removal rate of iron was 90%, 29% and 77% for samples taken on Day 0, 30 and 60, respectively (Table 6.4).

The concentration of zinc in the influent was between 0.027-0.036 mg/L and below the ADWG (2011) limit of 3 mg/L. The removal rate of zinc was more than 85% in all three effluent samples (Day 0, 30, 60).

The concentration of aluminium in the influent was between 0.084-0.140 mg/L and already complied with the ADWG (2011) limit of 0.2 mg/L before filtration. The removal rate of aluminium was greater than 68%. Higher removal rates could be explained by the smaller amount of influent rainwater passing through the GAC adsorption pilot filter compared to the laboratory column experiments (Sections 6.1 and 6.2).

The removal rate of manganese in the three effluent samples was greater than 64% with the concentration of manganese in the influent samples already below the ADWG (2011) limit.

No traces of copper, arsenic or cadmium were detected in the influent samples.

Anions and cations, total dissolved salts, water hardness and pH

The GAC adsorption filter had, in general, little to no effect in removing anions, cations, or reducing the total dissolved salts and water hardness. Any change in pH was minimal since no chemicals were added in this filtration process.

Orthophosphate, nitrate, nitrite and ammonia

Orthophosphate was detected in 2 of the 3 days of sampling (day 0 and 60). For these two days the removal rate was greater than 86% (Table 6.4). While sampling at day 0 showed no improvement for nitrate, the latter two samples (days 30 and 60) showed that the removal was greater than 88% (Table 6.4). The GAC adsorption filter also performed negligibly in removing nitrite except for the sample collected on day 60 when a 68% removal rate was achieved. Ammonia was reduced by more than 87% for all samples (Table 6.4). Large reduction rates were obtained when the influent concentrations were high.

Total coliforms and faecal coliforms

Other than the initial reduction of total coliforms observed on day 0, for the remainder of the pilot trial, the total coliform count increased over the 60 day period (Table 6.4). The faecal coliforms also highlighted a similar pattern with an increase in count after each sampling period (Table 6.4). The total and faecal coliform in the influent and effluent samples did not comply with the ADWG (2011) limit.

Summary

The filter did perform well in removing heavy metals such as zinc and lead of which the influent of lead did not comply with the ADWG (2011) limit until the GAC adsorption filter was used. It also removed aluminium, which in the laboratory column experiments was not removed. Similarly the influent of turbidity did not comply with the ADWG (2011) limit until it was filtered via GAC adsorption. The parameter that did not improve with GAC adsorption was total coliforms and faecal coliforms. Although the removal of most other parameters was observed after filtration, all other parameters already complied with the ADWG (2011) limit before filtration commenced.

Hydraulic performance of GAC adsorption filter

While observing the performance of the GAC adsorption filter during rainfall events, it was noted that when rainfall intensities exceeded approximately 2 mm/hr, which is not a large rainfall event, the GAC adsorption filter could not cope with the runoff from the roof collected in the gutter and drained through the down pipe and into the filter. The rainwater ponded in the gutter on the roof and subsequently over-topped the gutter once the filter's storage and flow capacity was exceeded. This evidently reduces the available yield of rainwater that could be captured and stored in the rainwater tank.

The laboratory experiments operated comfortably under gravity head at a flow rate of 1 m/hr. If this rate is applied to the pilot scale filter it equates to a flow capacity of only17.6 L/hr and far less than what is expected during a storm event. As the average Australian residential single dwelling house roof area is approximately 250 m² (see Chapter 2.4.1) and assuming a capture storm event has a moderate intensity of 10 mm/hr, a filter column with a rather large diameter of 0.565 m or equivalent cross-sectional area of 0.25 m² would be required.

As a rather large GAC pre-filter would not be economically appropriate, the alternative is to place the filter downstream of the rainwater tank. The average annual rainfall of the Sydney metropolitan area is 1215 mm of rainfall based on the last 153 years of available rainfall data with on average 143.2 days of rainfall experienced per year (Australian Bureau of Meteorology, 2012). With the average household roof area of 250 m² and assuming that all rainfall runoff from the roof is collected in the rainwater tanks, the average total annual yield equates to 303,700 L/year or 34.7 L/day.

If the 34.7 L/day average flow rate was passed through a filter column at a rate of 1 m/hr constantly all year round to maintain a steady influent feed supply of water to the GAC, the internal diameter of the filter required to maintain this flow rate is nominally 43 mm.

Table 6.4 Laboratory analysis of inline GAC adsorption filter upstream of rainwater tank

Day	**Day 0**			**Day 30**			**Day 60**		
Parameter	**Influent**	**Effluent**	**% Removal**	**Influent**	**Effluent**	**% Removal**	**Influent**	**Effluent**	**% Removal**
pH	**5.88**	6.81	-	6.44	6.55	-	6.28	6.49	-
Total Suspended Solids (mg/L)	13.0	0.5	96%	30.0	0.5	98%	19.0	0.5	97%
Turbidity (NTU)	2	1	50%	**5**	2	60%	**8**	2	75%
True Colour (PtCo)	77	12	84%	31	1	97%	114	1	99%
Total Dissolved Salts (mg/L)	35	76	NQ	72	68	5%	72	92	NQ
Water hardness (mg/L $CaCO_3$)	6	31	NQ	31	27	13%	22	30	NQ
Sodium (mg/L)	4.2	4.6	NQ	4.5	4.2	6%	5.1	6.2	NQ
Potassium (mg/L)	0.9	2.2	NQ	2.2	1.7	22%	4.1	3.5	14%
Calcium (mg/L)	1.2	11.3	NQ	11.3	9.8	13%	7.2	10.3	NQ
Magnesium (mg/L)	0.8	0.8	NQ	0.7	0.7	NQ	0.9	1.0	NQ

Notes:

- NQ – not quantified, this parameter is not reduced by adsorption
- Arsenic, Copper and Cadmium were not detected
- Values exceeding the ADWG (2011) limit are shown in bold

Table 6.4 Laboratory analysis of inline GAC adsorption filter upstream of rainwater tank (cont'd.)

Day	Day 0			Day 30			Day 60		
Parameter	**Influent**	**Effluent**	**% Removal**	**Influent**	**Effluent**	**% Removal**	**Influent**	**Effluent**	**% Removal**
Orthophosphate (mg/L)	0.047	0.005	89%	0.005	0.005	NQ	0.086	0.012	86%
Nitrate (mg/L)	0.026	0.035	NQ	0.604	0.047	92%	0.195	0.022	88%
Nitrite (mg/L)	0.005	0.005	NQ	0.005	0.005	NQ	0.016	0.005	68%
Ammonia (mg/L)	0.280	0.014	95%	0.465	0.026	94%	0.265	0.033	87%
Total Coliforms (cfu/100mL)	**7,200**	**2,000**	72%	**5,400**	**9,000**	NQ	**800**	**54,000**	NQ
Faecal Coliforms (cfu/100mL)	1	**10**	NQ	1	**330**	NQ	1	**1,100**	NQ
Total Organic Carbon (mg/L)	8.9	0.9	90%	7.5	1.0	87%	7.9	0.9	89%
Aluminium (mg/L)	0.084	0.027	68%	0.123	0.038	69%	0.140	0.012	91%
Iron (mg/L)	0.066	0.007	90%	0.054	0.039	29%	0.067	0.016	77%
Manganese (mg/L)	**0.109**	0.011	90%	0.032	0.012	64%	**0.118**	0.009	92%
Lead (mg/L)	**0.017**	0.001	96%	0.005	0.001	72%	**0.021**	0.001	97%
Zinc (mg/L)	0.027	0.002	93%	0.032	0.003	92%	0.036	0.006	85%

Notes:

- NQ – not quantified, this parameter is not reduced by adsorption
- Arsenic, Copper and Cadmium were not detected
- Values exceeding the ADWG (2011) limit are shown in bold

6.5. Conclusion

6.5.1. Laboratory GAC adsorption filter system

The quality of the influent during the two laboratory column experiments varied because raw rainwater was used. During the two experiments, totalling more than 240 days, influent samples that were analysed largely complied with the ADWG (2011) standard with the exception of total and faecal coliforms. Influent samples which at times did not comply were turbidity and heavy metals (iron, manganese and lead). The TSS, turbidity and colour all performed well with all effluent samples being below what ADWG (2011) recommended. The TSS, turbidity and colour all performed well with average removal rates of at least 50%, 42% and 26%.

Heavy metals such as manganese, zinc and lead also achieved good removal rates averaging at least 46%, 45% and 50%, respectively. The concentration of lead in all effluent samples complied with the ADWG (2011) limit with the exception of one sample being 0.011 mg/L. The concentration of iron in two effluent samples was more than 0.3 mg/L over the ADWG (2011) limit. Aluminium was not effectively removed by the GAC adsorption filter.

The GAC adsorption filter had a small to negligible effect in removing anions, cations, or in reducing the total dissolved salts and water hardness. The effectiveness of the GAC adsorption filter for orthophosphate, nitrate, nitrite and ammonia was generally small if not negligible. All influent and effluent for these parameters' samples were below the ADWG (2011) limit. Finally, other parameters not complying with the ADWG (2011) limit following GAC adsorption filtration were total and faecal coliforms. Further treatment is necessary for these parameters to comply with the ADWG (2011) limit if the rainwater is to be used for potable purposes.

6.5.2. Pilot scale inline GAC adsorption filter

Unlike the laboratory-based experiment which maintained continuous flow through the GAC adsorption filter, the pre-rainwater inline GAC adsorption filter operated with rain water flowing through it only every 10 to 30 days during a storm event. The pre-rainwater tank inline GAC adsorption filter did not operate effectively due to poor hydraulic performance. Nevertheless the filter did perform well in reducing the concentration of many parameters to the extent that all, except total coliforms and faecal coliforms, complied with the ADWG (2011) recommendation.

Chapter 7

University of Technology, Sydney

Faculty of Engineering &

Information Technology

7. Membrane Micro-Filtration

7.1. Introduction

Membrane filtration experiments were carried out using a stainless steel membrane (Steriflow Filtration Systems) and two polymeric membranes (INGE Watertechnologies AG, and Ultra Flo). The latter systems were tested in a dead-end mode of filtration with and without pre-treatment of Granular Activated Carbon (GAC). Raw rainwater was passed through filter columns to analyse the effectiveness of GAC at different flow rates. Water samples for analysis were collected from the raw rainwater and the effluent from the GAC pre-treatment. The effluent water from the pre-treatment experiments was collected for further experimental analysis through the membrane filters. The experimental set-up is described in detail in Section 3.3.3.

7.2. Clean membrane flux determination of micro-filtration membranes

Initially clean membrane flux tests were conducted under various trans-membrane pressures to determine their suitability as a membrane filter to treat raw water stored in rainwater tanks. All three filters (Steriflow Filtration Systems, INGE Watertechnologies AG, and Ultra Flo) were thoroughly cleaned to the manufacturer's specification and tested using distilled water.

Steriflow filtration systems stainless steel micro-filtration membrane

When the stainless steel membrane (Steriflow Filtration Systems) was tested using a column set at 2 m of driving head across the filter, it failed to flow. The membrane filter was then connected to a pump. The flux was about 350 L/m^2/hr at 50 kPa (5 m of head) (Figure 7.1).

The SteriFlow membrane is a rigid stainless steel structure, which relies on having a thicker membrane walls compared with flexible hollow fibre membranes. This thicker wall requires an increase of pressure head to drive the fluid through the membrane. The manufacturing process of the SteriFlow membrane system is currently not refined to produce reliable and durable thinner walled membranes. Due to the high pressure requirements, this type of membrane is not considered suitable for the gravity-driven filtration applications such as rainwater filtration.

INGE Watertechnologies AG Multibore micro-filtration membrane

The INGE Watertechnologies AG Multibore micro-filtration clean membrane filter demonstrated a reasonably linear relationship between the applied head of distilled water and flux (Figure 7.1). The Multibore membrane was capable of a flux as high as 82 L/m^2/hr with only a relatively small applied pressure head of 20 kPa (2 m of head). This is a reasonable flux considering the membrane has a specified pore size of 0.02 μm. The minimum pressure head required to initiate flow through the membrane was approximately 2 kPa (0.20 m of head).

Ultra Flo hollow fibre micro-filtration membrane

The Ultra Flo hollow fibre micro-filtration clean membrane filter demonstrated a linear relationship between the applied head of distilled water and the flux (Figure 7.1). The membrane produced a flux as high as 300 L/m^2/hr with only a relatively small applied pressure head of 20 kPa (2 m of head). This is a relatively high flux for a membrane that has a specified pore size of 0.1 μm. The minimum pressure head required to initiate flow through the membrane was approximately 1 kPa (0.1 m of head). This is a distinct advantage for a gravity head membrane-based filtration system for a rainwater tank.

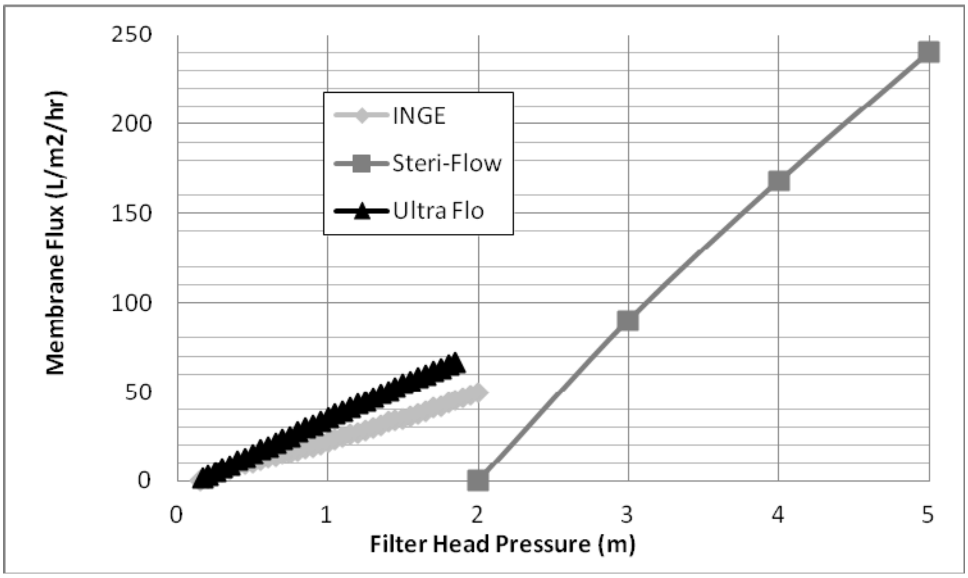


Figure 7.1 Clean membrane flux at incremental head pressures.

7.3. Ultra Flo hollow fibre micro-filtration under gravitational head

Ultra Flo membrane filtration experiments were carried out using a polymeric hollow fibre micro-filtration membrane (Table 3.10, Section 3.2.4) with and without a pre-treatment of GAC adsorption.

7.3.1. Flux decline

Flux decline without pre-treatment

The flux decline in the Ultra Flo hollow fibre micro-filtration membrane filtration was tested with raw rainwater at different driving heads. Rainwater was collected from a domestic roof and rainwater tank (T1) (Table 3.1, Section 3.2.1) located in Sydney, New South Wales (Figure 3.1, Section 3.2.1). The experimental set-up is described in Section 3.3.3. The flux decline results are shown in Figure 7.2. Experiments with a higher driving head on the membrane makes an initial higher flow rate possible. After approximately 100 hours of operation, all tests converged to a similar flux decline path regardless of driving head. The flux decline tests show that all driving heads generally result in a final stable flux of around 4 to 5 L/m^2/hr.

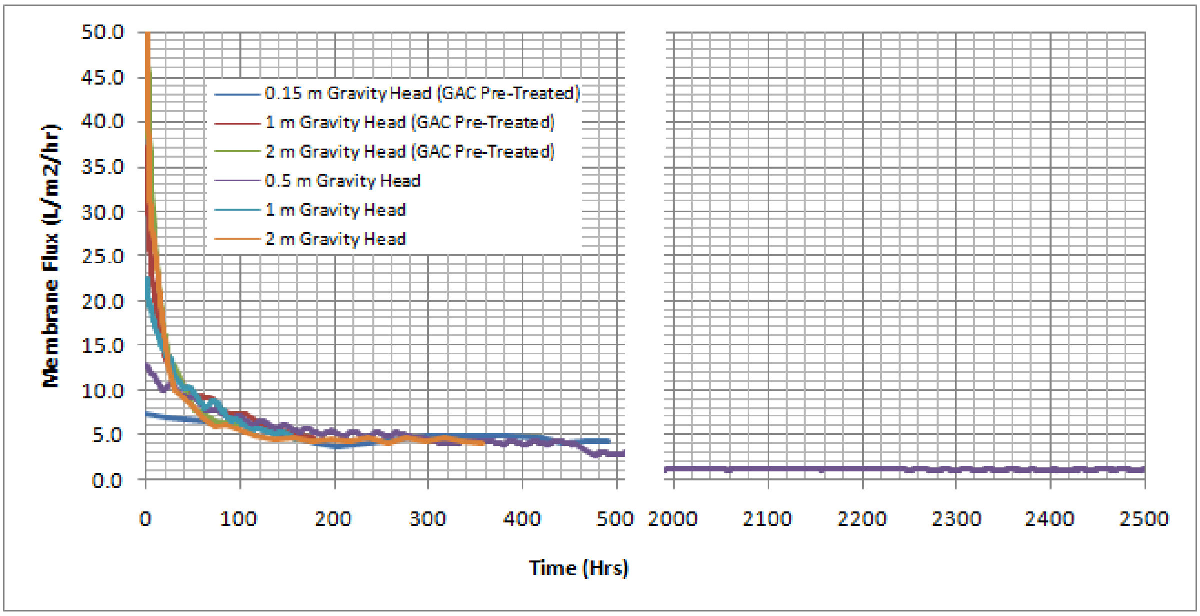


Figure 7.2 Comparison of Ultra Flo hollow fibre micro-filtration membrane flux decline under various gravity head with and without GAC adsorption pre-treatment.

Flux decline with pre-treatment

The flux decline of the Ultra Flo hollow fibre micro-filtration membrane was tested with different driving heads using GAC adsorption pre-treated rainwater. The experimental set-up is described in detail in Section 3.3.3. The results of the flux decline are shown in Figure 7.2.

Figure 7.2 shows that the flux decline with all three driving heads (0.5 m, 1 m and 2 m) resulted in a final stable flux of around 4-5 L/m^2/hr after 400 hours of operation regardless of the driving head. The initial flux decline of the membrane with 2.0 m of driving head resulted in a higher initial flux but merged with the same flux decline path of other driving heads after about 140 hours of operation. These results show that the GAC adsorption pre-treatment of rainwater prolonged the membrane flux decline but regardless of driving head did not ultimately achieve a dissimilar long-term flux. The final stable flux of around 4-5 L/m^2/hr was the same as that observed for membrane filtration without pre-treatment. In terms of flux decline, the benefit of GAC adsorption pre-treatment is that it permits a higher initial flux, with a slower flux decline for the first 140 hours after which the flux merges with the flux of untreated rainwater.

Long-term operation of micro-filtration membrane

The Ultra Flo membrane was tested using raw rainwater without pre-treatment and a 0.5 m driving head continuously in excess of 2600 hours (108 days) to observe how the membrane performed in the long-term. Figure 7.2 illustrates that the long-term flux decline was not steady after 500 hours and continued to slowly decline. After 2600 hours (108 days), the flux of the membrane was measured at an average of 1.2 L/m^2/hr, fluctuating between 1.03 and 1.32 L/m^2/hr. During this period the membrane was not back-washed or cleaned.

7.3.2. Membrane filtration without pre-treatment

Raw water quality

While monitoring the flux decline of the Ultra Flo membrane filter, grab samples of the influent and effluent were collected to analyse pollutant removal efficiencies of the membrane filter when operating without pre-treatment. Actual raw rainwater was used in the experiments and the water quality varied with the time of collection. Results of the parameters tested are shown in Table 7.1.

The influent raw rainwater quality of these membrane filtration experiments generally complied with most of the ADWG parameters tested (Table 7.1). The parameters that did not comply were total and faecal coliforms. The average total and faecal coliform counts in the samples of raw rainwater were relatively high at 560 and 500, respectively. The ADWG recommended limit for total and faecal coliform is less than 1 CFU/100 mL.

The majority of parameters tested were at concentrations less than specified in ADWG. The concentration of aluminium, copper, iron, manganese and lead were all very at low to negligible concentrations and were all below the ADWG limit. Zinc had an influent concentration of 0.15 mg/L and below the ADWG limit of 3 mg/L.

The ADWG has set recommended limits for nitrate, nitrite and ammonia at 50 mg/L, 3 mg/L and 0.5 mg/L, respectively. All samples for nitrate, nitrite and ammonia were below this limit. Furthermore the concentration of TOC in the rainwater samples was 3.68 mg/L.

Turbidity and total suspended solids

The ADWG (2011) recommend a turbidity limit of 5 NTU. The influent rainwater samples for the membrane filtration were below this recommended limit with an average concentration of 2.0 NTU. Membrane filtration reduced the turbidity of the effluent to 0.3 NTU or an overall 85% reduction in turbidity (Table 7.1). The average TSS concentration in the influent rainwater samples was 3.0 mg/L. The membrane filtered the influent rainwater to less than 0.5 mg/L (instrument detection limits) which represents an overall reduction in TSS of at least 83%.

Total organic carbon

Membrane filtration did not provide any improvement in the TOC removal (Table 7.1). TOC removal by membrane filtration is poor. Pre-treatment such as GAC filtration is normally applied to improve TOC removal.

Anions and cations, total dissolved salts, water hardness and pH

Membrane filtration had a small to no effect concerning the removal of anions, cations, or reducing the total dissolved salts and water hardness. There was no quantifiable change in pH (Table 7.1).

Orthophosphate, nitrate, nitrite and ammonia

All raw water samples were below the ADWG limit for orthophosphate, nitrate, nitrite and ammonia. Membrane filtration provided negligible to no reduction in levels of orthophosphate, nitrate nitrite concentrations (Table 7.1). Ammonia was reduced by 33%.

Heavy metals

Aluminium: The ADWG (2011) limit for aluminium is 0.2 mg/L. The influent rainwater concentration was below the recommended limit with an average concentration of 0.047 mg/L. Membrane filtration was able to reduce the average aluminium concentration to 0.037 mg/L or a 21% reduction (Table 7.1).

Copper: The ADWG limit for copper is 2 mg/L. The influent rainwater concentration was below this limit with an average concentration of 0.020 mg/L. Membrane filtration reduced the average copper concentration to 0.015 mg/L or a 25% removal (Table 7.1).

Iron: The ADWG (2011) limit for iron is 0.3 mg/L. The influent rainwater iron concentrations were below the ADWG (2011) recommended limit with an average concentration of 0.063 mg/L. The effluent when using membrane filtration had an average iron concentration of 0.054 mg/L which is a reduction of 14% (Table 7.1).

Lead: The ADWG (2011) recommends a concentration limit of 0.01 mg/L for lead. The influent rainwater lead concentrations were below the recommended limit with a very low average concentration of only 0.002 mg/L (Table 7.1). The membrane filtration of influent rainwater decreased the average lead concentration to less than 0.001 mg/L (instrument detection limits) which is a reduction of more than 50%.

Zinc: The ADWG (2011) recommends a zinc concentration of less than 3 mg/L. The raw rainwater concentrations were significantly below the ADWG limit with the average influent concentration at 0.15 mg/L being recorded. Following membrane filtration the average zinc concentration was 0.101 mg/L, a 32% reduction (Table 7.1).

All other concentrations of heavy metals were well within the ADWG recommended limits. The concentration levels of arsenic, boron, cadmium, chromium, manganese, mercury, molybdenum, nickel selenium and silver were all negligible at less than 0.001 mg/L.

Total coliform and Faecal colifom

The ADWG (2011) recommend a limit of less than 1 CFU/100 mL for e-coli, faecal and total coliform counts. All influent rainwater samples exceeded the recommended limits for these parameters. The microfiltration membrane was generally able to reduce all three types of coliforms to less than 1 CFU/100 mL (Table 7.1). Further disinfection treatment is required to either reduce this further or guarantee all removal while also targeting the removal of viruses.

7.3.3. Membrane filtration with pre-treatment

Raw water quality

While monitoring the flux decline of the Ultra Flo membrane filter, grab samples of the influent and effluent (including the effluent of the GAC adsorption filter) were collected to analyse the membrane filter's pollutant removal efficiencies when operating with pre-treatment.

The influent raw rainwater quality when experimenting with GAC pre-treatment followed by membrane microfiltration typically complied with most of the ADWG parameters tested and are presented in Table 7.2. Actual raw rainwater was used in the experiments and the water quality varied with the time of collection and differed for the three experiments conducted with driving head of 0.15 m, 1 m and 2 m (Table 7.2). The concentration of the water quality parameters in the raw rainwater varied significantly. Parameters that did not comply included turbidity, microbiological (total and faecal coliforms), and the heavy metals iron, manganese and lead.

The average turbidity of the influent raw water was 1.8 NTU in a range between 0.5-5.9 NTU (Table 7.2). The upper value exceeds the ADWG limit of 5 NTU. The total and faecal coliform counts in the samples of rainwater were relatively high with an average count of 182 (in a range of 20-1000) and 108 (in a range of <1-1000), respectively. The ADWG recommended limit for total and faecal coliform is less than 1 CFU/100 mL. While the average influent concentration of iron and manganese were both below the ADWG limits of 0.3 mg/L and 0.1 mg/L, respectively at 0.135 mg/L (in a range of 0.06-

0.83 mg/L) and 0.033 mg/L (in a range of <0.001-0.140 mg/L) respectively, some individual samples exceeded the ADWG limits (Table 7.2). Lead contained an average influent concentration of 0.008 mg/L (in a range of 0.001-0.031 mg/L) and less than the ADWG limit of 0.01 mg/L. However, it contained individual samples that did not comply. The concentration levels of aluminium, copper and zinc all showed very low to negligible concentration. The ADWG limits for nitrate, nitrite and ammonia are 50 mg/L, 3 mg/L and 0.5 mg/L, respectively. All samples for nitrate, nitrite and ammonia were below the ADWG limits except for one sample of ammonia which had a concentration of 1.364 mg/L.

Turbidity, true colour and total suspended solids

Turbidity: The ADWG (2011) recommend a turbidity limit of 5 NTU. The influent raw rainwater samples when using pre-treatment of GAC adsorption followed by membrane filtration were below the recommended limit with an average value of 1.8 NTU. The effluent of GAC adsorption pre-treatment had an average turbidity of 1.1 NTU, which was equivalent to a 38% removal efficiency. The effluent from membrane filtration had an average of 0.1 NTU (instrument detection limits) in a range of 0.1-0.2 NTU. The overall reduction in turbidity of GAC adsorption pre-treatment followed by membrane filtration achieved an average of 94% (Table 7.2).

True colour: The ADWG (2011) does not have a limit for true colour. The influent raw water samples varied with the actual rainwater being used. One portion of influent raw water had a true colour of 1-25 PtCo units (used for the experiment with 0.15 m driving head) while the remainder had a true colour of between 185-276 PtCo units (used for experiments with 1m and 2 m driving head). The GAC adsorption pre-treatment reduced the true colour down to between 100-174 PtCo for the poorer quality batch (used for experiments with 1m and 2 m driving head). Membrane filtration provided further reduction to between 25-70 PtCo for the poorer quality batch. The average reduction was 80%. This was generally consistent for both the better and poor influent concentrations (Table 7.2).

Total suspended solids: The influent raw rainwater samples for the membrane treatment with GAC adsorption pre-treatment were already quite low with an average TSS concentration of 1.4 mg/L. The GAC adsorption pre-treatment achieved an average TSS concentration of 1.0 mg/L or 28% removal efficiency. The effluent from membrane filtration had a TSS concentration of less than 0.5 mg/L (instrument detection limits). The overall average reduction in TSS using membrane filtration with GAC adsorption pre-treatment was 64% (Table 7.2).

Total organic carbon

The ADWG (2011) does not recommend a limit for total organic carbon. The influent raw rainwater samples for the membrane treatment with GAC adsorption pre-treatment contained an average concentration of 2.66 mg/L. The GAC adsorption pre-treatment achieved an average concentration of 0.95 mg/L or removal efficiency of 64%. The membrane filtration reduced the concentration of TOC to 0.56 mg/L. The overall reduction in TOC using membrane filtration with GAC adsorption pre-treatment was an average of 79%.

Anions and cations, total dissolved salts, water hardness and pH

Membrane filtration had a small to no effect in removing anions, cations, or reducing the total dissolved salts and water hardness. There was no quantifiable change in pH (Table 7.2).

Orthophosphate, nitrate, nitrite and ammonia

The concentrations of nitrate, nitrite and ammonia in all raw rain water samples were below the ADWG limits except for one sample of the latter (ammonia) being at 1.364 mg/L which was over two times the ADWG limit of 0.5 mg/L. Orthophosphate, nitrate and ammonia concentrations all fell with the GAC adsorption pre-treatment by 42%, 38% and 36%, respectively. Membrane filtration was able to further reduce orthophosphate, nitrate and ammonia concentrations with a total reduction of 58%, 40% and 39%, respectively. Nitrite showed negligible improvement with GAC adsorption but decreased by 28% following membrane filtration. This was most likely due to the very low influent concentration which was often already near detectable limits (Table 7.2).

Heavy metals

Aluminium: The ADWG (2011) recommends an aluminium concentration limit of 0.2 mg/L. The influent rainwater samples were all below the recommended limit with an average influent concentration of 0.038 mg/L. The GAC adsorption pre-treatment did not improve upon the influent water samples leading to an average concentration of 0.047 mg/L, which was higher than the influent concentration. This could be due to the initial removal by the GAC filter and its subsequent leaching out. Following pre-treatment, membrane filtration was able to reduce the aluminium levels to an average of 0.020 mg/L which is a 46% reduction from the influent concentration (Table 7.2).

Copper: The ADWG (2011) recommends a limit for copper of 2 mg/L. All water samples were below the ADWG limit with the highest influent sample at 0.016 mg/L. The GAC adsorption pre-treatment could reduce the average copper concentration by 63% to 0.003 mg/L while the membrane achieved over 90% removal to detectable limits of less than 0.001 mg/L (Table 7.2).

Iron: The ADWG (2011) recommends a limit for iron of 0.3 mg/L. The influent rainwater concentration for the membrane treatment with GAC adsorption pre-treatment generally complied with the recommended limit with an average concentration of 0.135 mg/L. Individual samples, however, exceeded the recommended limit with concentrations of up to 0.830 mg/L. The GAC adsorption pre-treatment achieved an average concentration of 0.122 mg/L or 9% removal efficiency of iron from the influent rainwater. The effluent from membrane filtration had an average iron concentration of 0.005 mg/L (instrument detection limits) giving an overall reduction of greater than 98% (Table 7.2).

Manganese: Manganese has an ADWG limit of 0.1 mg/L. Although the average influent concentrations were below the limit with an average concentration of 0.033 mg/L, some samples had concentrations higher than this - up to 0.140mg/L. The GAC adsorption pre-treatment was able to reduce the average manganese concentration by 47% to 0.018 mg/L while the membrane filtration, on average, achieved little further improvement over the pre-treatment with a total removal of 44% (Table 7.2).

Lead: The ADWG (2011) recommends a limit for lead of 0.01 mg/L. Some of the influent raw rainwater samples exceeded the recommended ADWG limit with an average influent value of 0.008 mg/L. The GAC adsorption pre-treatment reduced the average lead concentration to 0.006 mg/L providing a removal efficiency of 22%. The membrane filtration reduced the lead concentration to less than 0.001 mg/L (instrument detection limits). The overall reduction in lead concentration using membrane filtration with GAC adsorption pre-treatment was generally more than 87% (Table 7.2).

Zinc: The ADWG (2011) recommends a limit for zinc of 3 mg/L. All raw rainwater samples had zinc concentrations significantly below the ADWG limit with the highest influent concentration being 0.07 mg/L. The pre-treatment of GAC adsorption was able to reduce the average zinc concentration by 65% to 0.013 mg/L. Following membrane filtration the removal efficiency was 67% while the average concentration fell to 0.012 mg/L (Table 7.2).

All other heavy metals were well within the ADWG recommended limits. The concentration levels of arsenic, boron, cadmium, chromium, mercury, molybdenum, nickel selenium and silver were all negligible with concentrations of less than 0.001 mg/L.

Total coliform and Faecal colifom

The ADWG (2011) recommend a limit of <1 CFU/100 mL for faecal and total coliform counts. All influent rainwater samples had counts exceeding the recommended limits for faecal and total coliform counts. GAC adsorption pre-treatment showed some improvement in the coliform counts. However, after passing the pre-treated water through the membrane, the faecal coliform count reduced to less than 1 CFU/100 mL. Total coliform count declined by 59% but the count was still higher than the ADWG recommended limit.

Summary

Ultra Flo membrane treatment with GAC adsorption pre-treatment was able to reduce the concentration of all water quality parameters to below that specified in the ADWG (2011). The exception was for faecal and total coliforms. Further disinfection treatment is required to either reduce this further or to guarantee all removal. This is good practice as micro-filtration membrane treatment does not remove viruses.

Table 7.1 Laboratory analysis of Ultra Flo hollow fibre micro-filtration membrane pollutant removal without pre-treatment.

Parameter	ADWG limit	Detectable limit	0.5m head(3 samples)		
			Influent	Effluent	% Removal
pH	6.5 - 8.5	3.0 - 10	6.74	6.94	NQ
Total suspended solids (mg/L)	-	0.5	3 2 - 4	<0.5 <0.5 - <0.5	>83%
Turbidity (NTU)	5	0.2	2.0 1.5 - 3	0.3 0.2 - 0.5	85%
Total dissolved salts (mg/L)	-	0.1	53 51 - 54	53 51 - 54	NQ
Water hardness (mg/LCaCO3 equivalent)	200	0.1	30 28 - 33	29 27 - 31	NQ
Sodium (mg/L)	-	0.05	3.9 3.9 -4.0	3.97 3.9 - 4.1	NQ
Potassium (mg/L)	-	0.001	2.07 2.0 - 2.2	1.71 1.6 - 1.8	18%
Calcium (mg/L)	-	0.05	10.5 10.3 - 10.7	10.2 10.0 - 10.5	NQ
Magnesium (mg/L)	-	0.05	0.94 0.9 - 1.0	0.94 0.9 - 1.0	NQ

Orthophosphate (mg/L p)	-	0.005	0.029 0.026 - 0.032	0.028 0.026 - 0.030	NQ
Nitrate (mg/L n)	50	0.005	0.293 0.184 - 0.454	0.354 0.205 - 0.468	NQ
Nitrite (mg/L n)	3	0.005	<0.005 <0.005 - <0.005	<0.005 <0.005 - <0.005	NQ
Ammonia (mg/L n)	0.5	0.005	0.021 0.015 - 0.046	0.014 0.006 - 0.025	33%
Total coliforms (cfu/100 ml)	1	1	**560** **320 - 670**	<1 <1 - <1	>99%
Faecal coliforms (cfu/100 ml)	1	1	**500** **270 - 650**	<1 <1 - <1	>99%
Total organic carbon (mg/L)	-	0.001	3.68 1.48 - 5.42	4.21 1.78 - 5.76	NQ
Aluminium (mg/L)	0.2	0.001	0.047 0.023 - 0.055	0.037 0.020 - 0.051	21%
Copper (mg/L)	2	0.001	0.020 0.018 - 0.021	0.015 0.013 - 0.018	25%
Iron (mg/L)	0.3	0.005	0.063 0.048- 0.075	0.054 0.040- 0.065	14%
Manganese (mg/L)	0.1	0.001	0.001 <0.001 - 0.001	<0.001 <0.001 - <0.001	NQ

Lead (mg/L)	0.01	0.001	0.002 0.001 - 0.004	<0.001 <0.001 - 0.001	>50%
Zinc (mg/L)	3	0.001	0.150 0.07 - 0.21	0.101 0.07 - 0.15	32%

Notes:

- NQ – not quantified, this parameter has not been reduced
- Arsenic and Cadmium were not detected
- 1st row – Mean average value, 2nd row -range of value
- Values exceeding the ADWG (2011) limit are shown in bold

Table 7.2 Ultra-Flo membrane filtration with GAC adsorption pre-treatment

Parameters	ADWG limit	Detectable limit	Raw water (17 samples)	GAC (17 samples)	GAC % removed	0.15 m HEAD (12 samples)	1.0 m HEAD (3 samples)	2 m HEAD (2 samples)	Avg.GAC +Membrane (14 samples)	Avg.GAC +Membrane % removed
						GAC + membrane	GAC + membrane	GAC + MEMBRANE		
pH	6.5 - 8.5	3.0 - 10	7.01 6.59 - 7.55	6.97 6.5 - 7.47	NQ	6.96 6.58 - 7.12	6.96 6.88 - 7.00	6.98 6.58 - 7.37	6.96 6.58 - 7.37	NQ
Total Suspended Solids (mg/L)	-	0.5	1.4 <0.5 - 5.0	1.0 <0.5 - 3.0	28%	<0.5 <0.5 - <0.5	<0.5 <0.5 - <0.5	<0.5 <0.5 - <0.5	<0.5 <0.5 - <0.5	>64%
Turbidity (NTU)	5	0.1	1.8 0.5 - **5.9**	1.1 0.2 - 4.5	38%	0.12 0.1 - 0.2	0.1 0.1 - 0.1	0.1 0.1 - 0.1	0.10 0.1 - 0.2	94%
True Colour (PtCo)	-	1	76 1 - 276	42.6 1 - 174	44%	1.3 1 - 2	36 27 - 46	47.5 25 - 70	15.3 1 - 70	80%
Total Dissolved Salts (mg/L)	-	0.1	65 44 - 123	63 46 - 107	NQ	54 49 - 57	97 80 - 107	102 89 - 115	67 49 - 115	NQ
Water Hardness (mg/L $CaCO_3$)	200	0.1	21.6 5.3 - 49	18.2 4.8 - 39	15%	10.7 5.1 - 26.7	29 7 - 42	36.5 36 - 37	17.0 5.1 - 42	21%
Sodium (mg/L)	-	0.05	10.7 2.3 - 16.5	10.8 2.4 - 15.5	NQ	9.7 2.3 - 13.0	12.3 8.6 - 14.5	10 9 - 11	10.2 8.4 - 14.5	4%
Potassium (mg/L)	-	0.001	2.45 0.4 - 7.4	2.35 0.4 - 7.2	3%	0.7 0.5 - 1.4	5.4 1.7 - 7.5	6 6 - 6	2.2 0.5 - 7.5	8%
Calcium (mg/L)	-	0.05	7.0 1.0 - 16.8	5.9 0.9 - 13.1	15%	3.1 0.8 - 9.8	9.9 2.7 - 14.3	12.5 12 - 13	5.4 0.8 - 14.3	22%

Parameters	ADWG limit	Detectable limit	Raw water (17 samples)	GAC (17 samples)	GAC % removed	0.15 m HEAD (12 samples) GAC + membrane	1.0 m HEAD (3 samples) GAC + membrane	2 m HEAD (2 samples) GAC + MEMBRANE	Avg.GAC +Membrane (14 samples)	Avg.GAC +Membrane % removed
Magnesium (mg/L)	-	0.05	2.2 0.5 - 3.7	2.17 0.5 - 4.4	1%	2.4 0.5 - 3.4	1.1 0.2 - 1.6	1 1-1	2.0 0.5 - 3.4	9%
Orthophosphate (mg/L P)	-	0.005	0.038 0.004 - 0.152	0.022 0.005-0.051	42%	0.012 0.006-0.016	0.031 0.016-0.047	-	0.016 0.006 - 0.047	58%
Nitrate (mg/L N)	50	0.005	0.538 0.02 - 1.022	0.330 0.005 - 0.862	38%	0.438 <0.005-0.854	0.048 0.015-0.101	<0.005 all <0.005	0.318 <0.005-0.854	40%
Nitrite (mg/L N)	3	0.005	0.008 <0.005-0.038	0.011 <0.005-0.081	NQ	<0.005 <0.005-0.011	0.009 0.003-0.015	<0.005 all <0.005	0.005 <0.005-0.015	28%.
Ammonia (mg/L N)	0.5	0.005	0.142 <0.005-**1.364**	0.089 <0.005-0.306	36%	0.109 0.026-0.256	0.050 0.003-0.128	<0.005 all <0.005	0.086 <0.005-0.256	39%
Total Coliforms (cfu/100 ml)	1	1	**182** **20 - 1000**	**101** **20 - 420**	44%	**100** **30 - 160**	**16** <1 - **30**	<1 <1 - <1	**74** <1 - **160**	59%
Faecal Coliforms (cfu/100 ml)	1	1	**108** <1 - **1000**	**50** <1 - **330**	54%	**1** <1 - **1**	<1 <1 - <1	<1 <1 - <1	<1 <1 - <1	>90%
Total Organic Carbon (mg/L)	-	0.001	2.66 2.10 - 3.73	0.95 0.42 - 1.80	64%	0.43 0.29 - 0.70	1.09 0.068 - 2.12	-	0.56 0.068 **- 2.12**	79%
Aluminium (mg/L)	0.2	0.001	0.038 0.016 - 0.090	0.047 0.012-0.130	NQ	0.024 0.010-0.040	0.010 0.010-0.010	0.015 0.01 - 0.02	0.020 0.010-0.020	46%
Copper (mg/L)	2	0.001	0.009 0.004 - 0.016	0.003 <0.001-0.010	63%	<0.001 all <0.001	<0.001 all <0.001	<0.001 all <0.001	<0.001 <0.001-<0.001	>90%
Iron (mg/L)	0.3	0.005	0.135 0.006 - **0.830**	0.122 <0.005-**0.610**	9%	0.005 <0.005-0.011	0.007 <0.005-0.01	<0.005 all <0.005	0.005 <0.005-0.011	>98%

Parameters	ADWG limit	Detectable limit	Raw water (17 samples)	GAC (17 samples)	GAC % removed	0.15 m HEAD (12 samples) GAC + membrane	1.0 m HEAD (3 samples) GAC + membrane	2 m HEAD (2 samples) GAC + MEMBRANE	Avg.GAC +Membrane (14 samples)	Avg.GAC +Membrane % removed
Manganese (mg/L)	0.1	0.001	0.033 <0.001-**0.140**	0.018 <0.001-0.096	47%	0.007 0.004-0.014	0.037 0.010-0.060	0.058 0.036-0.079	0.018 0.004-0.079	44%
Lead (mg/L)	0.01	0.001	0.008 0.001-**0.031**	0.006 <0.001-**0.025**	22%	<0.001 all <0.001	<0.001 all <0.001	<0.001 all <0.001	<0.001 <0.001-<0.001	>87%
Zinc (mg/L)	3	0.001	0.038 0.018-0.070	0.013 0.002-0.031	65%	0.010 0.005-0.019	0.025 0.010-0.040	0.0075 0.007-0.008	0.012 0.005-0.040	67%

Notes:

- NQ – not quantified, this parameter has not been reduced
- Arsenic and Cadmium were not detected
- 1st row – Mean average value, 2nd row -range of value
- Values exceeding the ADWG (2011) limit are shown in bold

7.4. INGE membrane filtration with pre-treatment under gravitational head

Membrane filtration experiments were carried out using the INGE Watertechnologies AG Multibore micro-filtration membrane (INGE) (Table 3.9, Section 3.2.4) with a pre-treatment of GAC adsorption. The INGE membrane filter typically operates in an 'in-to-out' way and as such requires a pumped cross-flow configuration driving any pollutants through the membrane and back into the circulation feed water. In this study, however, the membrane had to be operated in dead-end mode rather than cross-flow mode, since there is normally no pump to circulate the water through the filter in a rainwater tank arrangement. Due to this configuration it was expected that premature, permanent blockage would occur within the membrane as pollutants enter and became trapped inside the membrane. For this reason, only GAC pre-treated feed water experiments were done to extend the operational life of the INGE membrane filter. This made it possible to test membrane pollutant removal performance, should a modified 'out-to-in' INGE Multibore membrane type filter become available.

7.4.1. Flux decline with pre-treatment

The flux decline of the INGE membrane filtration was tested with rainwater pre-treated with GAC adsorption at three different driving heads (0.3 m, 1 m, 2 m). Rainwater was collected from a domestic roof and rainwater tank (TD1) (Table 3.1, Section 3.2.1) located in Sydney (Figure 3.1, Section 3.2.1). The experimental set-up was described in section 3.3.3. The results of the flux decline are shown in Figure 7.3. The flux decline tests indicate that all driving heads generally resulted in a final stable flux of around 5 $L/m^2/hr$.

After approximately 200 to 250 hours of operation, all test converged to a similar flux decline path regardless of driving head. A higher driving head on the membrane allows an initial higher flow rate. Over the long-term all driving heads achieve the same flux.

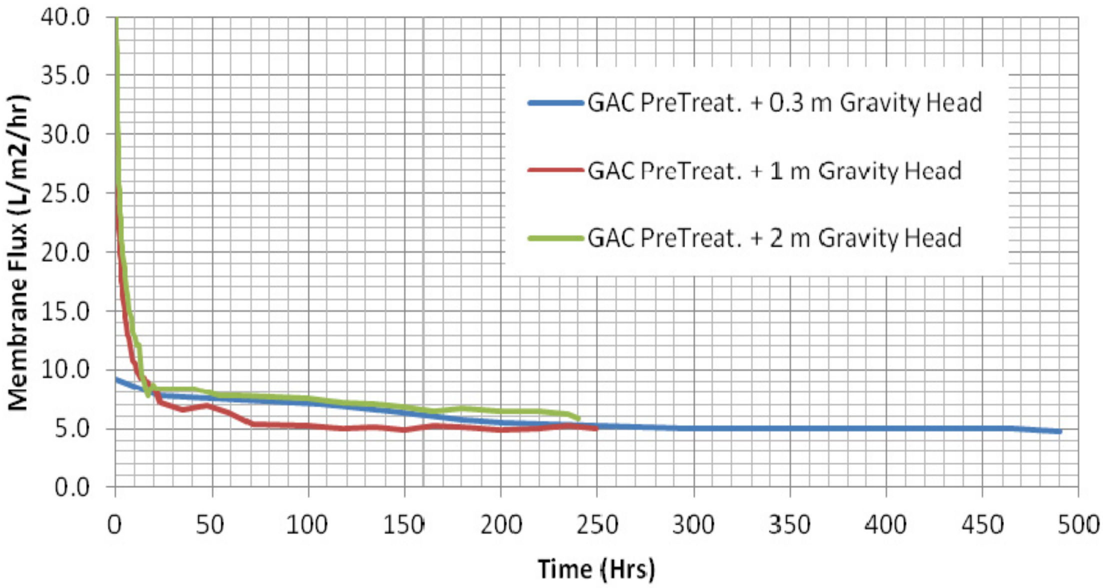


Figure 7.3 Comparison of INGE Watertechnologies AG Multibore micro-filtration membrane flux decline under various gravity heads with GAC adsorption pre-treatment.

7.4.2. Raw water quality

While monitoring the flux decline of the INGE membrane filter, grab samples of the influent and effluent of the GAC adsorption filter and the INGE membrane filtration were collected. This was done to analyse pollutant removal efficiencies of the membrane filter. Results of the parameters tested are shown in Table 7.3.

The influent water quality when experimenting with GAC pre-treatment followed by membrane filtration typically complied with most of the parameters tested. Parameters that did not comply included turbidity, microbiological (total and faecal coliforms), TOC, heavy metals (iron, manganese and lead) and ammonia.

The average turbidity of the influent raw water was 1.8 which was within the ADWG (2011) limit of 5 NTU. However one sample had a turbidity of 5.9 NTU and exceeded the prescribed limit. The total and faecal coliform counts in the rainwater samples were relatively high with an average count of 182 and 108, respectively. The ADWG (2011) recommended limits for total and faecal coliform are both less than 1 CFU/100 mL.

The heavy metals iron and manganese both contained average influent concentrations below the ADWG (2011) limits of 0.3 mg/L and 0.1 mg/L, respectively, and at 0.135 mg/L and 0.033 mg/L, also respectively. However, some individual samples exceeded these limits. The highest samples recorded for iron and manganese concentrations were 0.83 mg/L and 0.14 mg/L, respectively. The average concentration of lead was 0.008 mg/L which did not comply with the ADWG (2011) limit of 0.01 mg/L. The concentration levels of aluminium, copper, and zinc were at low to negligible concentrations.

The ADWG (2011) limits for nitrate, nitrite and ammonia are 50 mg/L, 3 mg/L and 0.5 mg/L, respectively. All samples were below these stipulated limits except for one sample of ammonia which had a concentration of 1.364 mg/L.

7.4.3. Membrane filtration with pre-treatment

Turbidity, true colour and total suspended solids

Turbidity: The ADWG (2011) recommend a limit for turbidity of 5 NTU. The turbidity of influent raw rainwater was below this limit with an average value of 1.8 NTU. The effluent of GAC adsorption pre-treatment had an average turbidity of 1.1 NTU, which was equivalent to 38% removal efficiency. The turbidity of the effluent from membrane filtration was 0.28 NTU. The overall reduction in turbidity after GAC adsorption pre-treatment followed by membrane filtration achieved an average of 84 (Table 7.3).

True Colour: The ADWG (2011) do not have a limit for true colour. The true colour of influent raw water samples varied with the raw rainwater used. One portion of influent raw water had a true colour of 1-25 PtCo units while the remainder had a true colour ranging between 185-276 PtCo units. Overall the GAC adsorption pre-treatment was able to reduce the true colour by an average of 34% with the membrane filtration providing further reduction to give a total average reduction of 93%, Table 7.3. This was generally consistent for both batches of influent raw rainwater with the high and low level of true colour.

Total Suspended Solids: The influent raw rainwater samples were already quite low with an average value of 1.4 mg/L. The GAC adsorption pre-treatment reduced the average concentration of TSS to 1.0 mg/L or 28% removal efficiency. The effluent from membrane filtration had a TSS concentration of less than 0.5 mg/L (instrument detection limits). The overall reduction in TSS using membrane filtration with GAC adsorption pre-treatment was greater than 89% (Table 7.3).

Total organic carbon

The ADWG (2011) does not recommend a limit for TOC. The average TOC concentration in the raw influent rainwater when using the membrane filtration with GAC adsorption pre-treatment was 2.66 mg/L. The pre-treatment of GAC adsorption reduced the average concentration of TOC to 0.95 mg/L or 64% removal efficiency. Membrane filtration resulted in a concentration of 1.04 mg/L. The reduction in TOC using membrane filtration with pre-treatment of GAC adsorption was an average of 60% (Table 7.3).

Anions and cations, total dissolved salts, water hardness and pH

GAC adsorption pre-treatment and membrane filtration had little to no effect when removing anions, cations, or reducing the total dissolved salts and water hardness. There was also negligible change to pH.

Orthophosphate, nitrate, nitrite and ammonia

The concentration of nitrate, nitrite and ammonia in all raw water samples were below the ADWG (2011) limits except one sample of the later (ammonia) being at 1.364 mg/L. This was over twice the ADWG (2011) limit of 0.5 mg/L. Orthophosphate, nitrate and ammonia concentrations were all reduced when employing GAC adsorption pre-treatment by 42%, 38% and 36%, respectively. Membrane filtration was able to further reduce orthophosphate, nitrate and ammonia concentrations with a total reduction of 65%, 27% and 42%, respectively (Table 7.3). Nitrite improved negligibly when utilising both GAC adsorption and membrane filtration. This was most likely due to the very low influent concentration which was often already near detectable limits.

Heavy metals

Aluminium: The ADWG (2011) recommend an aluminium concentration limit of 0.2 mg/L. The influent rainwater samples were all below the recommended limit with an average influent concentration of 0.038 mg/L. The GAC adsorption pre-treatment did not improve the influent water samples and provided an average concentration of 0.047 mg/L. This could be due to aluminium leaching out of the GAC filter after initial removal by the filter. Following pre-treatment, membrane filtration successfully reduced the aluminium concentration levels to an average of 0.020 mg/L which is an overall reduction of 45% (Table 7.3).

Copper: The ADWG recommend a copper concentration limit of 2 mg/L. All water samples were below the ADWG limit with the highest influent concentration being 0.016mg/L. The GAC adsorption pre-treatment was able to reduce the average copper concentration by 63% to 0.003 mg/L. Membrane filtration achieved over 85% removal with a reduction in the average concentration to 0.002 mg/L with some samples at the detectable limits being less than 0.001 mg/L (Table 7.3).

Iron: The ADWG (2011) recommended an iron concentration limit of 0.3 mg/L. Some influent raw rainwater samples almost exceeded the recommended limit with an average value of 0.135 mg/L. GAC adsorption pre-treatment achieved a reduction in iron of 9% to an average concentration of 0.122 mg/L. Membrane filtration reduced the concentration of iron in the pre-treated rainwater to an average of 0.010 mg/L with an overall reduction of more than 92% (Table 7.3).

Lead: The ADWG (2011) recommend a limit for lead concentration of 0.01 mg/L. Some of the influent raw rainwater samples exceeded the recommended limit with an average influent concentration of 0.008 mg/L. GAC adsorption pre-treatment achieved a reduction in lead concentration of 22% to an average concentration of 0.006 mg/L. Membrane filtration reduced lead concentrations to less than 0.001 mg/L (instrument detection limits). The overall reduction in lead using membrane filtration with GAC adsorption pre-treatment was over 87% (Table 7.3).

Manganese: The ADWG (2011) recommend a manganese concentration limit of below 0.01 mg/L. Although the average influent concentration was below this prescribed limit at 0.033 mg/L, some samples were recorded above this concentration at 0.140mg/L. The GAC adsorption pre-treatment was able to reduce the average manganese concentration by 47% to 0.018 mg/L. Membrane filtration achieved no further improvement beyond the pre-treatment (Table 7.3).

Zinc: The ADWG (2011) recommend a zinc concentration limit of 3 mg/L. All water samples were significantly below the ADWG limit with the highest sampled influent concentration at 0.038 mg/L. GAC adsorption pre-treatment was able to reduce the average zinc concentration by 65% to 0.013 mg/L. Membrane filtration achieved more than 90% reduction in zinc concentration to 0.004 mg/L (Table 7.3).

All other heavy metals were well within the ADWG (2011) recommended limits. The concentrations of arsenic, boron, cadmium, chromium, mercury, molybdenum, nickel, selenium and silver were all negligible and less than 0.001 mg/L.

Total coliform and Faecal coliform

The ADWG (2011) recommend limits of <1 CFU/100 mL for faecal and total coliform counts. All influent rainwater samples exceeded the recommended limits for faecal and total coliform counts. GAC adsorption pre-treatment did not show any improvement in coliform counts. However, after membrane filtration, the total coliform count fell by 53% and the faecal coliforms count decreased to less than 1 CFU/100 mL (Table 7.3).

Summary

INGE membrane treatment when using the GAC adsorption pre-treatment strategy successfully reduced the concentrations of all water quality parameters to below that specified in the ADWG (2011). The exception to this scenario concerned the faecal and total coliforms. Further disinfection treatment is required to either reduce this further or guarantee complete removal.

Table 7.3 INGE membrane filtration with GAC adsorption pre-treatment.

Parameters	ADWG limit	Detectable limit	Raw water (17 samples)	GAC (17 samples)	GAC % removed	0.3m head (8 samples)	1.0m head (1 samples)	2m head (1 samples)	Avg. GAC +Membrane (10 samples)	Avg. GAC +Membrane % removed
						GAC + membrane	GAC + membrane	GAC + membrane		
pH	6.5 - 8.5	3.0 - 10	7.01 6.59 - 7.55	6.97 6.5 - 7.47	NQ	6.96 6.61 – 7.08	6.94	6.61	6.92 6.61 - 7.08	NQ
Total Suspended Solids (mg/L)	-	0.5	1.4 <0.5 - 5.0	1.0 <0.5 - 3.0	28%	<0.5 <0.5 - <0.5	<0.5	-	<0.5 <0.5 - <0.5	<89%
Turbidity (NTU)	5	0.2	1.8 0.5 - **5.9**	1.1 0.2 - 4.5	38%	0.2 <0.2- 1.0	1.0	<0.2	0.28 <0.2 - 1.0	84%
True Colour (PtCo)	-	1	76 1 - 276	42.6 1 - 174	44%	1 1 - 1	38	6.0	5.2 1 - 38	93%
Total Dissolved Salts (mg/L)	-	0.1	65 44 - 123	63 46 - 107	NQ	55 51 – 58	105	105	65 51 - 105	NQ
Water Hardness (mg/L CaCO3)	200	0.1	21.6 5.3 - 49	18.2 4.8 - 39	15%	6 5- 7	39	35	12.2 5 - 39	43
Sodium (mg/L)	-	0.05	10.7 2.3 - 16.5	10.8 2.4 - 15.5	NQ	12.3 10.8 – 13.8	13.7	9	12.1 9 - 13.8	NQ
Potassium (mg/L)	-	0.001	2.45 0.4 - 7.4	2.35 0.4 - 7.2	3%	0.6 0.5 – 0.6	7.4	6	1.8 0.5 - 7.4	25

Parameters	ADWG limit	Detectable limit	Raw water (17 samples)	GAC (17 samples)	GAC % removed	0.3m head (8 samples)	1.0m head (1 samples)	2m head (1 samples)	Avg. GAC +Membrane (10 samples)	Avg. GAC +Membrane % removed
						GAC + membrane	GAC + membrane	GAC + membrane		
Calcium (mg/L)	-	0.05	7.0 1.0 - 16.8	5.9 0.9 - 13.1	15%	1.0 0.8 – 1.5	13.1	12	3.31 0.8 - 13.1	52
Magnesium (mg/L)	-	0.05	2.2 0.5 - 3.7	2.17 0.5 - 4.4	1%	3.1 2.6 – 3.4	1.5	1	2.73 1 - 3.4	NQ
Orthophosphate (mg/L P)	-	0.005	0.038 0.004 - 0.152	0.022 0.005-0.051	42%	0.011 0.003 – 0.014	0.031	-	0.013 0.003 - 0.031	65%
Nitrate (mg/L N)	50	0.005	0.538 0.02 - 1.022	0.330 0.005 - 0.862	38%	0.470 0.022 – 0.831	0.120	<0.005	0.389 <0.005 - 0.831	27%
Nitrite (mg/L N)	3	0.005	0.008 <0.005-0.038	0.011 <0.005-0.081	NQ	0.006 <0.001 – 0.012	0.031	<0.005	0.008 <0.005 - 0.031	NQ
Ammonia (mg/L N)	0.5	0.005	0.142 <0.005-**1.364**	0.089 <0.005-0.306	36%	0.101 0.048 – 0.274	0.004	<0.005	0.082 <0.005 - 0.274	42%
Total Coliforms (cfu/100 ml)	1	1	**182** **20 - 1000**	**101** **20 - 420**	44%	**103** **40 – 170**	<1	<1	**84** <1 - **170**	53%
Faecal Coliforms (cfu/100 ml)	1	1	**108** <1 - **1000**	**50** <1 - **330**	54%	<1 <1 – **1**	<1	<1	<1 <1 - **1**	>90%
Total Organic Carbon (mg/L)	-	0.001	2.66 2.10 - 3.73	0.95 0.42 - 1.80	64%	0.69 0.40 - 1.25	3.86	-	1.04 0.40 - 3.86	60%

Parameters	ADWG limit	Detectable limit	Raw water (17 samples)	GAC (17 samples)	GAC % removed	0.3m head (8 samples) GAC + membrane	1.0m head (1 samples) GAC + membrane	2m head (1 samples) GAC + membrane	Avg. GAC +Membrane (10 samples)	Avg. GAC +Membrane % removed
Aluminium (mg/L)	0.2	0.001	0.038 0.016 - 0.090	0.047 0.012-0.130	NQ	0.021 0.010 – 0.031	0.020	0.020	0.020 0.02 - 0.031	45%
Copper (mg/L)	2	0.001	0.009 0.004 - 0.016	0.003 <0.001-0.010)	63%	<0.001 <0.001 – 0.002	<0.001	0.004	0.001 <0.001 - 0.004	> 85%
Iron (mg/L)	0.3	0.005	0.135 0.006 - **0.830**	0.122 <0.005-**0.610**	9%	<0.001 <0.001 – 0.001	0.040	0.050	0.010 <0.001 - 0.050	>92%
Manganese (mg/L)	0.1	0.001	0.033 <0.001-**0.140**	0.018 <0.001-0.096	47%	0.005 0.003 – 0.011	0.060	0.090	0.019 0.003 - 0.090	42%
Lead (mg/L)	0.01	0.001	0.008 0.001-**0.031**	0.006 <0.001-**0.025**	22%	<0.001 <0.001 – <0.001	<0.001	0.001	<0.001 <0.001 - <0.001	>87%
Zinc (mg/L)	3	0.001	0.038 0.018-0.070	0.013 0.002-0.031	65%	0.002 <0.001 – 0.003	0.010	0.010	0.004 <0.001 - 0.010	>90%

Notes:

- NQ – not quantified, this parameter has not been reduced
- Arsenic and Cadmium were not detected
- 1st row – Mean average value, 2nd row -range of value
- Values exceeding the ADWG (2011) limit are shown in bold

7.5. Conclusion

Micro-filtration clean membrane flux tests were examined under various trans-membrane pressures to determine their suitability as a membrane filter to treat raw rainwater.

The stainless steel membrane (Steriflow Filtration Systems) is a strong and long-lasting filter. However, it requires a relatively high driving head which is not available in a typical residential rainwater tank.

The INGE membrane filter and Ultra Flo membrane filter were also tested. Both were polymeric hollow fibre membranes. The INGE membrane filter contains several bores within each fibre while the Ultra Flo contains a number of individual hollow fibres. These filters could operate at a reasonable flux within driving heads of 2 m which constitutes the upper limit of what is available in a typical residential rainwater tank.

The flux decline of the Ultra Flo hollow fibre micro-filtration membrane was monitored over various driving heads using either raw rainwater or rainwater pre-treated with GAC adsorption. The variation in flux over time indicated that after approximately 140 hours operation all configurations converged to a similar flux decline path regardless of driving head or whether GAC adsorption pre-treatment was used. GAC adsorption pre-treatment allows a higher initial flux to occur, with a slower flux decline for the first 140 hours. Similarly a higher driving head had an initial higher flux. It is observed that the long-term flux decline of the membrane was not stable after 450 hours and continued to slowly decline. After 2600 hours the flux of the membrane decreased to an average of 1.2 L/m^2/hr.

The INGE membrane filter performed in a similar manner. The flux decline of the INGE membrane filter was monitored over various driving heads using rainwater pre-treated with GAC adsorption. After operating for approximately 200 to 250 hours, all experiments converged to a similar flux decline path regardless of driving head being used. A higher driving head on the membrane encourages an initial higher flow rate.

One drawback of the INGE membrane filter is that it should typically operate under a cross-flow configuration, which is not possible if there is no pump to circulate the water through the filter in a rainwater tank arrangement. In this study it was operated under dead-end mode with pre-treated feed water only to extend the operation life of the membrane. Consequently, the membrane's ability to remove pollutants could be tested, should a modified 'out-to-in' INGE Multibore membrane type filter become available.

The Ultra Flo membrane filtration without pre-treatment did well when removing turbidity, leading to below detectable levels and compliance with the ADWG (2011). It also removed much of the total and faecal coliforms to detectable limits. It did not, however, remove TOC.

Finally, Ultra Flo membrane filtration with GAC adsorption pre-treatment was able to reduce the concentration of all water quality parameters to below that specified in the ADWG (2011). The exception in this case was faecal and total coliforms. Further disinfection treatment is required to either reduce this further or guarantee complete removal. The INGE filter performed in a similar way to the Ultra Flo membrane filter.

Chapter 8

University of Technology, Sydney

Faculty of Engineering & Information Technology

8. Stormwater Harvesting Pilot Scale Plant

8.1. Introduction

Kogarah Council, a local government authority in Sydney, introduced the Carlton Industrial Sustainable Water Program (CISWP), in part to minimise potable water consumption in the Carlton industrial area. One major facet of the CISWP was the design and installation of a stormwater harvesting plant at Carlton. The development of this stormwater harvesting plant is described in Appendix B. Pilot scale experiments were undertaken to determine the performance of various media and membrane filtration systems at the stormwater harvesting plant at Carlton (see Section 3.3.4). The experiments helped in the development of a rainwater treatment system which included stormwater harvesting (Section 8.5)

8.2. Laboratory scale experiments

Initially, laboratory scale experiments investigated the effects of various pre-treatments prior to membrane filtration using raw stormwater from the Carlton stormwater harvesting plant (Section 3.3.4). The results were used for comparison against the pilot scale Steriflow membrane filtration system experiments. These experiments were undertaken by Mohammad Abu Hasan Johir (Kus, Johir et al., 2012) and included here to provide information and background to the pilot scale experiments reported in this chapter.

8.2.1. Methodology

Raw water samples were collected from a stormwater harvesting plant located at the Lower West Street Reserve, Carlton, Sydney (Section 3.3.4). Different types of pre-treatment methods were used to treat raw stormwater as follows:

1. Flocculation: Flocculation was carried out using $FeCl_3$ as flocculant at a dose of 30 mg l^{-1}. The optimum flocculant dose (30 mg l^{-1}) was pre-determined using standard jar test. $FeCl_3$ were added into beakers. The samples were stirred rapidly for 1 min at 130 rpm to represent rapid mixing followed by 30 min of slow mixing at 30 rpm representing flocculation. This in turn was followed by a final 30 min of non-stirring to allow the flocs to settle.
2. GAC filtration: Short-term (5 h) GAC filtration experiment was carried out using a filtration velocity of 5 m h^{-1}. The particle size of GAC used in this study was 0.30–0.76 mm. Other properties are given in Table 8.1. The height of the filter media inside the filter column was 80 cm.
3. In line flocculation and fibre filtration: Filter column packed with fibre filter at a packing density of 115 kg m^{-3} was operated at a filtration velocity of 20 m h^{-1} to evaluate the fibre filter's efficiency. $FeCl_3$ at a dose of 15 mg l^{-1} was used as in-line flocculant.

Membrane filtration experiments were conducted initially using the laboratory scale Steriflow stainless steel filtration system (Section 3.3.4). The membrane has a surface area of 0.03 m^2 and pore size of 0.3 µm.

The effectiveness of the various treatments was studied in terms of turbidity, heavy metal concentration and organic removal.

8.2.2. Results

Laboratory experiments were undertaken to investigate the effect of various pre-treatments prior to membrane filtration. Three different treatment methods were examined: i) flocculation using $FeCl_3$, ii) GAC filtration, and iii) in-line flocculation-fibre media filtration.

The average TOC of the influent stormwater was about 5.35 mg/L while the average TOC removal by membrane filtration (MF) alone was only about 10%. MF alone cannot remove the TOC due to its large pore size (0.3 μm). The marginal removal sometimes observed is due to adsorption of organics onto the membrane. The use of pre-treatment improved the TOC removal efficiency from 10% to 90% (Table 8.1). Of the three pre-treatment strategies the GAC filter resulted in the highest TOC removal efficiency (88%, Table 8.1). The next highest TOC removal efficiency was with in-line flocculation using $FeCl_3$ followed by fibre filtration. Flocculation alone had a TOC removal efficiency of 55% (Table 8.1).

The turbidity following different treatments ranged between 0.5-1.2 NTU (Table 8.1) the exception being the GAC filtration where the effluent turbidity was 5 NTU. This may be due to the fact the particles are too small to be removed without any flocculation.

The flux decline of raw water (without pre-treatment) was between 35-40%, whereas after pre-treatment it fell to approximately 8-30% (Figure 8.1). In terms of flux decline in-line flocculation-fibre media filtration provided the lowest flux decline of the MF (8-9%) followed by GAC (24-30%) and flocculation (28-30%). The GAC filter demonstrated higher TOC removal efficiency but had a higher rate of fouling of MF compared to in-line flocculation with fibre filtration. This was because the GAC filter was not able to remove colloidal particles from the water, resulting in a higher turbidity value (5 NTU) compared with in-line flocculation fibre filtration which provided better removal of turbidity (0.5 NTU). The membrane flux was restored by chemical cleaning (with NaOH solution at pH of 12 for 2 minutes) and one minute backwash using filtrate water.

Table 8.1 Filtrated water quality after different pre-treatments (raw water turbidity = 25.3 NTU; TOC = 5.35 mg/L).

Pre-treatment option	**Turbidity (NTU)**	**TOC removal efficiency (%)**
Flocculation (FeCl3=15mg/L)	1.23	55.7
GAC filtration (particle size=0.3-0.67mm, velocity=5m/h)	5.0	88.2
In-line floculation-fibre filtration (FeCl3=15 mg/L, v=20 m/h)	0.5	62.0
Flocculation (FeCl3=15mg/L) with MF as post-treatment	0.12	58.1
Microfiltration alone (MF, pore size=0.3 μm)	0.13	10
GAC filtration (particle size=0.3-0.67mm, velocity=5m/h) with MF as post-treatment	0.10	90.0
In-line floculation-fibre filtration (FeCl3=15 mg/L, v=20 m/h)+MF	0.10	62.0

Notes:

- Values exceeding the ADWG (2011) limit are shown in bold

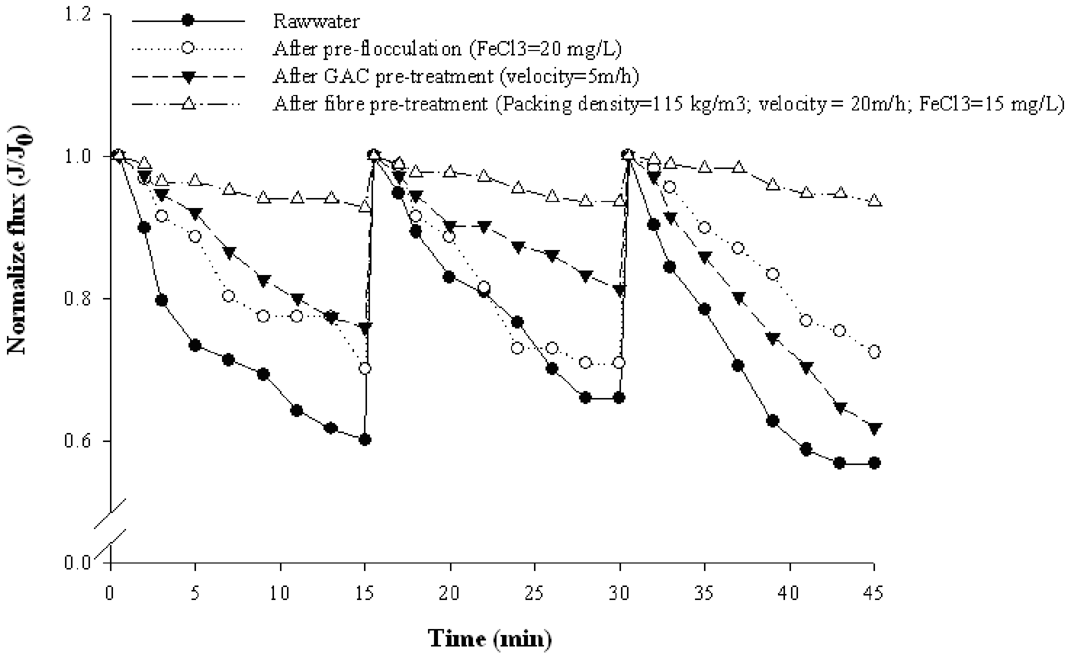


Figure 8.1 Comparison between pre-treatments and cross-flow microfiltration (stormwater; membrane area = 0.03 m^2; pore size = 0.3 µm; cross flow velocity=0.5 m/s; pure water flux at IP=125±3 and OP=100±3 kPa is 0.44 $m^3/m^2.h$; IP=Inlet pressure and OP=outlet pressure)

Table 8.2 Turbidity, TOC and Heavy Metals results of Steriflow filtration system

Parameter	Raw (mg/L)	Steriflow Filtration (mg/L)	% Removal	ADWG (2011) (mg/L)
Aluminium	0.624 - 1.348 1.077	0.005 - 0.009 0.007	99%	<0.2
Arsenic	0.002 - **0.008** 0.005	<0.001 - 0.002 0.002	62%	<0.007
Chromium	0.002 - 0.002 0.002	0.002 - 0.002 0.002	0%	<0.05
Copper	0.027 - 0.050 0.041	0.004 - 0.014 0.008	80%	<2
Iron	1.66 - 3.31 2.684	<0.01 - 0.02 0.013	99%	<0.3
Manganese	0.042 - 0.170 0.109	0.003 - 0.139 0.063	42%	<0.1
Lead	0.009 - 0.015 0.013	ND	>20%	<0.01
Selenium	<0.001 - 0.002 0.002	ND	0%	<0.01
Zinc	0.056 - 0.109 0.080	<0.001 - 0.017 0.007	89%	<3

Notes:

- N/A – data on this parameter is not provided from this source
- ND – not detected
- Values exceeding the ADWG (2011) limit are shown in bold

Table 8.3 Heavy Metals results of Steriflow filtration system with GAC adsorption as pre-treatment

Parameter	Raw (mg/L)	Pre-Treated (GAC Adsorption) (mg/L)	% Removal	Steriflow Filtration + GAC (mg/L)	% Removal	ADWG (2011) (mg/L)
Turbidity	15.90- 48.40 26.06	1.59 - **5.73** 3.78	85%	0.21 - 0.78 0.40	98%	<5
TOC	6.547 – 12.556 7.681	0.611 - 0.812 0.680	91%	0.001 - 0.001 0.001	99%	-
Total Phosphorus	0.75 - 0.75 0.75	0.08 - 0.41 0.233	69%	0.03 - 0.06 0.05	93%	<5
Total Nitrogen	6.47 - 6.47 6.47	2.88 - 4.02 3.31	49%	2.34 - 2.51 2.45	62%	<10
Aluminium	0.25 - 0.25 0.25	0.038 - 0.194 0.121	52%	<0.005 - 0.062 0.024	90%	<0.2
Arsenic	0.004 - 0.004 0.004	0.001 - 0.002 0.002	50%	0.001 - 0.002 0.002	50%	<0.007
Chromium	0.002 - 0.002 0.002	0.001 - 0.002 0.001	50%	0.001 - 0.002 0.001	50%	<0.05
Copper	0.025 - 0.025 0.025	0.019 - 0.038 0.027	NQ	0.002 - 0.010 0.005	80%	<2
Iron	1.277 - 1.277 1.277	0.082 - 0.291 0.212	83%	<0.005 - 0.005 0.005	99%	<0.3
Manganese	0.478 - 0.478 0.478	0.048 - 0.098 0.066	86%	0.035 - 0.084 0.051	89%	<0.1
Lead	0.003 - 0.003 0.003	0.002 - 0.008 0.004	NQ	<0.001 - 0.003 0.0017	43%	<0.01
Zinc	0.058 - 0.058 0.058	0.017 - 0.038 0.026	55%	0.001 - 0.004 0.002	97%	<3

Notes:

- NQ – not quantified, this parameter has either not been detected or the percentage removed has not been reduced
- Values exceeding the ADWG (2011) limit are shown in bold

8.3. Steriflow membrane filtration

8.3.1. Membrane filtration without pre-treatment

Experiments were also carried out using pilot-scale Steriflow stainless steel membrane filtration at Carlton. The methodology for these experiments is explained in more detail in Section 3.3.4.

TOC

The concentration of TOC in the influent raw stormwater feed was between 3.94-9.73 mg/L (Figure 8.2). Although MF membranes do not normally remove TOC without any other pre-treatment, the Steriflow system could reduce TOC levels in the filtrate by 45% to between 1.49-6.15 mg/L. This could be partly due to the high removal of turbidity from the feed with which some of the organic matter was associated. The TOC reduction by the membrane filter could also have been due to the adsorption onto the membrane.

Turbidity

The influent raw stormwater contained turbidity levels in the range of 72-575 NTU (Figure 8.3). The turbidity levels of influent raw stormwater during this experiment were high and coincided with a period of rainfall and heavy stormwater runoff. Steriflow membrane filtration without any pre-treatment achieved an effluent filtrate turbidity of between 0.79-0.99 NTU which was well below the ADWG (2011) limit of 5 NTU.

Heavy Metals

The influent raw stormwater itself had generally low concentrations of heavy metals (Table 8.2). No traces of cadmium or mercury were detected in the samples. The Steriflow membrane filtration performed effectively with significant reductions in removing most heavy metals. The concentrations of all sampled heavy metals in the effluent were below the ADWG (2011) limits. The removal rates for aluminium, copper, iron and zinc were high (Table 8.2). Lead was removed to below detection limits. Steriflow filtration provided a smaller improvement regarding manganese with an average reduction of 42%. The sampling indicated that there was minor removal of chromium and selenium although the concentrations of both these in the raw stormwater were already very low. The removal of heavy metals by the membrane filtration may have been due to the fact that most heavy metals would have been associated with sediment particles.

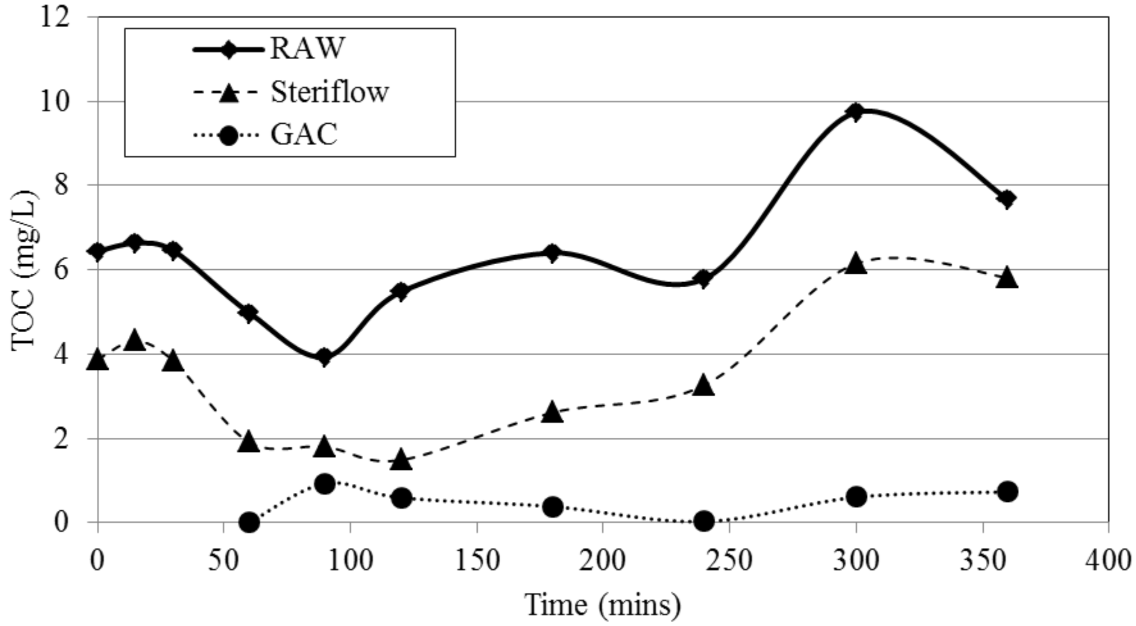


Figure 8.2 TOC with Steriflow membrane filter results followed by post-treatment with GAC

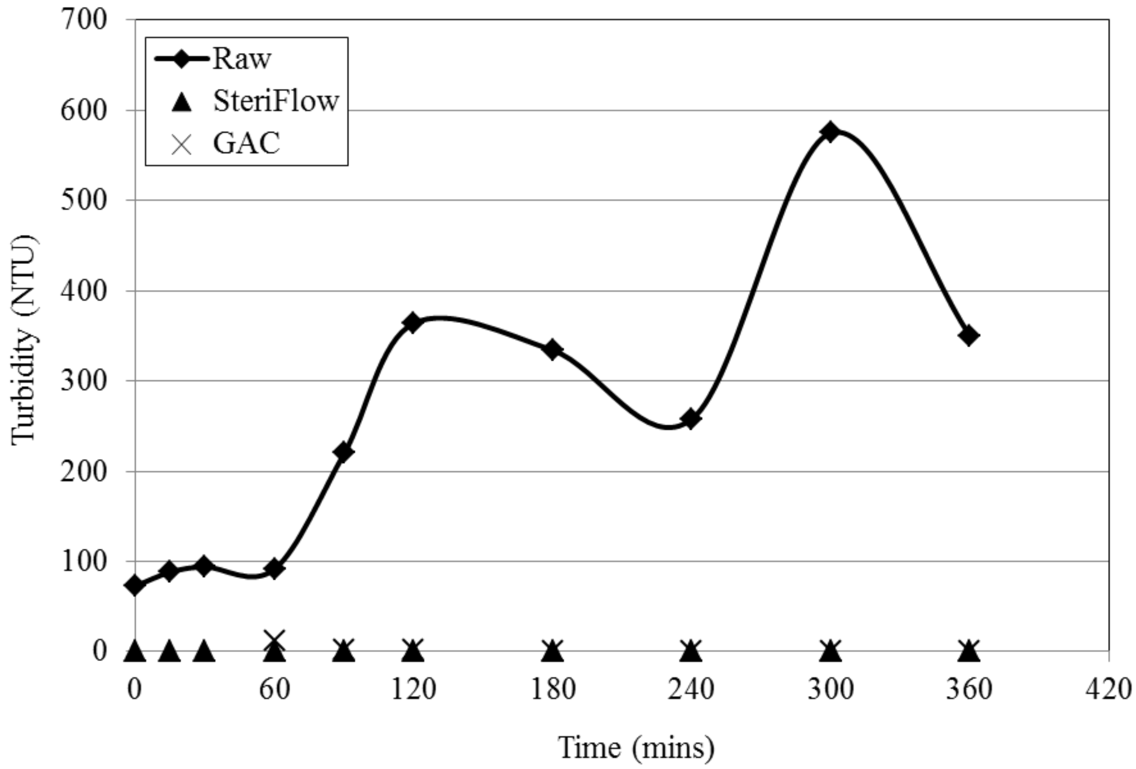


Figure 8.3 Turbidity with Steriflow membrane filter treatment followed by post-treatment with GAC

8.3.2. Steriflow membrane filtration with post-treatment of GAC adsorption

Experiments were also carried out using pilot-scale stainless steel membrane filtration and a post-treatment of GAC adsorption at Carlton. The methodology used in this experiment is described in Section 3.3.4. The results of the experiment with GAC adsorption as post-treatment following SteriFlow membrane treatment demonstrate that the GAC treated stormwater effectively reduced TOC concentrations in the influent feed (feed from SteriFlow membrane filtration) from 1.49-6.15 mg/L (with an average of 3.51 mg/L) to between 0.61-0.81 mg/L and an average concentration of 0.68 mg/L (Figure 8.2). The GAC filter did not provide any further improvement in the turbidity level following Steriflow membrane filtration which was already well below the 5 NTU ADWG (2011) limit (Figure 8.3). The GAC filter provided a small additional improvement to the removal of heavy metals.

Membrane flux decline

The experiment conducted with the Steriflow membrane system which treated raw stormwater without any pre-treatment indicated a large decline in flux. The influent raw stormwater's turbidity levels during this experiment were high, ranging from 72-575 NTU (Table 8.3), which coincided with a period of rainfall and heavy stormwater runoff. The fluxes recorded at the initial stages of the experiment were consistently between 50-55 L/m²/hr and declined continuously over the 6 hr duration of the experiment to a final flux of 37 L/m²/hr.

The Steriflow membrane filtration experiment where the raw stormwater was pre-treated with GAC adsorption showed negligible flux decline with the flux generally maintaining a steady 60-66 L/m²/hr throughout. This is likely due to the pre-treated influent having a low turbidity in the range of 1.59-5.73 NTU (Table 8.2). The built-in back pulsing of the membrane system (Section 3.3.4) every 3 seconds for 0.08 seconds and the 1 second back flush every 4 minutes also helped achieve a small decline in flux. The operation of membranes in practice demonstrates that pre-treating the influent feed water, in this case stormwater, allows the membrane system to operate at a higher flux for longer without cleaning or maintenance. Pre-treatment removes the particulate matter and fine sediments that can cause premature membrane blockage.

8.3.3. Steriflow membrane filtration with pre-treatment of GAC adsorption

Experiments were also carried out using pilot-scale stainless steel membrane filtration and a pre-treatment of GAC adsorption filtration at Carlton. The methodology concerning this experiment is explained in Section 3.3.4.

TOC

GAC adsorption filter-treated stormwater effectively reduced the TOC in the influent feed to between 0.61-0.81 mg/L and to an average concentration of 0.68 mg/L (Table 8.3). Steriflow membrane filtration further reduced the pre-treated influent to below detectable levels. This could be partly due to the high removal of turbidity from the feed which partly consisted of particulate organic matter. The membrane filter's reduction of TOC could also have been due to its adsorption onto the membrane.

Turbidity

The GAC adsorption filter effectively pre-treated the stormwater and reduced the turbidity of the feed water to concentrations approximating the ADWG (2011) limit of 5 NTU, in fact ranging from 1.59-5.73 NTU. Steriflow membrane filtration provided a further 98% reduction in the turbidity level with effluent samples in the range of 0.21-0.78 NTU, which are well below the ADWG (2011) limit (Table 8.3).

Heavy Metals

All raw stormwater feed samples complied with the ADWG (2011) limits except for iron which had an average value of 1.277 mg/L and manganese with an average value of 0.478 mg/L (Table 8.3). It was observed that the GAC pre-filter performed effectively with significant reductions in removing most heavy metals. The raw stormwater influent levels of the heavy metals aluminium, arsenic, iron, manganese and zinc were reduced by 52%, 50%, 83%, 86% and 55%, respectively. The GAC pre-filter had negligible removal rates for copper and lead, however, all influent and effluent sample concentrations of these two metals were below the ADWG (2011) limits of 2 mg/L and 0.01 mg/L, respectively (Table 8.3).

Steriflow membrane filtration provided further reductions in most of the heavy metals with reductions in aluminium, copper, iron, manganese, lead and zinc being 90%, 80%, 99%, 89%, 43%, and 97%, respectively. It did not provide any further reduction in arsenic, although the concentration of this was already close to instrumental detectable levels (Table 8.3). No traces of cadmium, chromium, selenium, silver or mercury were detected in the influent samples.

Total Phosphorus (TP) and Total Nitrogen (TN) Analysis

The GAC adsorption pre-filter worked effectively in reducing the average TP influent feed of 0.75 mg/L by 69% to an average of 0.233 mg/L. Steriflow membrane filtration further reduced the concentration of TP by 93% to an average of 0.05 mg/L. All samples complied with the ADWG (2011) limit of 5 mg/L (Table 8.3). The GAC adsorption pre-filter also performed effectively in reducing the average TN influent feed of 6.47 mg/L by 49% to an average of 3.31 mg/L. The Steriflow system further reduced the concentration of TN by 62% to an average of 2.45 mg/L (Table 8.3). The removal of TN is most effectively done using biological removal processes and systems except for the Steriflow filtration system which was not able to achieve high removal rates. All samples, however, still complied with the ADWG (2011) limit of 10 mg/L.

8.4. Staged media filtration for stormwater treatment

This study assessed how well filtration performed using different media, specifically GAC, sand and anthracite to determine their effectiveness as pre-treatment strategies in removing suspended solids, organics and heavy metals from stormwater. This was followed by Ultra Flo membrane filtration as used at Carlton in Sydney. The methodology for these experiments is outlined in Section 3.3.4.

TOC

The details of the concentration of TOC in the influent stormwater are summarised in Table 8.4. The treatment train of the GAC filter column followed by Ultra Flo membrane filtration (see Figure 3.26) demonstrated that the GAC treatment could successfully reduce the influent TOC concentrations by more than 97%, even down to detectible limits for some samples (Table 8.4 and Figure 8.4). The GAC adsorbed a majority of organic matter. The submerged membrane filter system used as a post-treatment following the GAC filter resulted in negligible improvement in TOC removal. The water quality of stormwater treated with media filtration (GAC) is similar to raw rainwater collected off roofs. This raw rainwater can be stored in the same storage tank as GAC treated stormwater.

Two other treatment trains were tested. One treatment train was an anthracite filter column followed by a GAC filter column and then by membrane filtration (Figure 3.27). The other treatment train was an anthracite filter column followed by a sand column, then a GAC filter column and this in turn followed by membrane filtration (Figure 3.28). The addition of an anthracite filter column and a sand filter did not provide any additional benefit to the overall removal of TOC. The average TOC removal rates were 10.8% and 15.2% for anthracite filter and the sequence of anthracite and sand filters, respectively. These amounts correspond to a reduction in concentration of 4.1 mg/L and 3.9 mg/L (Table 8.4, Figures 8.5 and 8.6). While the sand and anthracite filters mainly removed the suspended solids, they had only a minimal ability to adsorb organic matter.

Table 8.4 TOC results (based on 7 samples taken daily for 3 consecutive days)

Treatment Train	Influent (mg/L)	Anthracite (mg/L)	% Removal	Sand (mg/L)	% Removal	GAC (mg/L)	% Removal	Membrane (mg/L)	% Removal
GAC Filter and membrane post-treatment	2.3–6.9 4.6	NA	NA	NA	NA	N.Q.–1.0 0.1	>97%	N.Q.–1.0 0.1	>97%
Anthracite + GAC and membrane post-treatment	2.3–6.9 4.6	2.3–6.4 4.1	10.8%	NA	NA	N.Q.–1.7 0.1	>97%	N.Q.–1.7 0.1	>97%
Anthracite + sand + GAC and membrane post-treatment	2.2–6.6 4.6	2.3–6.4 4.1	10.8%	2.1–6.3 3.9	15.2%	N.Q.–0.9 0.1	>97%	N.Q.–0.8 0.1	>97%

Notes:

- N.Q. – not quantified, this parameter has not been reduced
- NA – not applicable

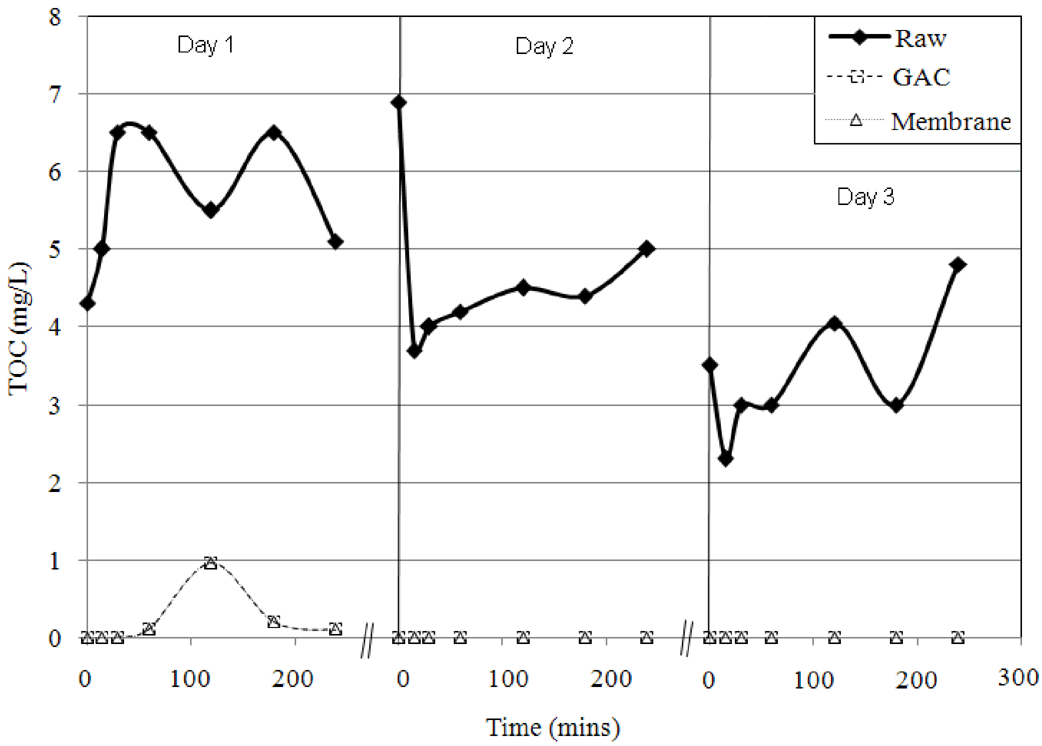


Figure 8.4 TOC results with GAC filter followed by membrane filter filtration

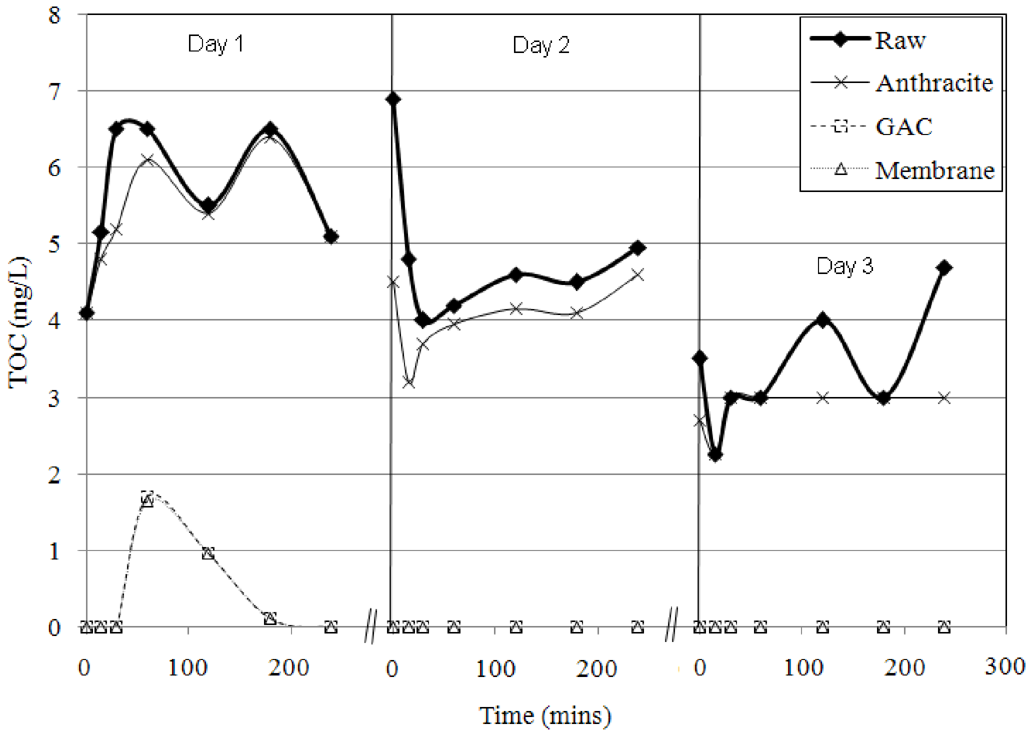


Figure 8.5 TOC results with anthracite, GAC and membrane filter treatment

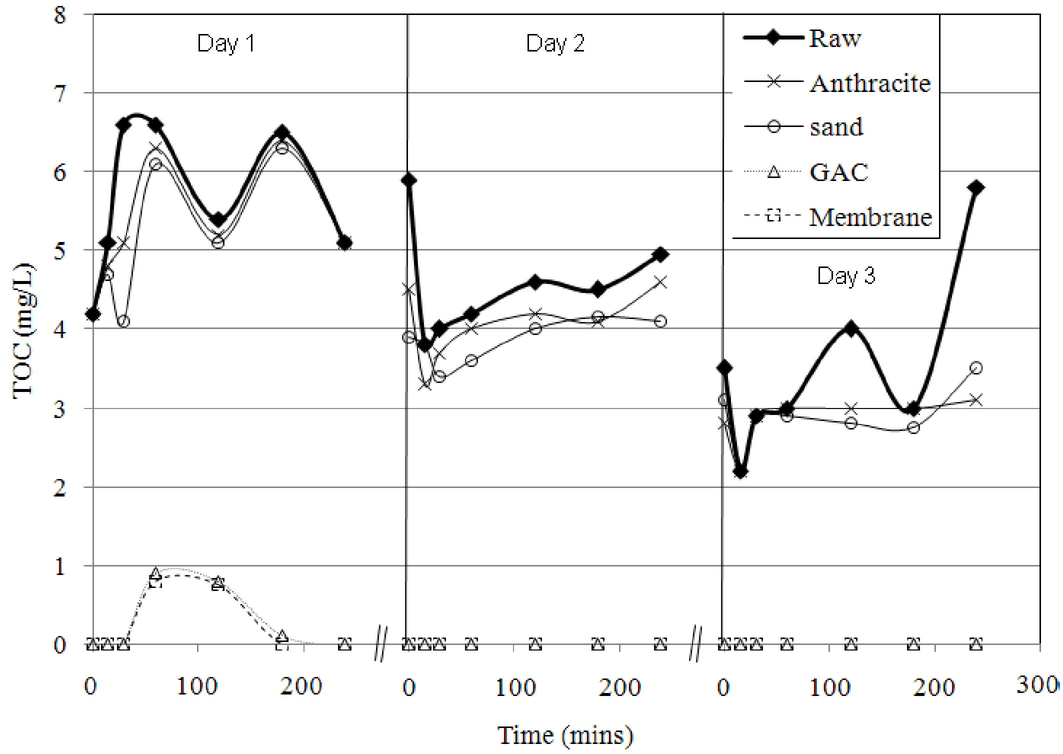


Figure 8.6 TOC results with anthracite, sand, GAC and membrane filter treatment

Turbidity

The details for the concentration of turbidity in the influent stormwater are provided in Section 3.3.4. The treatment train using the GAC filter column followed by membrane filtration (Figure 3.26) was able to reduce turbidity by more than 99% (Table 8.5 and Figure 8.7). The GAC filter by itself successfully reduced turbidity to an average of 84%. The membrane had a pore size of 0.1 µm, which removed practically all the suspended matter. The average turbidity levels of influent raw water, and GAC filter effluent was 29.2 NTU and 4.5 NTU, respectively. The turbidity following membrane filtration was below detection levels. The GAC treatment and membrane treatment both achieved turbidity levels below the ADWG (2011) limit of 5 NTU.

The addition of the anthracite filter in the treatment train (anthracite filter column followed by a GAC filter column and then followed by membrane filtration) (Figure

3.27), achieved an average turbidity removal efficiency of 71%. In fact it reduced it from 28.9 NTU to 8.5 NTU (Table 8.5 and Figure 8.8). This allowed the GAC filter to reduce the turbidity down to 4.2 NTU. Membrane filtration again reduced the turbidity to below detection limits. The anthracite filter alone could not reduce the turbidity levels to below the ADWG (2011) limit of 5 NTU. However, the subsequent GAC and membrane treatment both achieved turbidity levels below the ADWG (2011) limit.

The addition of the sand filter after the anthracite filter in the treatment train (anthracite filter column followed by a sand filter column, then a GAC filter column in turn followed by membrane filtration (Table Figure 3.28), produced a small improvement in removal of turbidity. It decreased to 5.5 NTU following the sand filter (Table 8.5 and Figure 8.9). This further improved the GAC filter's performance which decreased the average turbidity to 3.5 NTU. Following membrane filtration, the turbidity fell to below detection levels which represent an average total removal of 99%. The sand filter could not reliably reduce the turbidity levels to below the ADWG (2011) limit of 5 NTU. However, the subsequent GAC and membrane filtration both achieved turbidity levels below the ADWG (2011) stipulation.

The benefit of using these other filter media before GAC filtration is to provide a screening barrier for sediments and other suspended solids which might otherwise clog and reduce the life of the GAC.

Table 8.5 Turbidity results (based on 7 samples taken daily for 3 consecutive days)

Treatment Train	**Influent (NTU)**	**Anthracite (NTU)**	**% Removal**	**Sand (NTU)**	**% Removal**	**GAC (NTU)**	**% Removal**	**Membrane (NTU)**	**% Removal**
GAC Filter and membrane post-treatment	**14.0– 48.5** **29.2**	NA	NA	NA	NA	3.0 – **6.0** 4.5	85%	N.Q.	>99%
Anthracite + GAC and membrane post-treatment	**14.5 – 48.0** **28.9**	**5.0 – 12.0** **8.5**	71%	NA	NA	3.0 – **7.5** 4.2	85%	N.Q.	>99%
Anthracite + sand + GAC and membrane post-treatment	**14.5 – 48.0** **28.9**	**5.0 – 17.0** **8.8**	70%	4.0 – **8.0** 5.5	81%	2.0 – **5.0** 3.5	88%	N.Q.	>99%

Notes:

- N.Q. – not quantified, this parameter has not been reduced
- NA – not applicable
- Values excceding the ADWG (2011) limit are shown in bold

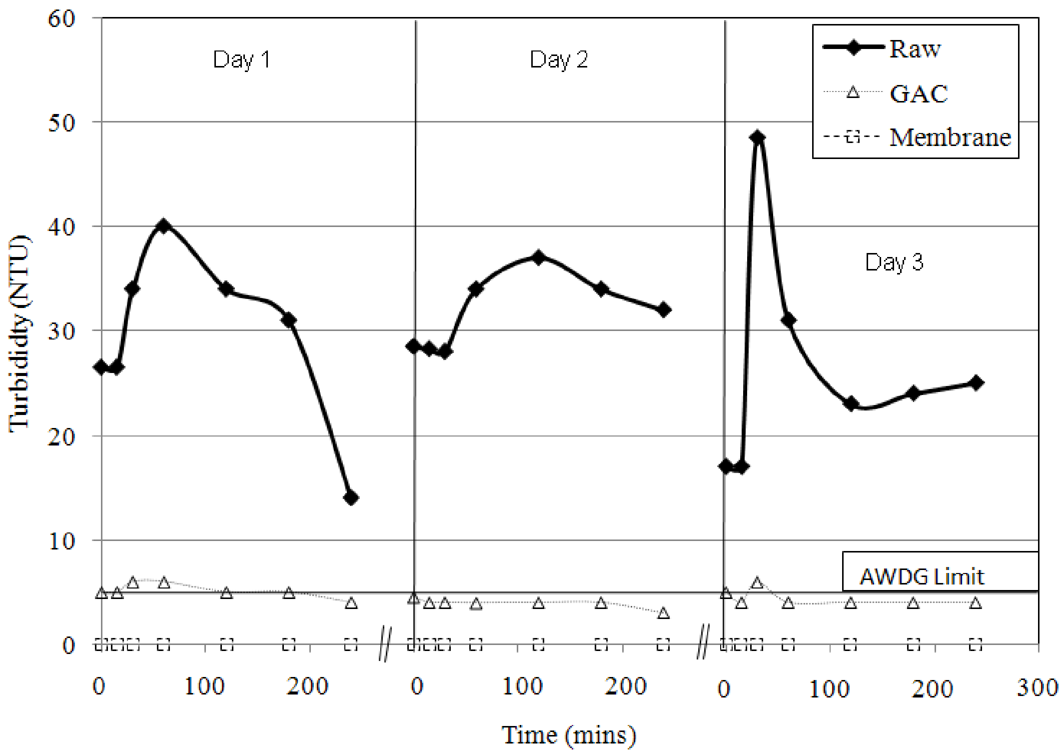


Figure 8.7 Turbidity with results for the GAC filter followed by membrane filter

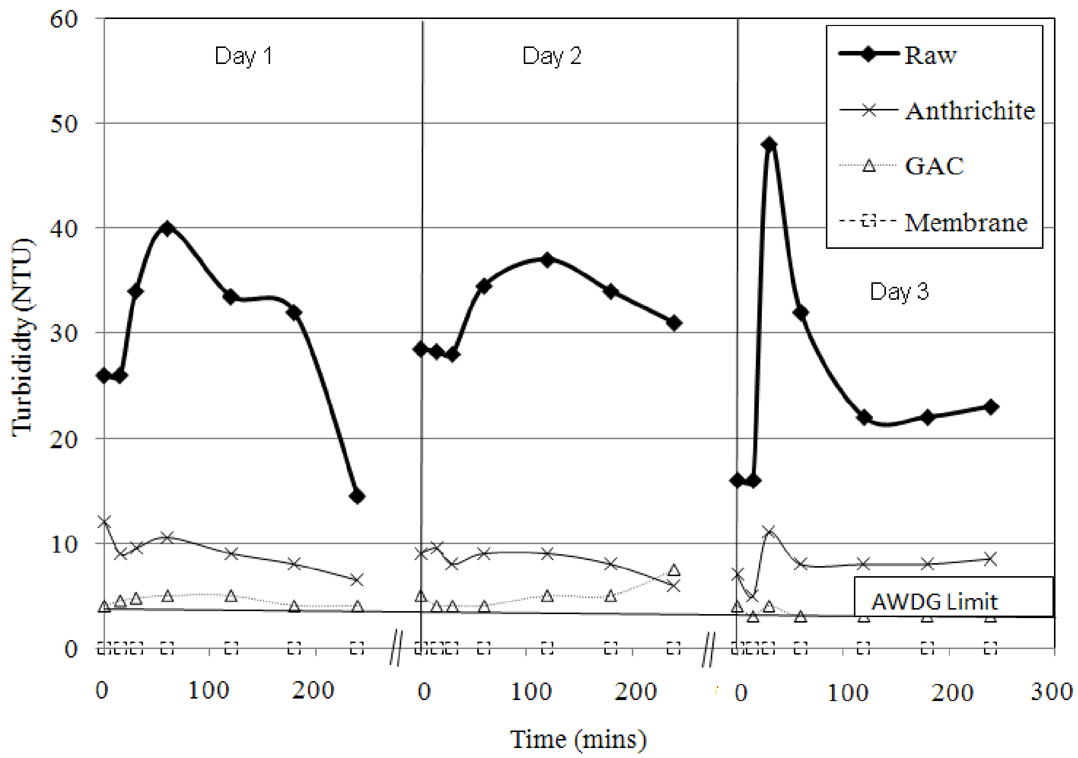


Figure 8.8 Turbidity results with anthracite, GAC and membrane filter treatment

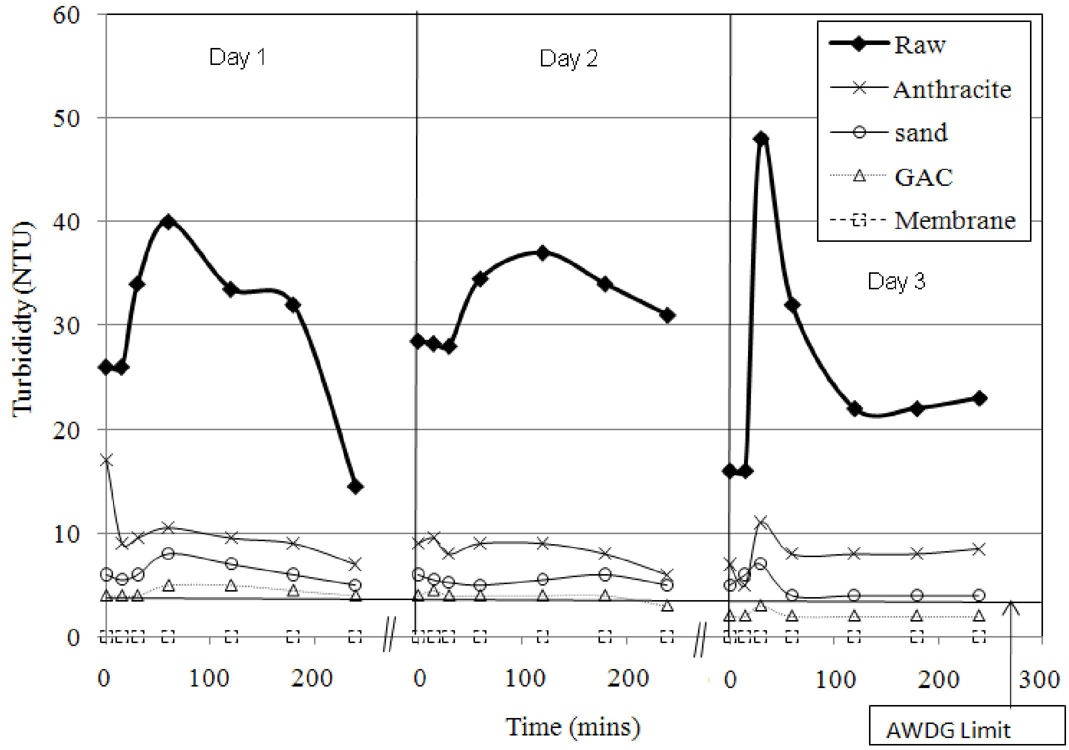


Figure 8.9 Turbidity results with anthracite, GAC and membrane filter treatment

Table 8.6 Removal of heavy metals with various treatment trains

Parameter	Raw Canal (mg/L)	Treatment Train									ADWG (2011) Limit (mg/L)
		GAC filter and membrane filter			Anthracite filter, GAC filter membrane filter			Anthracite filter, sand filter GAC filter and membrane filter			
		Pre Treatment[1] (mg/L)	Effluent (mg/L)	% Removal	Pre Treatment[2] (mg/L)	Effluent (mg/L)	% Removal	Pre Treatment[3] (mg/L)	Effluent (mg/L)	% Removal	
Aluminium	0.25	0.038	> 0.005	> 98%	NA	0.062	75%	NA	> 0.005	> 98%	< 0.2
Copper	0.025	NA	0.01	60%	0.019	0.003	88%	0.025	0.002	92%	< 2
Iron	1.277	0.082	0.005	99%	NA	> 0.005	> 99%	NA	> 0.005	> 99%	< 0.3
Manganese	0.478	0.098	0.084	82%	0.053	0.035	93%	0.048	0.035	93%	< 0.1
Lead	0.003	NA	0.003	0%	0.002	> 0.001	> 67%	0.002	> 0.001	> 67%	< 0.01
Zinc	0.058	0.021	0.004	93%	0.038	0.002	97%	0.017	0.001	98%	< 3

Notes:

- NQ – not quantified, this parameter has not been reduced
- [1] following GAC filtration
- [2] following anthracite and GAC filtration
- [3] following anthracite, sand and GAC filtration
- NA – not available
- Values excceding the ADWG (2011) limit are shown in bold,

Heavy metals

The details regarding the concentration of heavy metals in the influent stormwater are provided in Section 3.3.4. The influent raw stormwater itself had generally low concentrations of heavy metals (Table 8.6). No traces of arsenic, cadmium, chromium, selenium, silver or mercury were detected in the samples. The concentration of copper and zinc were below ADWG (2011) limits.

The GAC filter as a pre-treatment to membrane filtration worked effectively, resulting in significant reductions in most heavy metals. The GAC filters removed the majority of heavy metals by adsorption. An average of 85% of aluminium, 94% of iron, 80% of manganese and 64% of zinc were removed (Table 8.6). The GAC filter followed by membrane filtration reduced the concentration of all heavy metals to very low levels and to well within the ADWG (2011) limit (Table 8.6).

Organic matter characterisation

Categorization of organic matter was conducted on samples of the raw influent stormwater and following pre-treatment (Table 8.7). The concentration of DOC of the canal water was 5.86 mg/l out of which 66% was hydrophobic and the remaining 34% was hydrophilic. In the hydrophilic portion the majority of the substances were humic substances (52%), building blocks (23%) and biopolymers (8%), and less molecule neutrals and acids (16%). For comparative purposes, Table 8.7 summarises the values for raw rainwater.

After GAC filtration of stormwater, the concentration of DOC was 1.76 mg/l which represents a removal of 70%. It was found that 61% of organic matter was hydrophobic and 38% was hydrophilic. The GAC filter removed more than 75% of hydrophobic substances. The hydrophilic portion consisted of biopolymers (15%), humic substances (52%), building blocks (27%) and lower molecule neutrals and acids (7%).

After pre-treatment of raw stormwater using GAC filtration followed by membrane filtration, the concentration of DOC was 1.17 mg/l which represented 80% removal. It should be noted that most of the organic removal occurred through adsorption onto GAC. Any additional removal of organic matter by the membrane may be due to adsorption onto the membrane. The majority of this organic matter was hydrophobic (75%) compared to 25% being hydrophilic in character. In the hydrophilic portion the majority of the substances were humic substances (52%), building blocks (32%) and lower molecule neutrals and acids (14%).

Table 8.7 Fractionation of organic compounds by LC-OCD

Sample	DOC Dissolved mg/l, %DOC	HOC Hydrophobic mg/l, % DOC	CDOC Hydrophilic mg/l, % DOC	BIO- polymers mg/l, % DOC	Humic Substances (HS) mg/l, %DOC	Building Blocks mg/l, % DOC	LMW Substances mg/l, % DOC
Raw Stormwater	5.86	3.87	1.99	0.17	1.04	0.46	0.32
	100%	66%	34%	8%	52%	23%	16%
GAC filter	1.76	1.08	0.68	0.10	0.35	0.18	0.05
	70%	61%	38%	15%	52%	27%	7%
GAC filter and membrane filter	1.17	0.88	0.29	N.Q.	0.15	0.09	0.04
	80%	75%	25%	N.Q.	52%	32%	14%
Raw Rain Water (for comparison)	1.63 NA	1.26 77%	0.37 23%	N.Q.	0.2 54%	0.1 27%	0.06 16%

Notes:

- N.Q. – not quantified, this parameter has not been reduced
- LMW – low molecular weight

8.4.1. Practical implication of the study

The media filter performed at a high rate of 10 m/hr. It was able to remove suspended solids, organic matter and heavy metals in a consistent manner despite fluctuation in the influent concentration of these pollutants (Tables 8.4 to 8.7 and Figures 8.4 to 8.9). It could do this over a period of 5 hours for three consecutive days. This type of operation and experimental set-up mimics a stormwater harvesting system (Figure 8.10). In urban areas, rainfall events and the stormwater arising from them do not last for more than several hours. The stormwater needs to be treated at a high rate. The effluent from the media filtration is suitable for non-potable purposes such as street cleaning, irrigation of parks, etc.

Effluent from the media filter can be stored in a manner as shown in Figure 8.10. The stored water can be filtered under gravity through membrane filters. Though the filtration rate is slower the water quality of the effluent is high and for many parameters achieves drinking water standard (Tables 8.4 to 8.7 and Figures 8.4 to 8.9). The volume of water required for potable purposes is less compared to non-potable uses. This system can be suitably configured to meet these different demands.

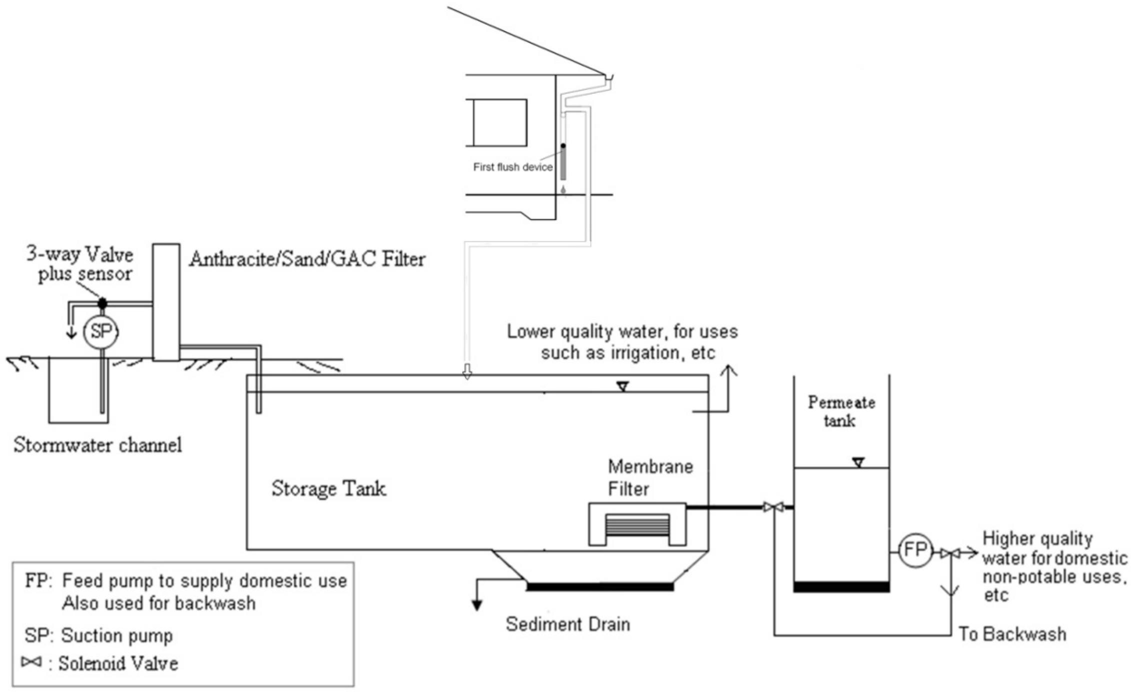


Figure 8.10 Possible prototype application of high rate media filtration followed by membrane filtration membrane (operated under gravitational head)

8.5. Conclusion

Laboratory studies showed that pre-treatment improved the quality of the filtrate as measured by the turbidity and TOC removal efficiency, resulting in less fouling of the MF and a smaller decline in flux. Using pre-treatment improved the TOC removal efficiency by between 10%-90%. Of the three pre-treatment methods, the GAC filter resulted in the highest TOC removal efficiency (88%). The turbidity following different treatments was between 0.5-1.2 NTU except with GAC filtration where the effluent turbidity was 5 NTU. The flux decline of raw stormwater (without pre-treatment) ranged between 35-40%, whereas after pre-treatment it decreased to between 8-30%. In-line flocculation-fibre media filtration indicated the lowest flux decline (8-9%) of the MF followed by GAC (24-30%) and flocculation (28-30%).

Pilot scale experiments showed that the Steriflow membrane filter treatment without any pre-treatment achieved an effluent filtrate turbidity of between 0.79-0.99 NTU, which was well below the ADWG (2011) limit of 5 NTU. The influent raw stormwater had generally low concentrations of heavy metals. Following membrane filtration the concentrations of all heavy metals shrank to very low levels and to well within the ADWG (2011) limits. The membrane filter could not remove TOC in significant amounts.

GAC adsorption used as post-treatment following Steriflow membrane treatment effectively reduced the TOC influent feed levels. GAC filtration of stormwater resulted in 70% removal of organics; in fact it removed all types of organics. The GAC filter did not further improve the turbidity level or heavy metal concentration following treatment using the Steriflow membrane system. However, the Steriflow membrane system which treated raw stormwater without any pre-treatment demonstrated a large decline in flux.

GAC adsorption used as pre-treatment following Steriflow membrane treatment effectively reduced the TOC influent feed levels. This treatment train effectively reduced the concentration of turbidity and heavy metals in the raw stormwater. It was observed, however, that the GAC filter used as a pre-treatment strategy in turbid water could clog and reduce the life of the GAC.

Finally, the addition of an anthracite filter column and sand filter to the treatment train of a GAC filter and membrane filter did not lead to any additional removal of the TOC. The benefit of using these other filter media before GAC filtration is in providing a screening barrier for sediments and other pollutants which might otherwise clog and reduce the life of the GAC.

Chapter 9

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9. Pilot Scale Rainwater Tank

9.1. Introduction

Research on membrane technology in recent times has increased exponentially and this has resulted in advances in operating efficiency. These improvements include reduced pressure requirements, decreased manufacturing costs through mass production, improvements in membrane materials leading to better treated water quality and of the wider implementation of membranes for new unique treatment applications. The combination of all these improvements has generated widespread application of membrane treatment strategies previously only available in large filtration plants operated by specialised water companies. Applications available today range from low-energy home-based membrane treatment systems such as under-sink cartridge filters or small reverse osmosis filters, to other applications such as decentralised small-scale filtration plants.

Typical collection and storage of rainwater introduces the potential for chemical, physical and microbial contamination. Rainwater contains contaminants including particles and micro-organisms. Rainwater harvested from roofs can contain animal and bird faeces, mosses and lichens, windblown dust, particulates from urban pollution, pesticides, inorganic ions from the sea (Ca, Mg, Na, K, Cl, SO_4), and dissolved gases (CO_2, NOx, SOx). High levels of pesticide are also found in rainwater. Concentrations of heavy metals in rain water tanks can also exceed the recommended ADWG (2011) levels and therefore makes it unsuitable for human consumption (Magyar et al., 2007, 2008; Han et al., 2006; Simmons, 2001). Rainwater storage tanks also accumulate contaminants and sediments that settle on the bottom.

Detailed sampling of residential rainwater tanks was undertaken in the Sydney metropolitan area. These results indicated that the quality of the water concerning many parameters met the ADWG (2011) stipulation. The pollutants that did not comply in a few rainwater tanks were the heavy metals, and in particular the concentrations of iron and lead, the pH and turbidity. The rainwater tank samples on average were generally below the ADWG (2011) iron limit of 0.3 mg/L, however, each tank contained at least one sample over this limit with individual concentrations as high as 4.70mg/L. The lead concentration was also of concern with most tanks containing water exceeding the ADWG (2011) lead limit of 0.01mg/L with samples as high as 0.067mg/L. The water in one of the rainwater tanks registered values below the minimum recommended pH level of 6.5 with an average pH of 5.7. A number of other tanks had water close to the minimum pH level of 6.5. The rainwater tanks containing water with low pH levels were harvested from Colorbond or Zincalum metal roofing. Although the average turbidity of water in each rainwater tank complied with the ADWG (2011) recommended limit of 5 NTU, 3 tanks contained occasional samples with turbidity up to 12 NTU (Kus et al., 2010).

This chapter presents the results of monitoring of a pilot scale rainwater treatment system consisting of a media (Granular Activated Carbon - GAC) filter and a gravity fed membrane filter (Ultra Flo). It was operated for a period of 120 days at a residential household in Kogarah, Sydney. The water quality and flux decline was monitored over this period. The experimental details were discussed in Chapter 3 (Section 3.4).

9.2. Results and discussion

9.2.1. Flux Decline

The recorded flux decline is shown in Figure 9.1. The results of the pilot scale experiment indicate that the flux continued to decline to 0.47 L/m²/h after 120 days although it was relatively stable over the final 60 days of the experiment. During this period the membrane was backwashed twice (on days 8 and 12) for a period of 30 seconds. On both occasions there was an increase in flux, which was temporary. The increase in flux generally lasted several hours before it returned to levels prior to the backwash. No further backwashing was undertaken following day 12.

At the completion of the experiment the cartridges were opened up and examined. There was no apparent cake formation observed on the membrane during the study period which was due to the very low turbidity (0.58 NTU) and DOC (0.12 mg/L) in the rainwater after it was pre-filtered with the GAC adsorption filter.

Without the requirement of backwashing, the initial set-up cost of a filtration system can be substantially reduced as there is no need for an automated backwash system which generally includes pressure sensors, a pump, other electronics and a treated water reservoir for the backwashing. The laboratory experiments were carried out for 20 days and the flux at the end of the period was around 4.5 L/m²/h (Section 7.3). The pilot scale study was carried out 6 times so it is expected that the probability of experimental failure would have occurred: firstly, if the experiment had been prolonged; and secondly, if there been no backwashing. Furthermore in the case of the laboratory experiment the GAC pre-treatment had a larger amount of GAC (Table 3.16, Chapter 3, Section 3.3.3).

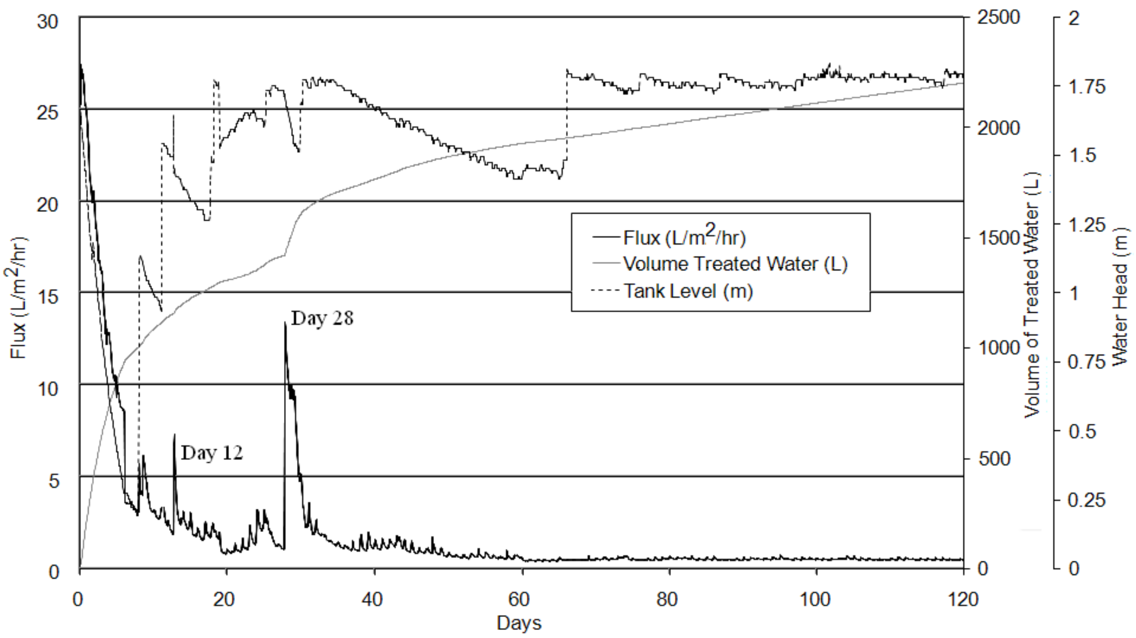

Figure 9.1 Membrane flux decline over the duration of the pilot scale study. Also shown is the water head in the rainwater tank and the volume of water treated.

Table 9.1 Cartridge Operation

Type	**Experiment Duration (Days)**	**Treatment Water (L)**	**Steady Flux (L/hr/m²)**
Granular Activated Carbon	120	2196	*
Hollow Fibre Membrane (MF)	120	2196	0.47

* Limited by Membrane Flux

9.2.2. Water quality results

TOC

The treatment train of the GAC filter followed by membrane filtration demonstrated that the GAC treatment was able to reduce the influent TOC concentrations by an average of 71.1% from 0.42 mg/L to 0.12 mg/L (Figure 9.2 and Table 9.2). In the initial period, during the first 30 days of operation, the TOC in the effluent was higher at approximately 0.4 mg/L (Figure 9.2). This corresponded to the period of development of biofilms on the GAC. Beyond this initial period the concentration of TOC declined to below detectable limits after 30 days' operation. The removal rate of TOC during this period was more than 99%. The submerged membrane filter system used as post-treatment to the GAC filter indicated a small improvement in TOC removal, particularly during the initial first 30 days.

Turbidity

The treatment train of the GAC filter column followed by membrane filtration reduced the turbidity by between 60-96% or to levels of 0.3-0.4 NTU (Figure 9.3 and Table 9.2). The GAC filter achieved significant reductions in turbidity (Figure 9.3). Membrane filtration provided additional turbidity removal of up to 20%. The turbidity following membrane filtration was small and below 0.4 NTU and the ADWG (2011) advice a limit of 5 NTU.

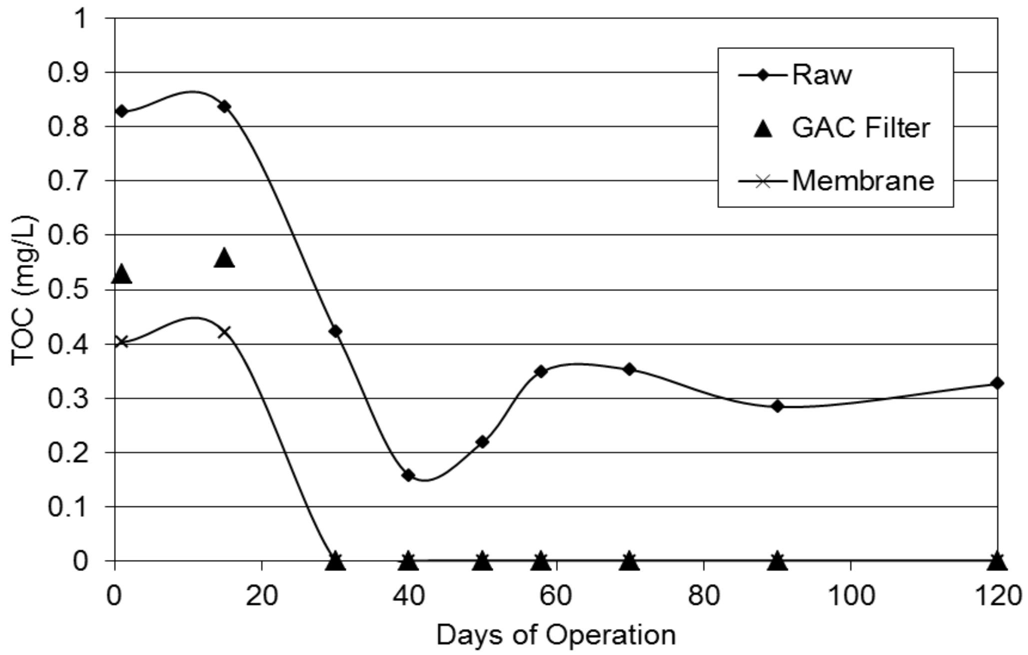

Figure 9.2 TOC results with GAC filter followed by membrane filter filtration.

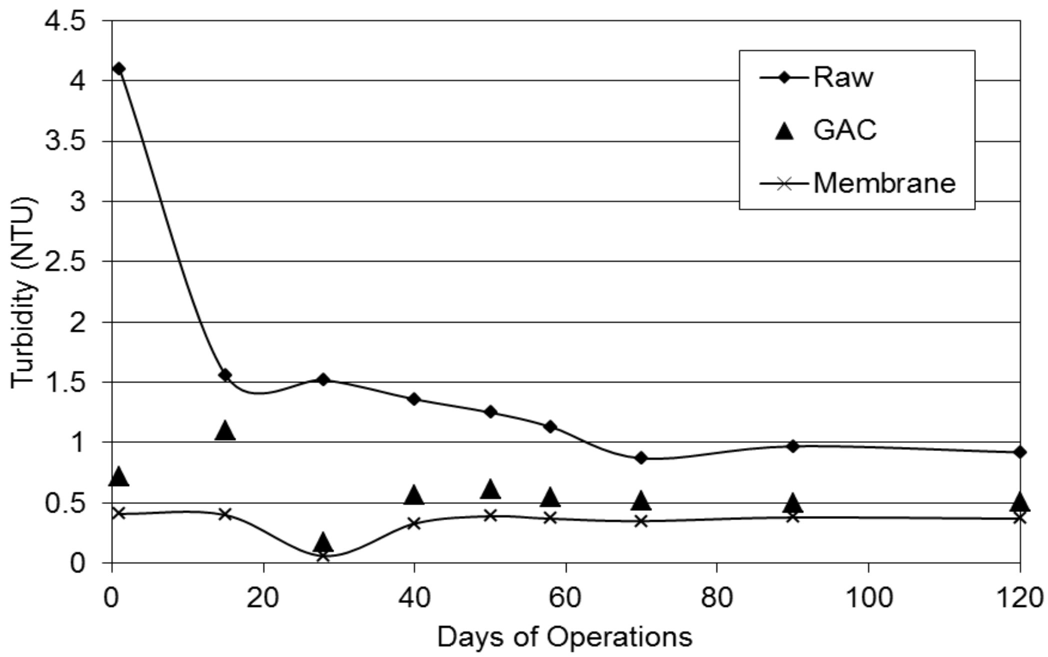

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Figure 9.3 Turbidity results with GAC filter followed by membrane filter filtration.

Table 9.2 Turbidity and TOC results based on samples taken during the pilot trial

Parameters	ADWG (2011) Limit	Detectable Limit	Tap Water	Summary of Analysis*				
				Raw Water*	GAC Filter	GAC Filter Removed (%)*	Pre-Treatment + Membrane*	GAC + Membrane Filter Removed (%)*
Turbidity (NTU)	5	0.2	<0.5	1.52 (0.87 - 4.1)	0.58 (0.17 - 1.1)	62%	0.34 (0.06 - 0.41)	78%
Total Organic Carbon (mg/L)	0.2	0.001	< 5	0.42 (0.16-0.84)	0.12 (<0.001-0.56)	71%	0.09 (<0.001-0.42)	78%

* mean average value followed by range of values.

Heavy Metals

The influent raw stormwater itself had generally low concentrations of heavy metals (Table 9.3). No traces of cadmium, selenium, silver or mercury were detected in the samples. The concentration of a range of heavy metals, namely aluminium, arsenic, copper, iron, manganese, nickel and zinc were below ADWG (2011) limits. The concentration of lead was notably above the ADWG (2011) recommendation. The concentration of aluminium, arsenic, cadmium, copper and iron were at or below levels supplied in potable tap water in Sydney.

The treatment train of the GAC filter followed by membrane filtration functioned effectively with significant reductions occurring in most heavy metals. Aluminium, iron and manganese fell to below detection limits. The concentration of lead reduced to below 0.005 mg/L, which was below the ADWG (2011) limit of 0.01 mg/L. The treatment train reduced the concentration of all heavy metals to very low levels and well within the ADWG (2011) limit (Table 9.3).

Table 9.3 Heavy metal results based on samples taken during the pilot trial

Parameters	ADWG (2011) Limit	Detectable Limit	Tap Water	Summary of all Analysis			
				Raw Rainwater	Pre-Treatment (GAC)	Pre-Treatment + Membrane	Pre-Treatment + Membrane % Removed
Aluminium (mg/L)	0.2	0.005	0.013	0.011	0.010	<0.005	>55%
Copper (mg/L)	2	0.001	0.247	0.004	0.018	0.027	No Improvement
Iron (mg/L)	0.3	0.005	0.007	0.006	0.010	<0.005	>50%
Manganese (mg/L)	0.1	0.001	<0.001	0.002	0.002	<0.001	>50%
Lead (mg/L)	0.01	0.001	<0.001	0.011	0.005	0.005	55%
Zinc (mg/L)	3	0.001	0.024	0.033	0.514	0.504	No Improvement

Organic matter characterisation

Categorisation of organic matter was conducted for raw rainwater and after pre-treatment (Table 9.4). It emerged that the concentration of DOC of the raw rainwater was 1.74 mg/l out of which 30.3% was hydrophobic and the remaining 69.7% was hydrophilic. In the hydrophilic portion the majority of substances were biopolymers (5.2%), humic substances (25.3%), building blocks (12%) and lower molecule neutrals and acids (24.5 and 2.6, respectively). Biopolymers were below detection limits.

After pre-treatment of raw rainwater using GAC filtration followed by membrane filtration, the concentration of DOC was 1.14 mg/l which represents 34% removal efficiency. The majority of organic matter was hydrophilic (32.9%) compared to 67.1% of hydrophobic organic matter. In the hydrophilic portion, the majority of substances were biopolymers (5.2%), humic substances (43.3%), building blocks (8.2%) and less molecule neutrals and acids (9.7% and 2.1%, respectively). It was found that rainwater treated with the GAC filter had the majority of its organic substances removed. The GAC filter removed all types of organic material. In general membrane filtration can remove only a small amount of organics.

Table 9.4 Fractionation of organic compounds by LC-OCD

Sample	DOC Dissolved mg/l, %DOC	HOC Hydrophobic mg/l, % DOC	CDOC Hydrophilic mg/l, % DOC	BIO-polymers mg/l, % DOC	Humic Substances mg/l, %DOC	Building Blocks mg/l, % DOC	LMW Neutrals mg/l, % DOC	LMW Acids mg/l, % DOC
Raw rainwater	1.74	0.53	1.20	0.09	0.44	0.21	0.43	0.05
	100%	30.3%	69.7%	5.2%.	25.3%	12.0%	24.6%	2.6%
GAC filter and membrane filter	1.14	0.38	0.77	0.04	0.50	0.09	0.01	0.02
	100%	32.9%	67.1%	3.8%	43.3%	8.2%	9.7%	2.1%

LMW – low molecular weight;

9.3. Conclusion

This pilot scale study monitored the water quality and membrane flux decline from a single use, gravity-driven membrane and GAC cartridge filter system during a 120-day trial at a residential household. The long-term stable flux achieved was 0.47 $L/m^2/hr$ with minimal reduction over the final 60 days of the experiment.

It was found that rainwater treated by the GAC filter removed the majority of organic substances. The system reduced turbidity by between 60-96% or to levels of 0.3-0.4 NTU, below the ADWG (2011) limit of 5 NTU. The influent raw rain water initially contained low concentrations of heavy metals. The concentration of lead fell to less than 0.005 mg/L, below the ADWD limit of 0.01 mg/L. The treatment of all other heavy metals resulted in very low levels, well within the ADWG (2011) limit.

The pilot plant experiment demonstrated that this type of system could result in a low cost and low maintenance operation. Based on the results of the pilot-trial the cartridge filters need to be replaced every 3 months to ensure that the water quality obtained from household rainwater tank results in the removal of harmful pollutants. More generally, this period depends on the quality of the raw rainwater used in the testing. It will also depend on the experimental conditions.

Chapter 10

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10.Conclusion and Recommendations

10.1. Introduction

Rainwater has been harvested throughout the history of mankind as a means of providing daily water consumption. In modern society rainwater tanks are becoming accepted as an alternative source of non-potable grade water which can be utilised for non-potable uses such as flushing toilets, washing clothes, watering gardens and washing cars. Rainwater harvesting with treatment could be utilised to augment decentralised potable water supplies. This study investigated the development of a cost-effective filtration system to improve water quality in rainwater tanks. This concluding chapter summarises the key outcomes of the study and suggests areas of future research.

10.2. Characterisation of rainwater

The water quality of rainwater collected in tanks at various locations in Sydney metropolitan and rural NSW areas were analysed in terms of physical, chemical and organic characteristics. These were then compared against drinking water standards (ADWG, 2011). The results indicated that before treatment, the rainwater had already complied with many of the parameters specified in the ADWG (2011), though as previous studies have demonstrated, certain pollutants have the potential at times to exceed the ADWG (2011) limit. These parameters included heavy metals (iron and lead) as well as pH and turbidity for rainwater tanks in the Sydney metropolitan area and pH for rural rainwater tanks. Faecal and total coliforms also emerged as a problem in most rainwater tanks. Treatment is required to comply with ADWG (2011).

Analysis of first flush roof runoff from an urban residential roof located in the Sydney metropolitan area showed that the quality of stored rainwater could be significantly improved after bypassing the first 2 mm of rainfall. Other pollutants (excluding faecal and total coliforms) turbidity and lead required bypassing approximately the first 5 mm of rainfall. The findings demonstrate that diverting the first flush off a roof, which is heavily polluted, is important and could significantly reduce treatment requirements and associated energy consumption.

10.3. Water demand

Water consumption patterns in the Sydney metropolitan area were analysed based on metered potable water usage between January 2002 and October 2009, and included residential properties that installed a rainwater tank and received a rebate from Sydney Water. The data showed that the average annual water consumption per household in Sydney's metropolitan area declined from 282 kL/annum to 200 kL/annum during the study period. When excluding the impact of rainwater tanks, the average water consumption fell by 24% due to effective demand management techniques such as the Sydney-wide water restrictions and the introduction of water efficient fixtures. By installing a rainwater tank the average water use was further reduced by an average of 9% or 24 kL. The analysis was conducted on a LGA (local government authority) basis and this study provides data on demand that can be used to optimise the volume of the permeate storage tank of the proposed rainwater tank treatment system.

10.4. GAC adsorption filtration

Chapter 6 formed the first part in the development of an affordable adsorption and membrane-based treatment system by investigating the long-term performance of a GAC adsorption filter as pre-treatment to micro-filter membrane filtration used to treat raw rainwater. During the ensuing experiments, influent samples that were analysed largely complied with the ADWG (2011) recommendations with the exception of total and faecal coliforms and at times turbidity and heavy metals (iron, manganese and lead). GAC adsorption performed well with turbidity in all effluent samples being below the ADWG (2011) limit. The concentration of heavy metals in all effluent samples complied with the ADWG (2011) with the exception of lead in one sample and iron in two samples. Further treatment will still be necessary for total and faecal coliforms.

A pilot scale inline GAC adsorption filter was developed and operated as a pre-tank filter to a rainwater tank. The inline GAC adsorption filter did not operate effectively due to poor hydraulic performance. Nevertheless the filter did perform well in reducing the concentration of many parameters to the extent that all, except total coliforms and faecal coliforms, complied with the ADWG (2011).

10.5. Membrane micro-filtration

Micro-filtration membrane tests (Steriflow Filtration Systems, Ulra Flo and INGE) were examined in Chapter 7 under various trans-membrane pressures to determine their suitability as a membrane filter for treating raw and pre-treated rainwater under gravity head (energy efficient). The Steriflow stainless steel membrane is a strong and long-lasting filter. However, in its current state, it requires a relatively high driving head which is not suitable for the purposes of a typical residential rainwater tank.

The Ultra Flo and INGE filters were able to operate at a reasonable flux within driving heads of 2 m which is the upper limit of what is typically available in a residential rainwater tank. The variation of flux with time showed that it was common for various operational configurations of driving head to converge to a similar flux decline path regardless of driving head or whether GAC adsorption pre-treatment was employed. The flux took longer to reduce and converge if the driving head was high or if the rainwater was pre-treated.

The Ultra Flo membrane filtration without pre-treatment performed well when removing TSS, turbidity, and lead to below detectable levels and compliance with the ADWG (2011). It also removed much of the total and faecal coliforms to detectable limits. However, it did not remove TOC. Pre-treatment of GAC adsorption was able to assist with removing the TOC. Although the INGE filter performed in a manner similar to the Ultra Flo membrane filter, it was not suitable as its current cross-flow operation mode required pumping. Further treatment is required to remove faecal and total coliforms or at least guarantee all removal. This is good practice as micro-filtration membrane treatment does not remove viruses.

10.6. Stormwater harvesting pilot scale plant

On completion of laboratory experiments, pilot scale filtration systems were developed to determine the performance of various media filter and membrane filter systems. Chapter 8 examined a Steriflow membrane filter under various operation configurations at a stormwater harvesting plant based in Carlton, Sydney. Its harvested stormwater baseflow had a water quality, comparable to rainwater.

Pilot scale experiments showed that the Steriflow membrane filter treatment without any pre-treatment removed a majority of the organic substances and removed all other parameters analysed to below the ADWG (2011). The membrane filter could not remove TOC in significant amounts. When GAC adsorption was used as pre-treatment to the Steriflow membrane, it effectively reduced the TOC influent feed levels. GAC adsorption used as post-treatment following Steriflow membrane treatment also effectively reduced the TOC influent feed levels.

It was observed that utilising the GAC filter as pre-treatment with turbid water could clog and reduce the life of the GAC. Similarly, operating the Steriflow membrane system with raw stormwater (without any pre-treatment) demonstrated a larger flux decline. The addition of an anthracite filter and/or sand filter prior to the GAC filter and membrane filter did not provide any additional reduction in the overall removal of TOC. However, these filters provided a screening barrier for sediments and other pollutants which might otherwise clog and reduce the life of the GAC.

10.7. Design and operational simulation of a pilot scale rainwater tank

Chapter 9 utilised the research and results in this study and employed them in a pilot scale rainwater filtration system. The gravity-driven GAC adsorption and membrane filtration system installed at a residential household was monitored for its effectiveness over 120 days. The long-term stable flux achieved was 0.47 L/m^2/hr with minimal reduction over the final 60 days of the experiment. It was found that rainwater treated by the GAC filter removed most of the organic substances and removed other analysed parameters to below the ADWG (2011) limits.

The pilot plant experiment demonstrated that this type of system could result in a low cost and low maintenance operation. Based on the results of the pilot-trial the cartridge filters (GAC and membrane filters) need to be replaced every 3 months to ensure that they adequately remove pollutants and the effluent water quality complies with ADWG (2011) standards. More generally, this period depends on: firstly, the quality of the raw rainwater to be treated; and secondly, on the experimental conditions.

In summary a cost effective filtration system was developed that did improve the water quality in rainwater tanks. This culmination of this study can be further explored and developed through future research.

10.8. Future research

This study builds on the published literature on Australian rainwater tank water quality analysis and provides data on current trends in domestic residential water usage in the Sydney metropolitan area. It has also defined the role of rainwater tanks in reducing the demand for potable water. With continuing advances in membrane technology, the costs associated with operating and replacing membranes will continue to fall. This opens up opportunities to use rainwater for providing a potable grade water source in each residential dwelling's backyard. This will effectively complement centralised water supplies and delay the expansion in infrastructure that will be required when population densities increase in the next few decades. The following areas should be looked at in future studies on this important topic:

- The treatment system developed in this study provides a basis for a potable rainwater treatment system. The treatment system should be augmented for control of viruses and bacteria. Detailed monitoring over the long-term for a whole range of water quality parameters is required to fully define and describe safe operation.

- More detailed demand analysis is required to fully define storage requirements for plotting seasonal and long-term variations against a more diverse range of risk profiles and intended uses.

Appendix A

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Appendix A - Water Demand High Resolution Figures

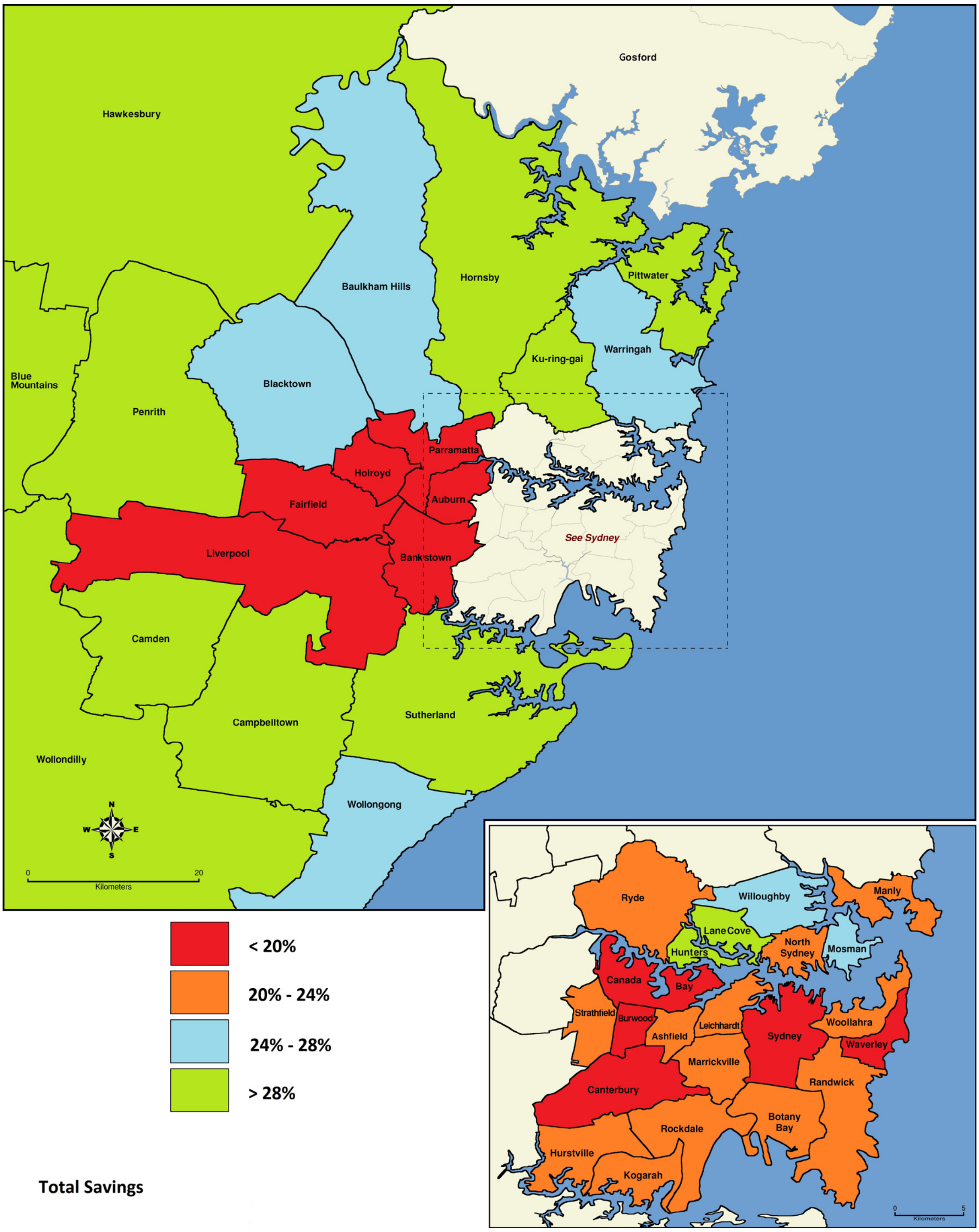

Figure 5.2a Water consumption for single residential houses in the Sydney metropolitan area - Reduction of water consumption in each LGA over the study period

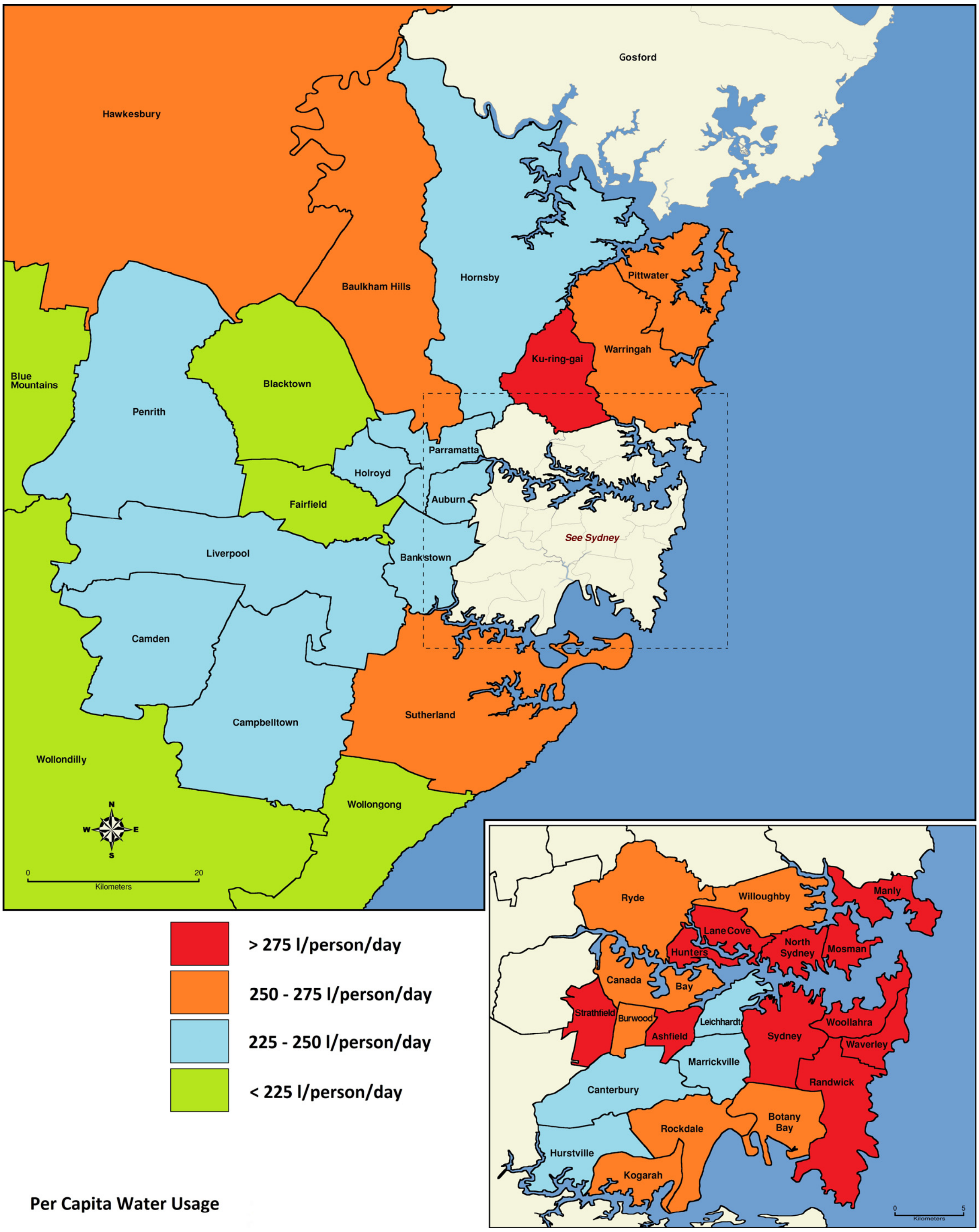

Figure 5.2b Water consumption for single residential houses in the Sydney metropolitan area - Average daily per capita water usage

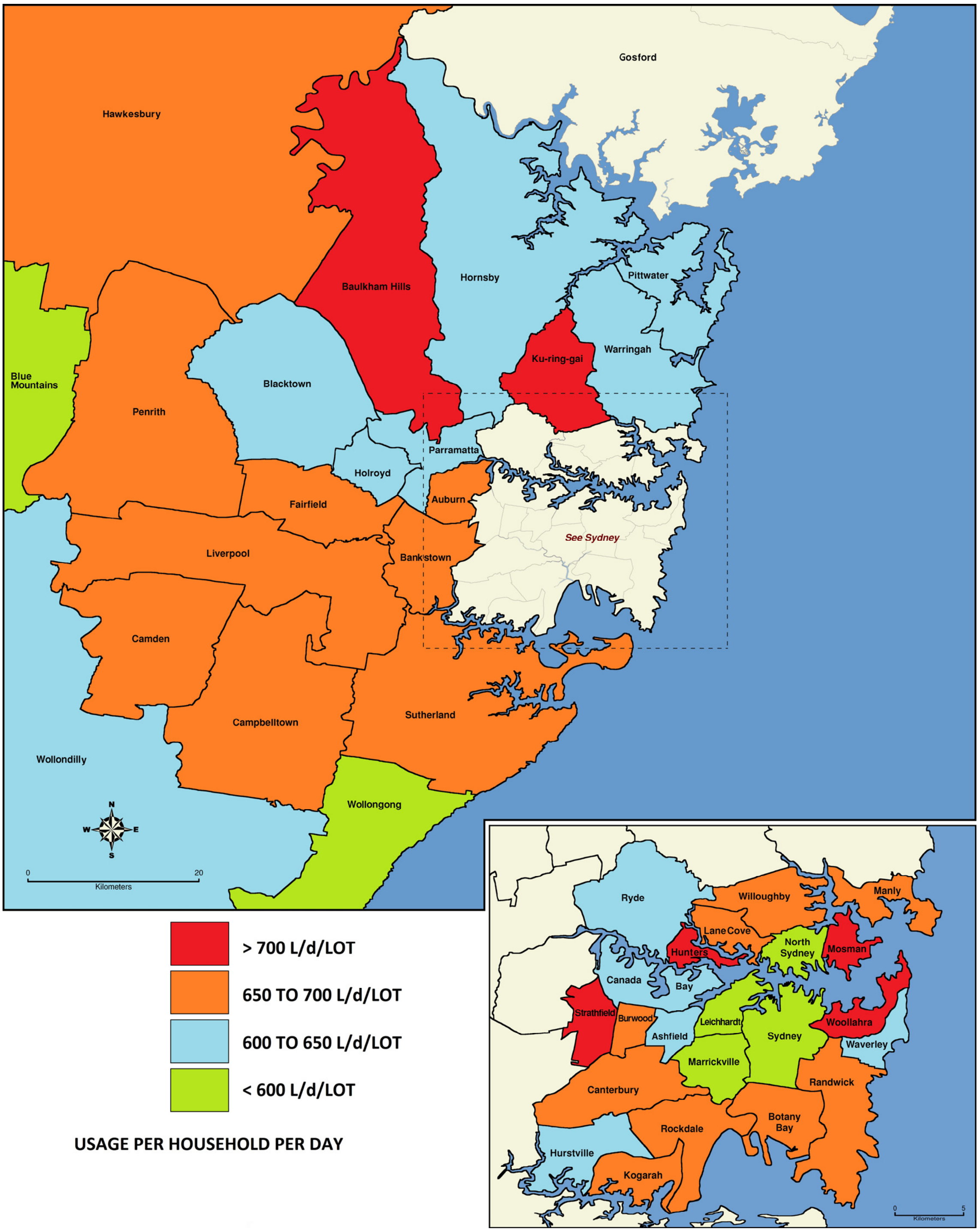

Figure 5.2c Water consumption for single residential houses in the Sydney metropolitan area - Average daily household water usage

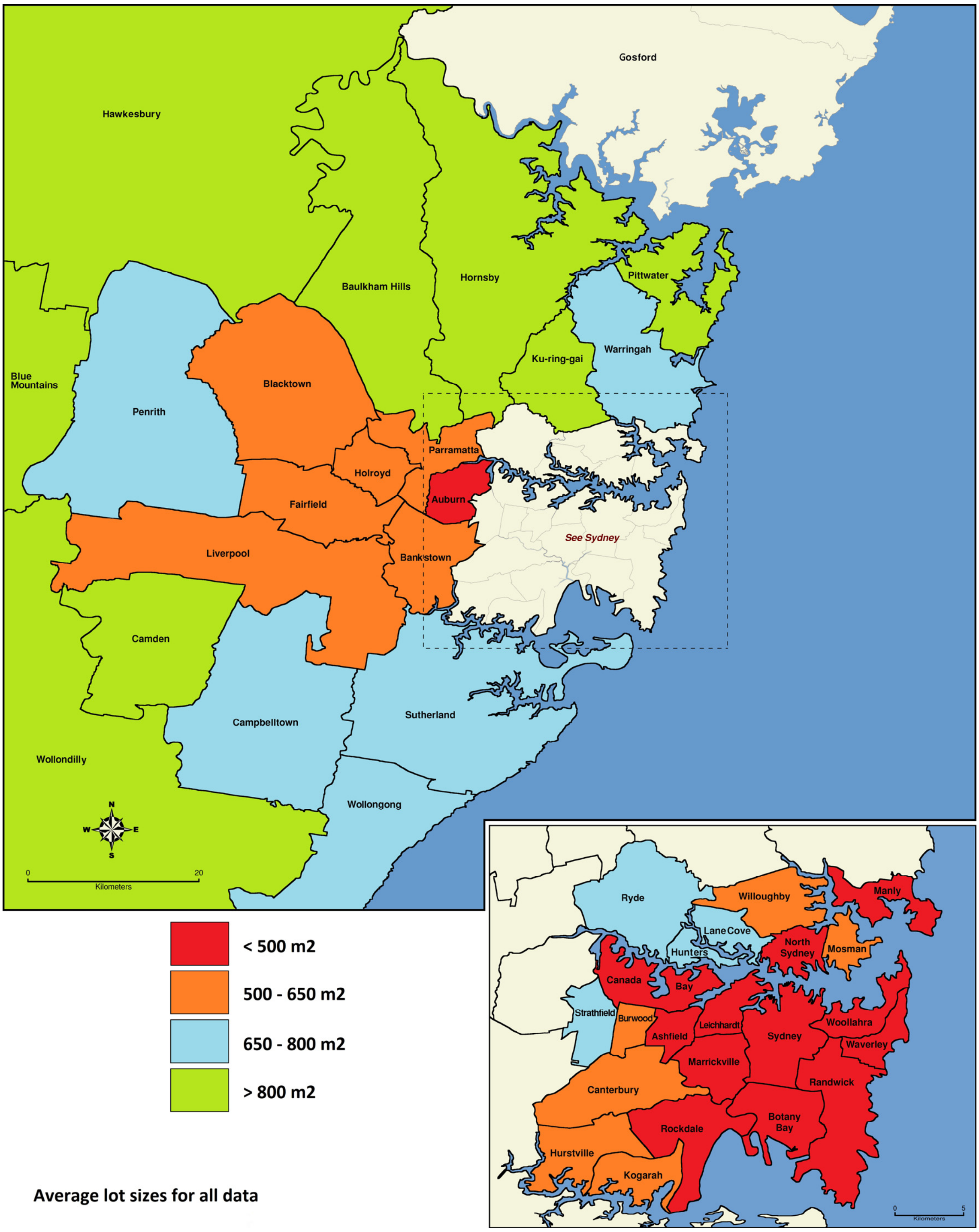

Figure 5.3a Average area and number of persons for single residential houses in Sydney – Average lot sizes

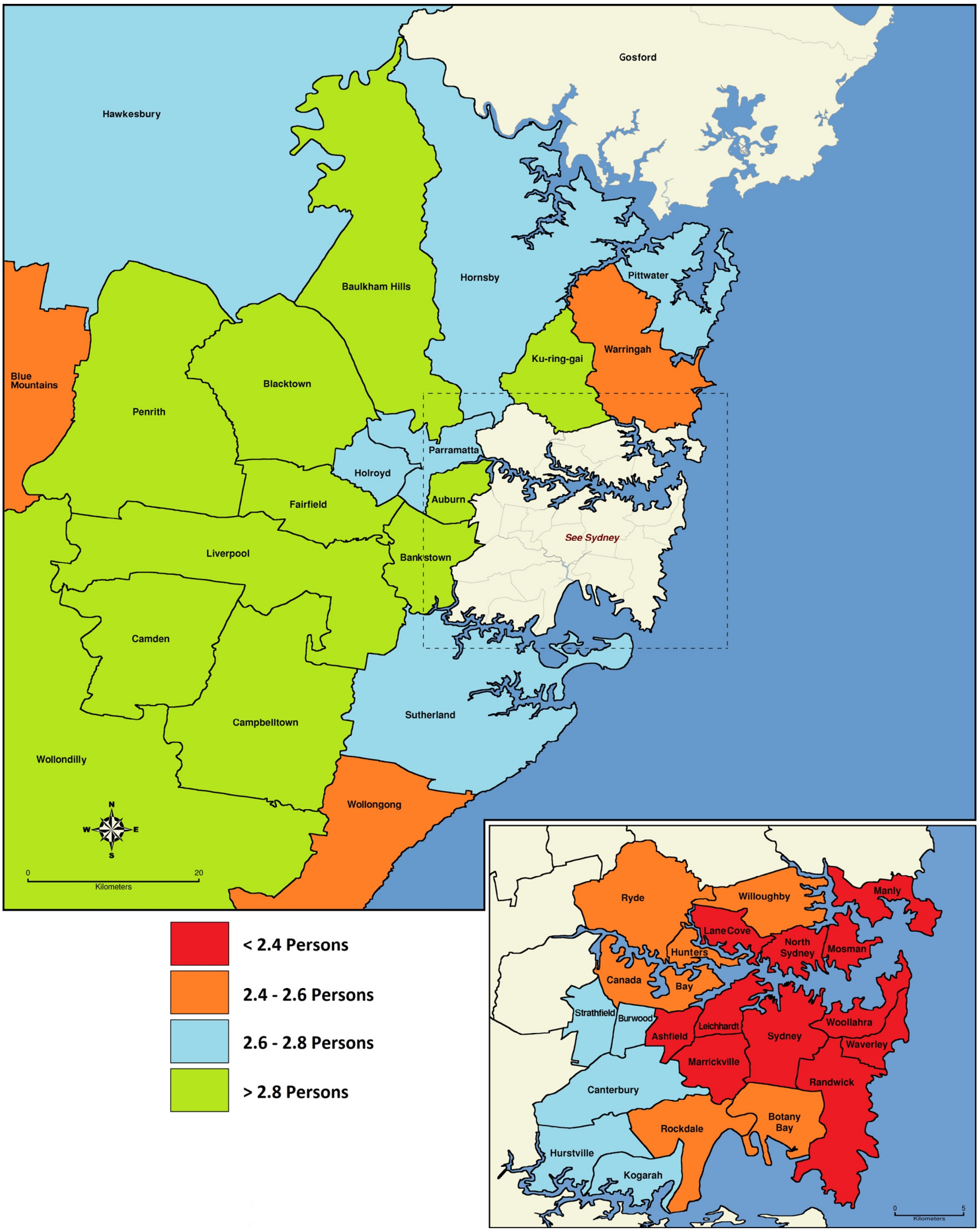

Figure 5.3b Average area and number of persons for single residential houses in Sydney – Average number of persons per household

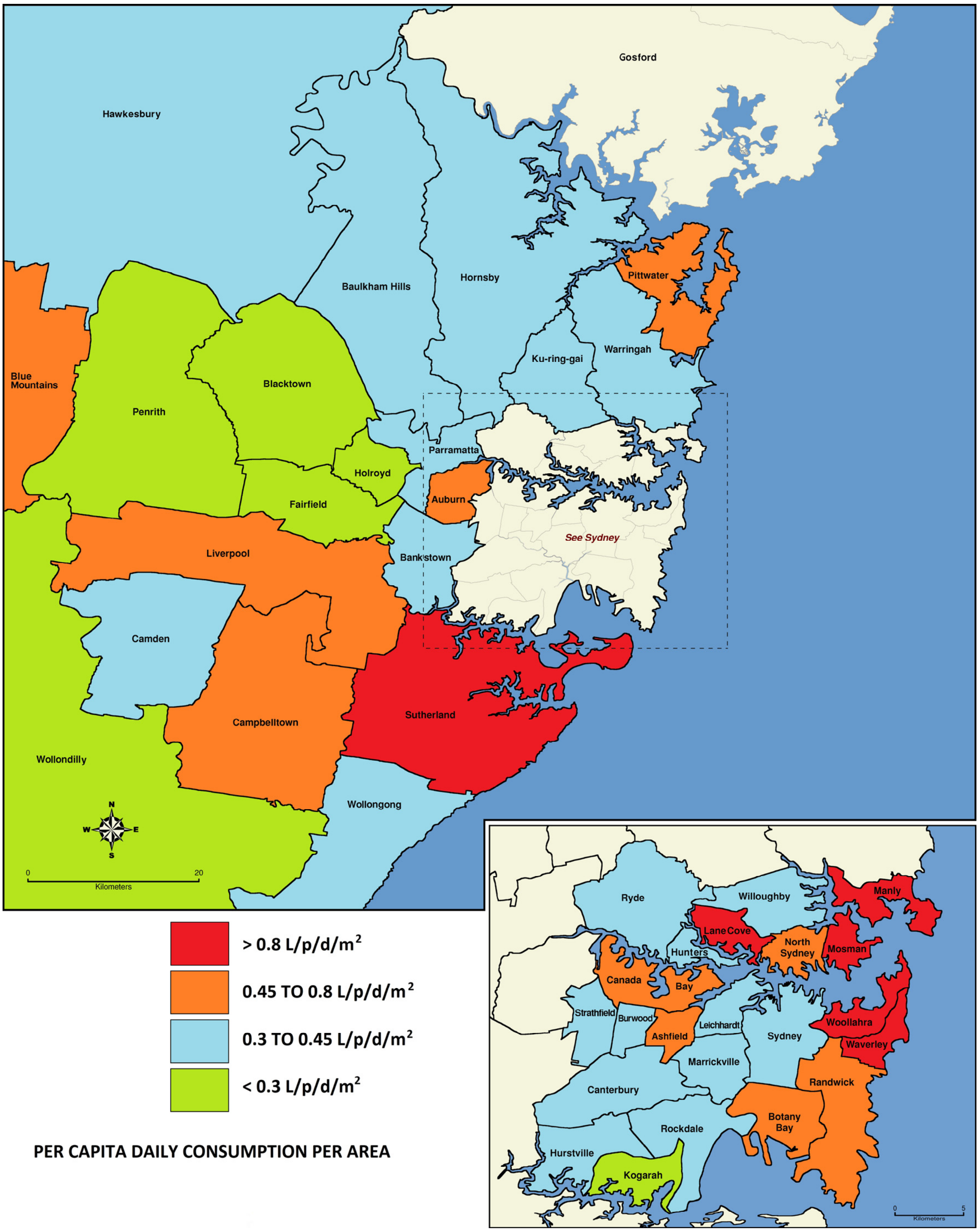

Figure 5.3c Average area and number of persons for single residential houses in Sydney – Average daily per capita water usage per square meter of lot area

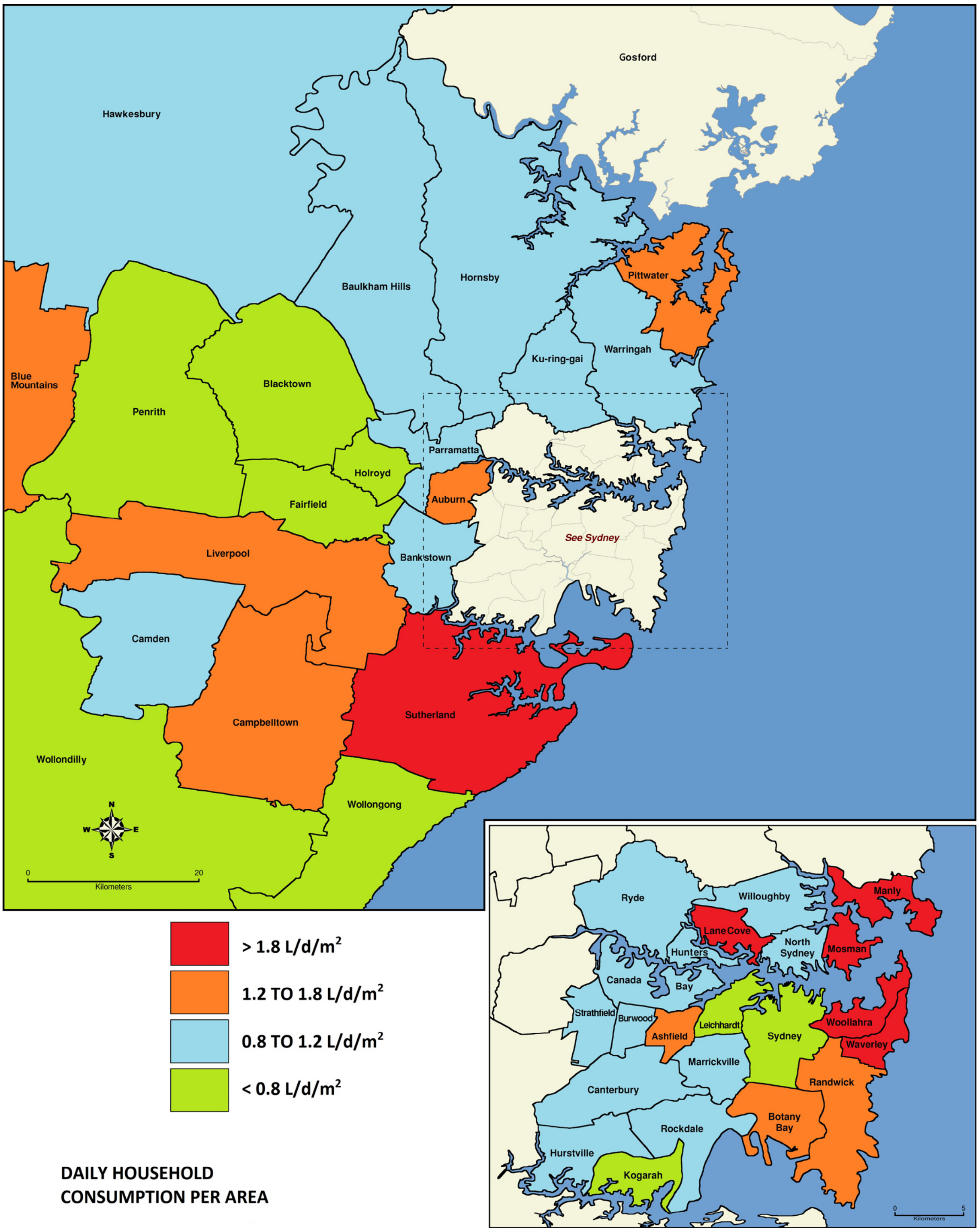

Figure 5.3d Average area and number of persons for single residential houses in Sydney – Average daily household water usage per square meter of lot area

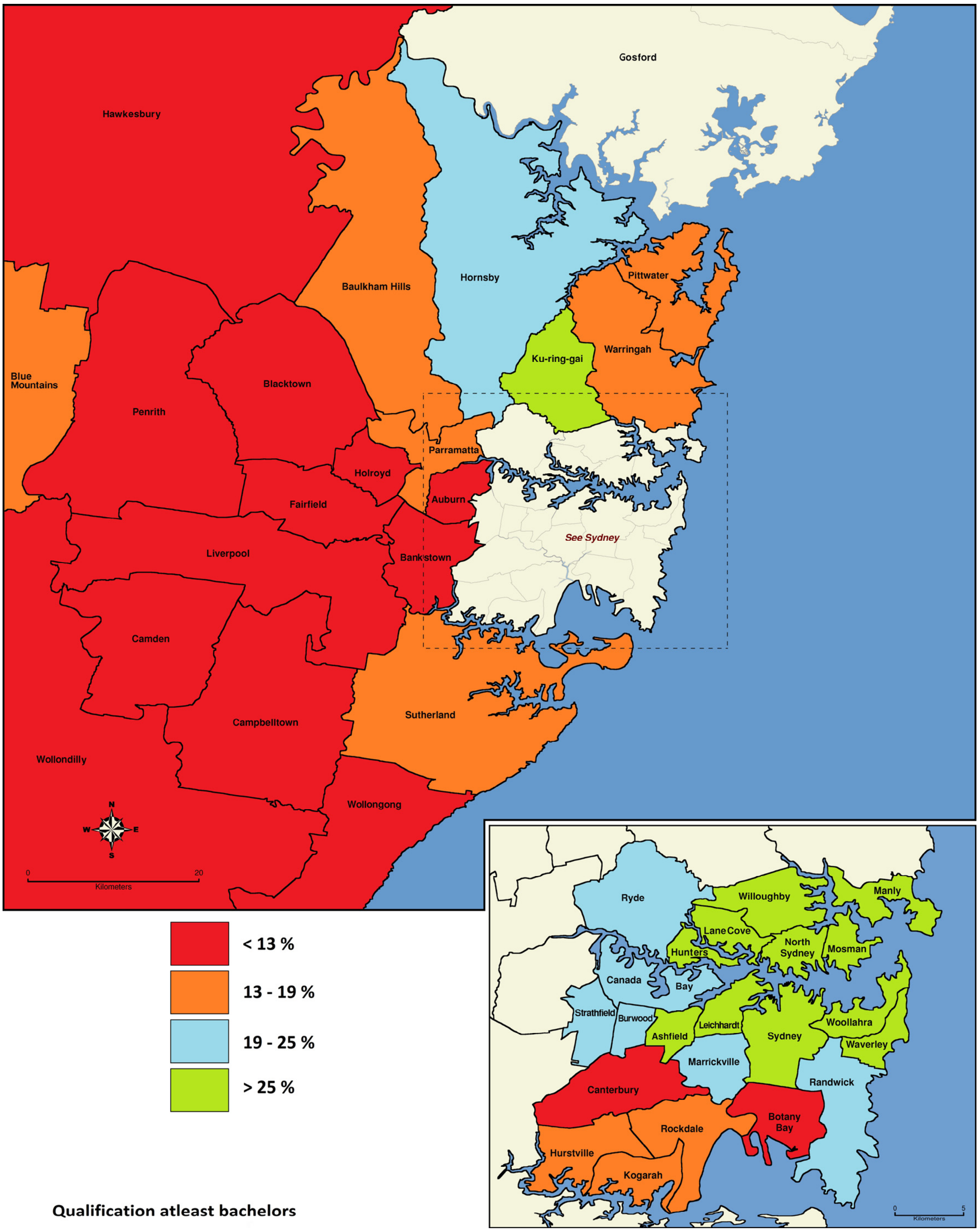

Figure 5.4a Socio-economic indicators in the Sydney metropolitan area – Average number of people with Bachelor qualifications or higher

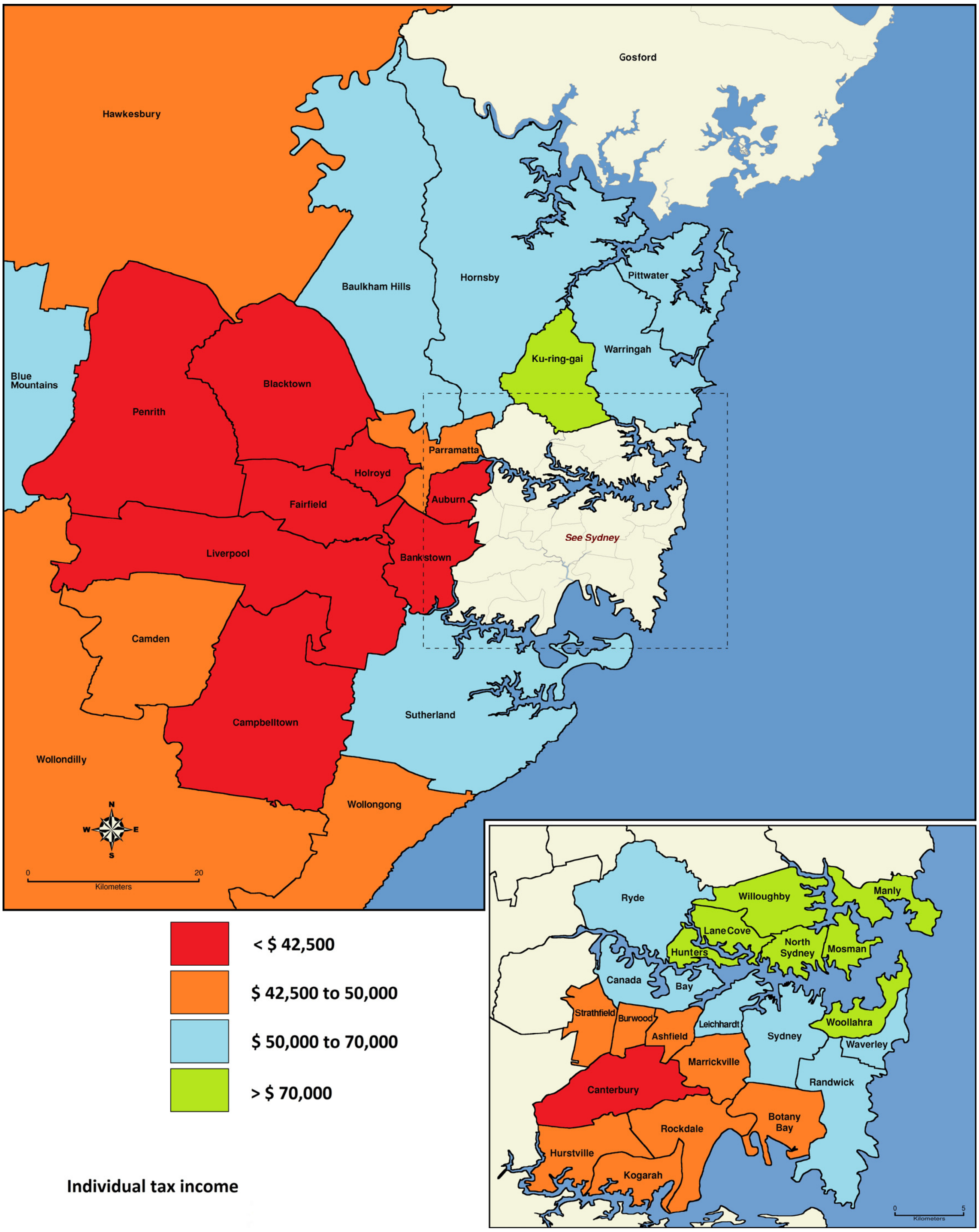

Figure 5.4b Socio-economic indicators in the Sydney metropolitan area – Mean taxable income

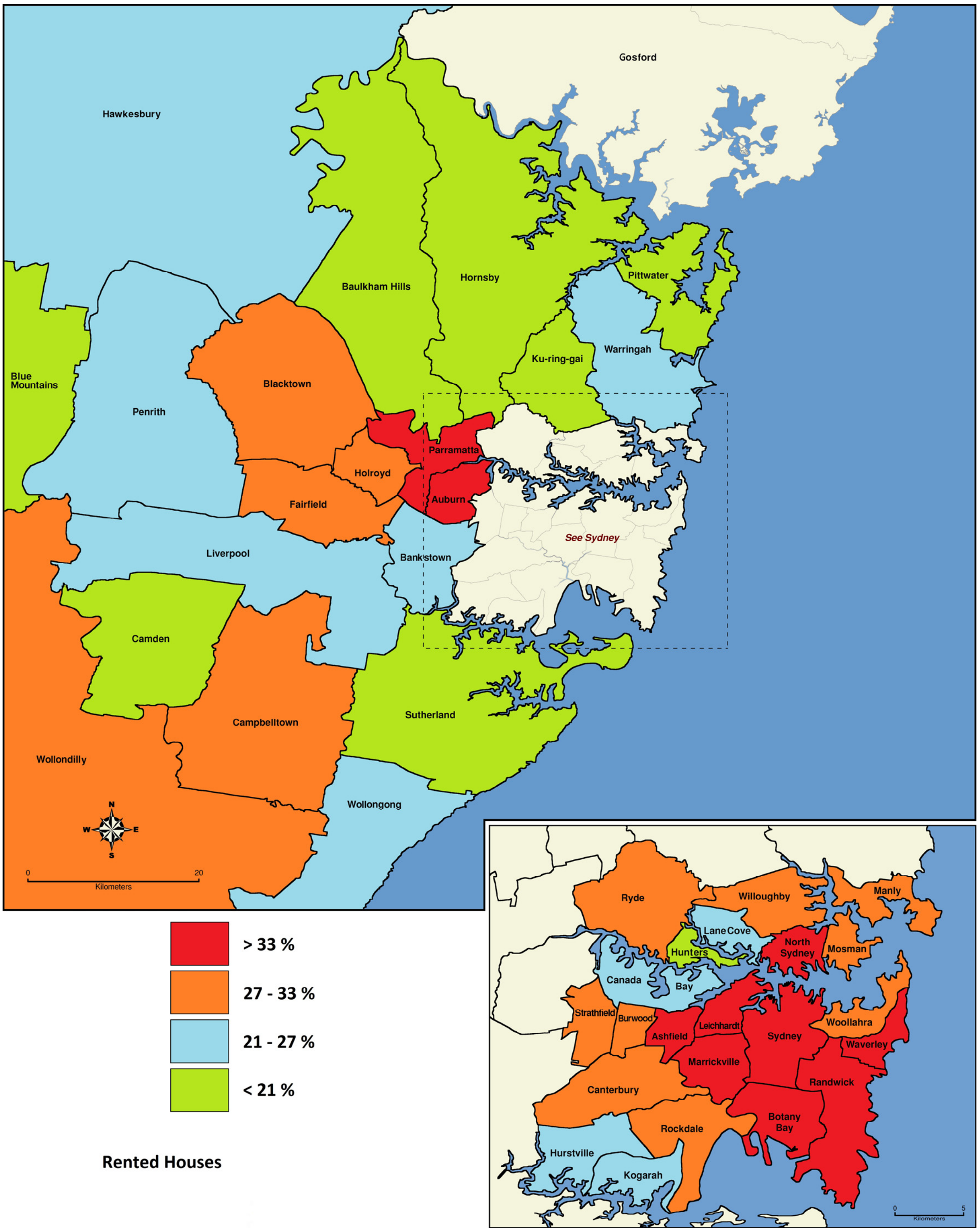

Figure 5.4c Socio-economic indicators in the Sydney metropolitan area – Percentage of houses that are not owner occupied (rented)

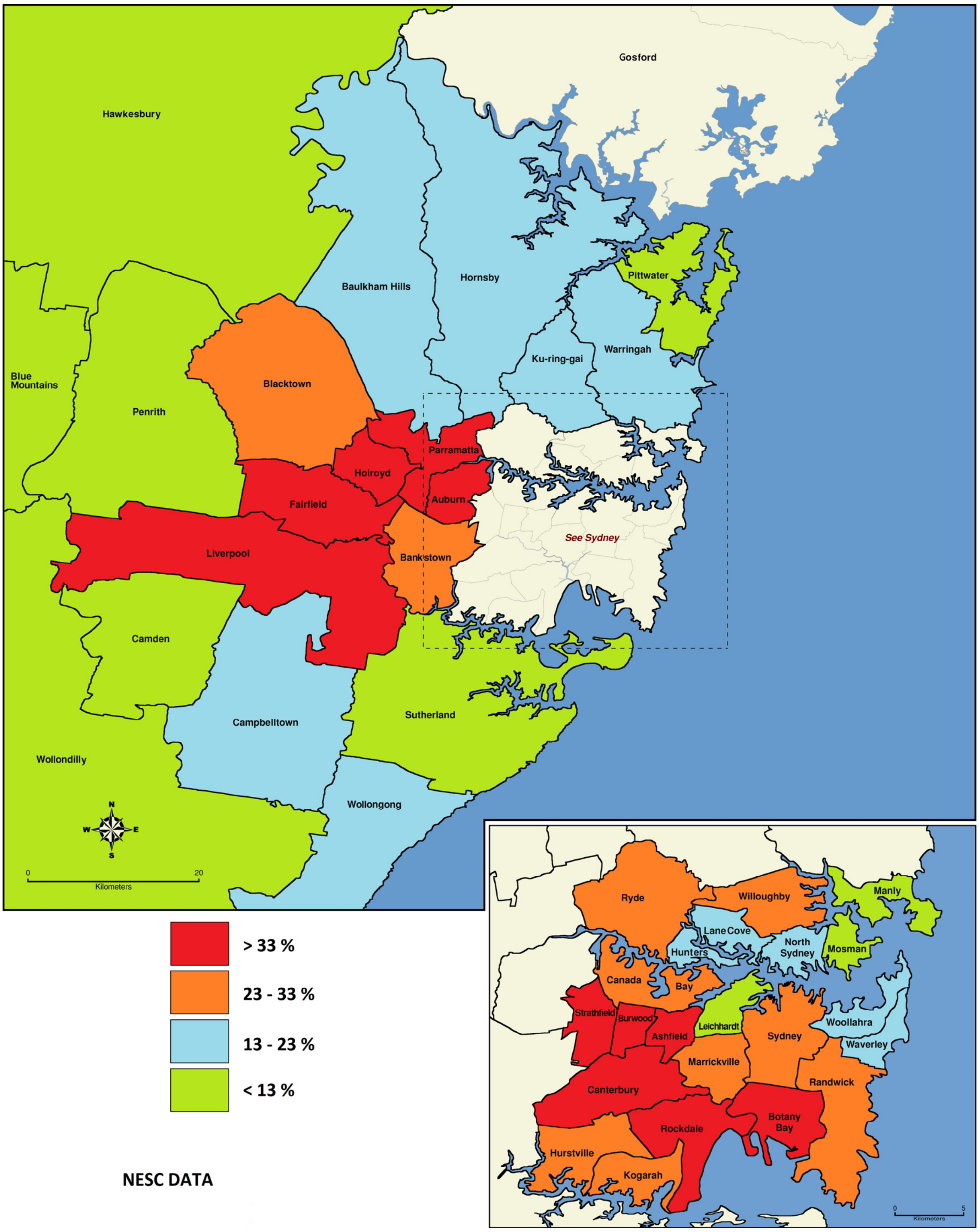

Figure 5.4d Socio-economic indicators in the Sydney metropolitan area – Percentage of residents born in non-English speaking countries (NESC)

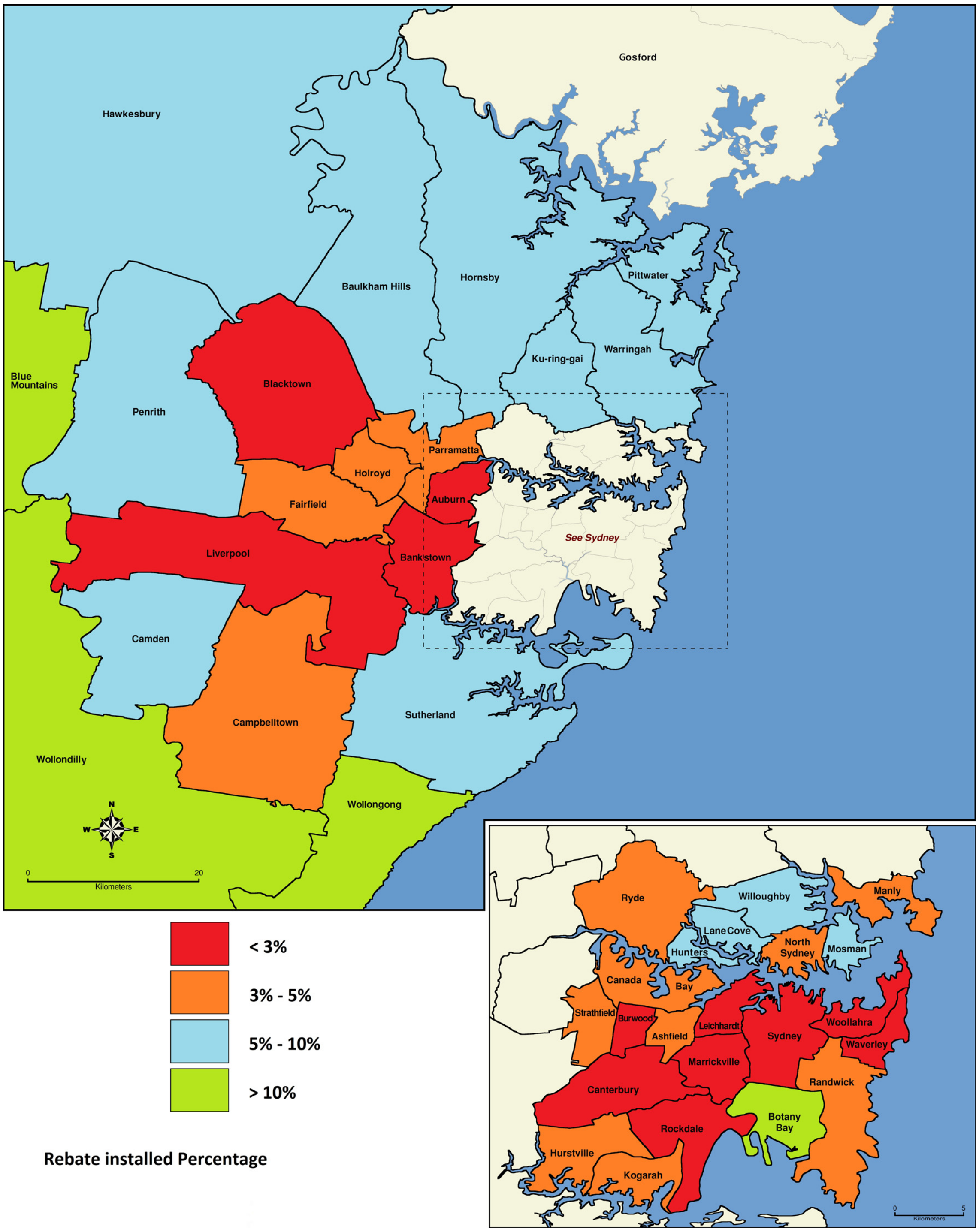

Figure 5.5a Water savings from rebated rainwater tanks in the Sydney metropolitan area – Percentage of rebated rainwater tanks installed

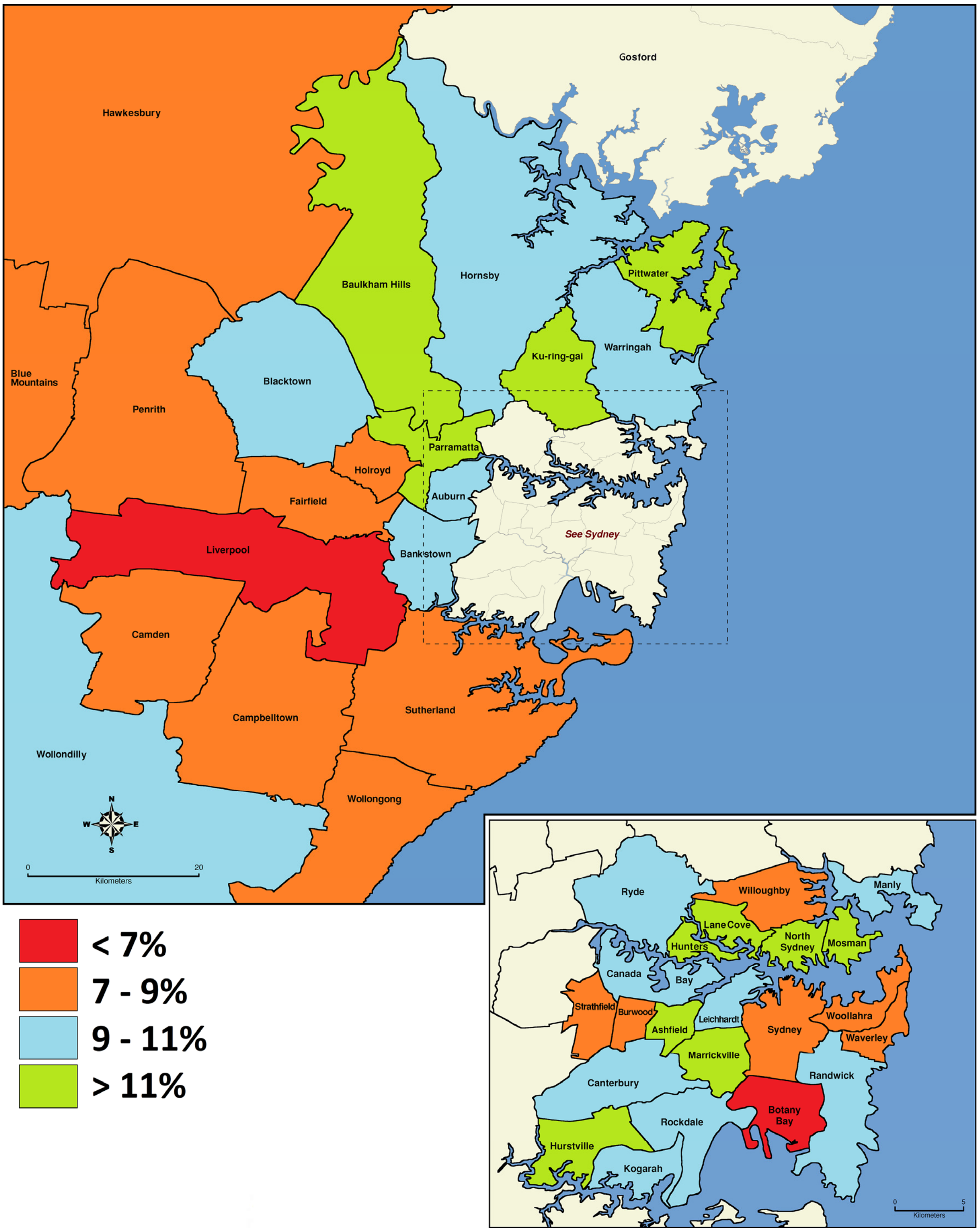

Figure 5.5b Water savings from rebated rainwater tanks in the Sydney metropolitan area – Average percentage of water savings from rebated rainwater tanks

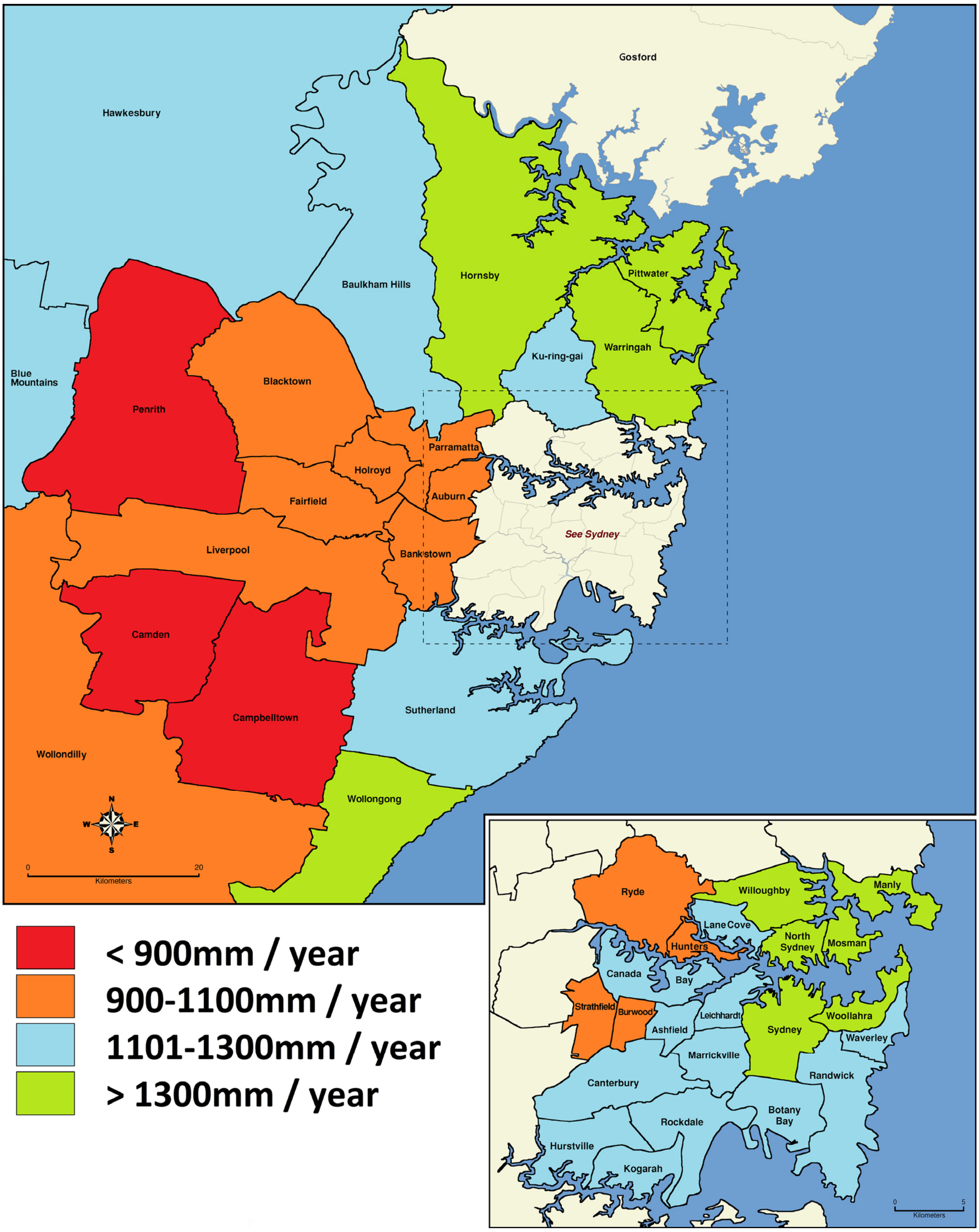

Figure 5.5c Water savings from rebated rainwater tanks in the Sydney metropolitan area – Sydney Metropolitan area long term average rainfall by LGA from 1961 to 1990

Appendix B

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Appendix B - Author's Contribution to the Development of the Carlton Stormwater Harvesting Plant

B.1 **Introduction**

Kogarah City Council has a history of developing joint partnerships with the University of Technology Sydney (UTS) for both student research and the Council’s benefit. Previous project partnerships aimed at saving water, treating and reusing water or preventing pollutants from ruining the environment. It was decided early on that Kogarah Council wanted to harvest water from a canal within its Local Government Authority (LGA) and to use this for potentially potable water grade usage in Carlton’s industrial area. This area is in fact the Council’s primary industrial area and it used a high amount of water annually.

UTS provided services to Kogarah Council in return for it providing UTS allocated space onsite at this harvesting plant to test and research filtration systems on actual real online harvested water. The author of this study was the project manager for this harvesting plant project and was responsible for the design, negotiations, construction and approval of the civil works, the documentation and reporting to the funding grant body, and consultation with the community and stakeholders. Since the plant’s completion the author and other research projectshave utilised the facility.

B.2 **Project summary**

Mayor Nick Katris launched the Carlton Industrial Sustainable Water Program (CISWP) on 2nd^{d} November 2007 in the Kogarah Council Depot. His speech highlighted Kogarah Council and The NSW Environmental Trust’s initiative to foster sustainable management of our precious water resource with local businesses and the community. During the ceremony the Mayor and Sydney Water representatives formalised their partnership by signing a Memorandum of Understanding (MOU).

Initial stakeholder consultation was conducted to deliver detailed program information and formalise the program partnerships. The consultation resulted in 82% of businesses located in Carlton's industrial area joining the program. To date 58 businesses have been audited each year using an innovative environmental audit checklist developed internally, which included a water saving component. From these audits several positive transformations took place in the businesses including: the installation of rainwater tanks, water efficient taps and toilets, onsite water treatment and recycling of waste water and improvements in general business operations to reduce pollution and environmental impact.

Based on the water consumption data provided by Sydney Water, all high water users were identified and preliminary consultation was conducted with business owners/managers about the possibility of using recycled stormwater for industrial purposes. One business that stood out from the rest was the Holcim (formally Cemex) Concrete Batching Plant, which has the capacity to use 3 to 4 ML/year of recycled stormwater. It is located in a prime position across the road from the Lower West Street Reserve stormwater channel.

As a result, an innovative stormwater plant was designed and constructed in the Lower West St Reserve. It has been operating successfully to its design specifications and is showing promising results. Once commissioning is completed stage two of the project will be to under-bore a pipeline to Holcim (formally Cemex) and provide recycled stormwater to supplement and even replace their 3 to 4 ML/year demand for batching concrete.

B.3 Methodology - key project activities undertaken between June 2007 and December 2010

B.3.1 Development, submission & approval of the business plan with the NSW Environmental Trust

After the NSW Environmental Trust and Kogarah Council signed the Grant Agreement on 14th June 2007, CISWP Project Team Members prepared a detailed Business Plan outlining all activities to be undertaken over the project's duration including budget details. A draft Carlton Industrial Sustainable Water Program Business Plan was submitted for review and approval to The NSW Environmental Trust electronically and as a hard copy on 31st July 2007.Kogarah Council amended the Business Plan as requested by The NSW Environmental Trust and it was approved on 23rd October 2007.

B.3.2 Agreements and memoranda of understanding

MOU with Sydney Water

A memorandum of understanding (MOU) was signed during the CISWP launch. Mayor Nick Katris and Fernando Ortega from Sydney Water formally signed the MOU after the Mayor's speech. Kogarah Council prepared a draft MOU outlining both parties' involvement in the CISWP. The document was sent to Sydney Water for review. After both parties were satisfied with the content, and consequently the MOU was signed on 2nd November 2007.

MOUwith Veolia Water Solutions

Veolia Water Solutions participated in the CISWP by providing its revolutionary high-flow stormwater filtering system for a six-month trial. If the results are satisfactory Veolia will offer Kogarah Council the opportunity to purchase the filtering system at a discounted price. Kogarah Council and Veolia Water Solutions both signed the confidentiality agreement and the conditional MOU on 8th September 2008. The MOU was in review after trial runs of the Veolia filtration plant to adjust predicted target filtration values that were agreed on. The updated MOU was signed again on 11th December 2009.

MOU with University of Technology Sydney

Kogarah Council has established a long-term partnership with University of Technology Sydney to institute research project to support the stormwater recycling industry. Specifically the focus has been on developing a high-flow stormwater filtration system. Kogarah Council and UTS signed the MOU on 14th June 2008.

MOU with Streamwatch and Local Schools

Upon DECC's approval of the CISWP Business Plan, Kogarah Council submitted a draft MOU for review to Streamwatch. Due to staff restructuring the document could not be finalised within the time frame identified in the project schedule. The MOU was signed on 17th December 2007.

MOU with Participating Businesses in the Program

To ensure sustainable water management results for the Carlton industrial area, Kogarah Council drafted a MOU between it and local businesses. The MOU outlines the expected participation in the program by both parties including information such as: "In case the business moves to other premises all water saving devices provided by the Council and Sydney Water must remain on premises".

Following the first two consultations, during the first round of the industrial audit business owners were offered additional information on the MOU and encouraged to sign the document. One hundred per cent of the CISWP participating businesses did in fact sign the MOU. However, following the businesses' genuine interest in the CISWP in the first water savings audit, there was a noticeable decline in participation and interest from these businesses during the economic downturn in 2009.

In an attempt to reignite relationships between business owners and this project, Kogarah Council organised a Business Meeting & Presentation to be held in the

Council Depot in the Carlton Industrial Area. It was scheduled for 17th March 2009 from 3:30pm to 4pm. There was no interest from the industrial area's businesses in attending the meeting, which had to be cancelled. Information to be conveyed at the meeting was re-worked into audit material and was delivered to business owners during the second round of audits.

Council revised the original audit forms used in the first round of audits to provide an easier stream lined approach to auditing to reduce spent with each business.The second round of audits was carried out between 8th April and 13th May 2009.

MOU with Holcim Australia Pty. Ltd. (formerly Cemex)

Holcim will be using the recycled water produced from the stormwater reuse plant for batching concrete and washing its trucks. Kogarah Council and Holcim reviewed changes to the MOU from both parties. Final pricing of the recycled water supplied to Holcim was still being modelled and reviewed due to finalising operating and maintenance costs.In early February 2009, Holcim changed its water requirements which meant a more stringent filtration process was required.

On 20th February 2009, Holcim announced that for the foreseeable future, there would be a "freeze" on all capital expenditure for Holcim Australia due to the economic downturn. Holcim remains positive about the project, but is no longer able to provide funds for any capital works on the Carlton plant.

November 2009 – Council contacts Holcim for an update. Cemex is in the process of merging with the new parent company Holcim.

February 2010 – Holcim is ready to start negotiations with the freeze on capital spending being lifted in the near future.

Sydney Water – stormwater harvesting agreement

February 2009 – Council has requested changes to the Sydney Water Corporation (SWC) agreement. SWC officer had resigned. Council is waiting for new contact to get up to speed with the project.

April 2009 – The new contact in SWC is now up to speed with the project. SWC is requesting that before it can approve the stormwater harvesting agreement, further testing is required on the water to prove that it is not from a leaking SWC main.

April 2009 – Council's legal team has reviewed the SWC agreement and requested changes.

May 2009 – Fluoride testing is carried out and sent to laboratory for analysis. Awaiting results

June 2009 – Fluoride testing results were returned to Council and were forwarded to SWC. The level of fluoride in all samples ranged from 0.6 – 0.7 mg/L.

June 2009 – SWC's response is that *"the normal fluoride level in potable water ranges from 0.9 to 1.5 mg/L. Therefore, they advised by one of their technical specialists that it's likely that there is potable water in the seepage which has travelled some distance and mixed with another water source. Another (more remote) possibility is that the seepage may contain sewage, but it's unlikely unless it smells and there is a high faecal coliform count."*

Council believes that the low flow in the canal is simply ground water and will continue with the stormwater harvesting plant.

June – December 2009 – Council's & SWC's legal teams have been discussing issues and come to a mutual agreement.

January 2010 – Council reviews final agreement as SWC requests additional insurance clause. Council's legal team begins applying for required insurance.

4th March 2010 – Council receives insurance cover and is ready to sign the agreement.

5th March 2010 – Council has signed and returned the agreement. Awaiting SWC's approval to start operating the stormwater harvesting plant.

B.3.3 License to harvest water from the Department of Water & Energy (DWE)

March 2009 – Council applied to DWE for a license to harvest water from the canal.

February 2010 – Council has received the written approval for the water license.

B.3.4 Macro invertebrate testing report

8th May 2009 – Council's environmental scientist has revised the macro invertebrate testing procedure before commencing monitoring.

11th May 2009 – Council carried out macro invertebrate testing on the subject canal and a similar canal’s receiving waters.

10th November 2009 – Council carried out round 2 of the macro invertebrate testing on the subject canal and a similar canal’s receiving waters.

The results show that no macro invertebrates were living near the outlets of both stormwater channels.

B.3.5 Stormwater harvesting plant – final plant design, construction & commissioning

Below is a chronology of the important events that took place and leading to the project’s completion. The use of the word Council herein represents the author from the University of Technology Sydney acting on behalf of Kogarah City Council. The construction issue documentation that the author prepared is provided in Figures B.10 to B.17.

21st November 2008 – Council has applied to Energy Australia for the electrical connection onsite within the reserve.

2nd December 2008 – Council has applied for a trade waste agreement to allow the discharge of filter backwash wastewater to be discharged to the Sydney Water sewer main. Sydney Water has advised that a conditional consent for the trade waste agreement will be issued for 12 months from the time of commissioning the filtration plant. This conditional consent will be revised at the end of 12 months and be based on the recorded quantity of flows and pollutant concentrations.

12th December 2008 – Council has prepared plans for the design of the stormwater harvesting plant.

23rd March 2009 – Council has applied to the DWE for a license to harvest water from the canal.

25th March 2009 – Through new investigations and design, Council has designed a rubber speed hump / bitumen hot-mix weir-like structure with smooth curved surfaces and easy maintenance that will overcome problems with debris and blockages.

Council has sourced the required materials to construct the weir and proposed the new water extraction method to Sydney Water.

Following the denied application to construct a weir in the SWC's canal in March 2009, Council proceeded with a new sump design for the canal.

April 2009 – Veolia Water Solutions (VWS) has decided to move the plant's location due to the risk of flooding. Council begins to investigate an alternative location.

Council has prepared plans to investigate directional boring technology with MSW Plant Hire on 8th April 2009. Unfortunately the site is not large enough to utilize this technology (14th April 2009). MSW Plant Hire has referred Council to Auss Boring Pty. Ltd. who can use a mini-borer. Plans were prepared and posted on 14th April 2009.

8th May 2009 – Auss Boring has instructed Council that boring is not feasible due to the amount of excavation required. Auss Boring has also instructed Council to seek approval from Sydney Water to excavate up to the Canal Wall and carry out hand boring to place pipe work in the sump located in the canal.

27th May 2009 – Council sends plans to SWC for approval to construct a sump in the canal. SWC requests changes and more details.

11th June 2009 – Council sends amended plans to SWC for approval.

August 2009 – The original design was re-investigated to place the stormwater harvesting plant at a lower elevation onsite situated in a flood affected area and to accept the risks. Water proofing of the proposed cabinets was investigated as an alternative.

1st September 2009 – Community consultation on the new design. No objections were made to the design. Proposed tree planting plan for residents.

2nd November 2009 – SWC approves Council's plan of works in the canal.

20th November 2009 – Temporary Fence Hire erected site fencing and Kogarah Council Works (KCW) erected Tree Protection Zone fencing and commenced excavation to locate existing stormwater pipe.

Figure B.1 Lower West Street Reserve, Carlton before commencement of works & clearing of trees

Figure B.2 Excavation of site for sump and pipe installation

Figure B.3 Installation of conduits for hydraulic line connections though slab

Figure B.4 Installation of product water, control valve and return pits and sump pit

Figure B.5 Completion of canal works and control panel inside plant cabinet

15th February 2010 – VWS cabinet and Plant cabinet is delivered and installed.

Figure B.6 Delivery and installation of Harvesting Plant cabinets

4th March 2010 – VWS have finalised control valves and monitoring equipment installation in the control valve pit. Council has installed cage protection over backflow prevention device and fabricated control valve pit steel protection cover.

Figure B.7 Completion of control valve pit hydraulics

Figure B.8 Installation of sampling taps for raw water and various stages of filtered water

9th March 2010 – SWC onsite inspection to approve the trade waste connection. SWC will post trade waste consent within one week with monitoring and sampling requirements.

17th March 2010 – VWS started commissioning the stormwater reuse plant and commencing the 6 month trial of its pilot plant. Unfortunately there was a hydraulic restriction in the control valve pit that prevented the plant from operating at its required specifications.

From March until 28th June, meetings between Council, Veolia Water Solutions and ITT were held to sort out how to rectify the problem. A new hydraulic design was drafted, new parts were ordered and the commissioning of the plant recommenced on 28th June 2010.

The site has now been reinstated with trees planted and turf laid.

Figure B.9 Completed Harvesting Plant with security fencing and re-vegetation of site

B.3.5.1 **Issues that held up the project**

- The duration it took Council’s and Sydney Water Corporation's legal teams to come to a mutual agreement over the stormwater harvesting agreement.

- SWC taking time to respond to construction requirements when Council wanted to construct the harvesting plant over the SWC sewer main.

- DWE / DECCW taking 9 months to send the written approval for the licence to harvest water before we could commence construction.

- The Holcim (formally Cemex) concrete batching plant not being able to provide funds for any capital works due to the economic downturn and having a new owner.

- Hydraulic design fault of the stormwater reuse plant, requiring a replacement part to be ordered which took one month and the reconfiguration of the entire control valve pit.

B.3.5.2 **Future opportunities**

- The stormwater reuse connection is still planned to go ahead with Holcim

- Council organises and runs periodic educational school and business fieldtrips to many of Kogarah Council's previous projects such as the Beverly Park Sewage Treatment Plant. The CISWP will be added to the one of the places visited during these field trips to teach the community about stormwater harvesting, stormwater pollution and protecting our water ways.

- The CISWP project has created the facility within the stormwater reuse plant so that further research can be conducted with the University of Technology Sydney on stormwater filtration.

- The relationship with the Environmental Health Unit has created ongoing business education and audit programs.

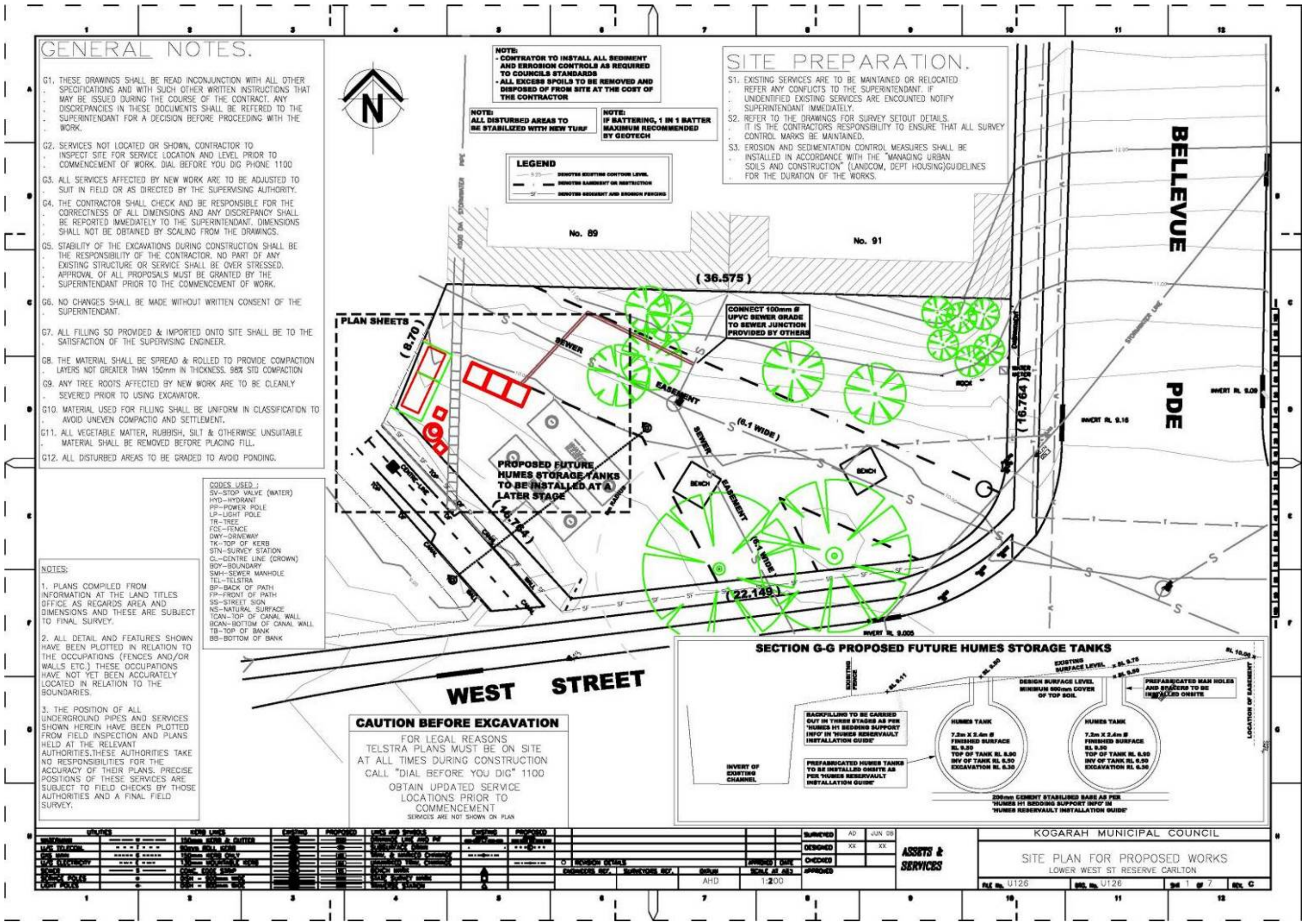


Figure B.10 Construction issue documentation concerning the stormwater harvesting plant

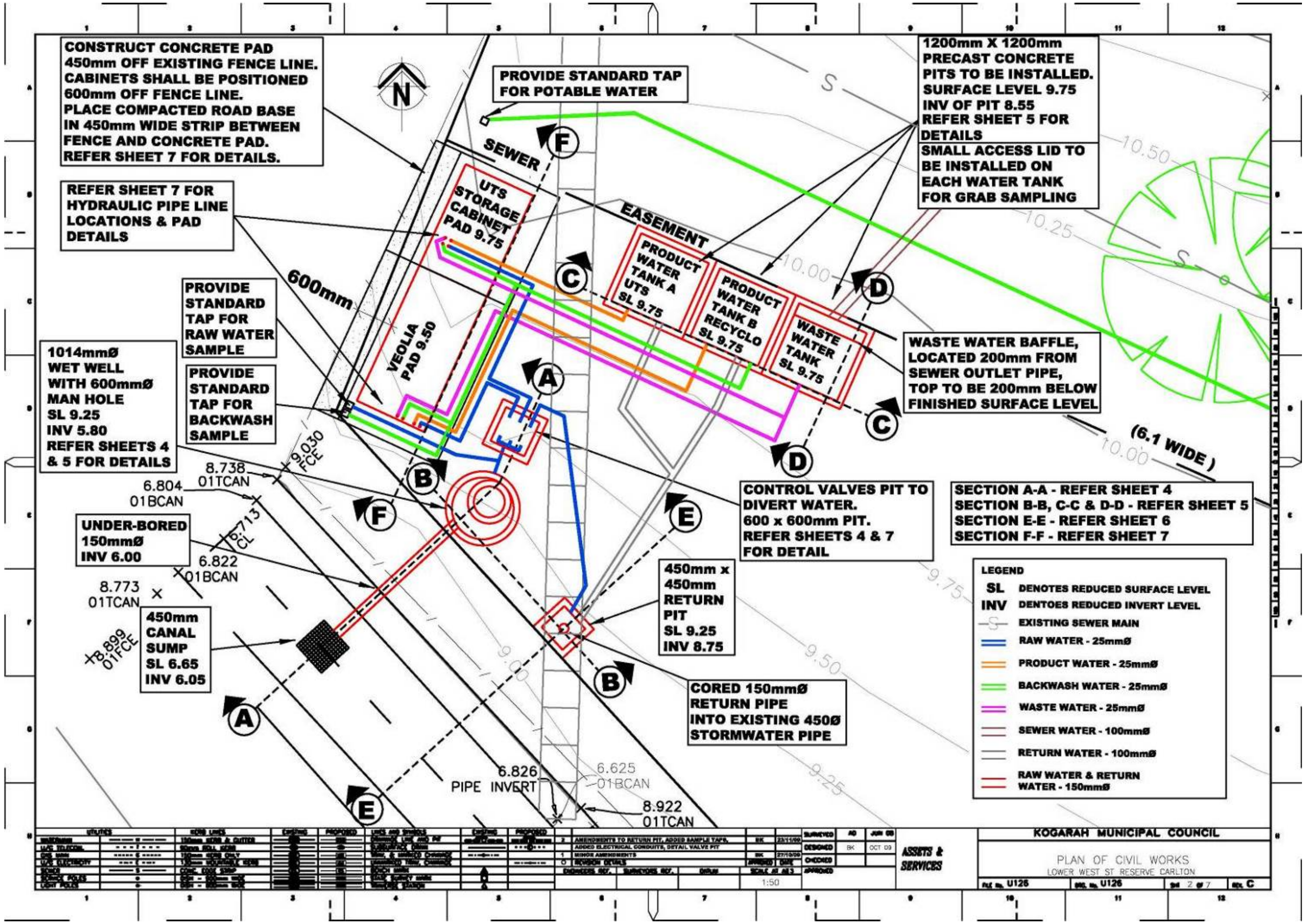
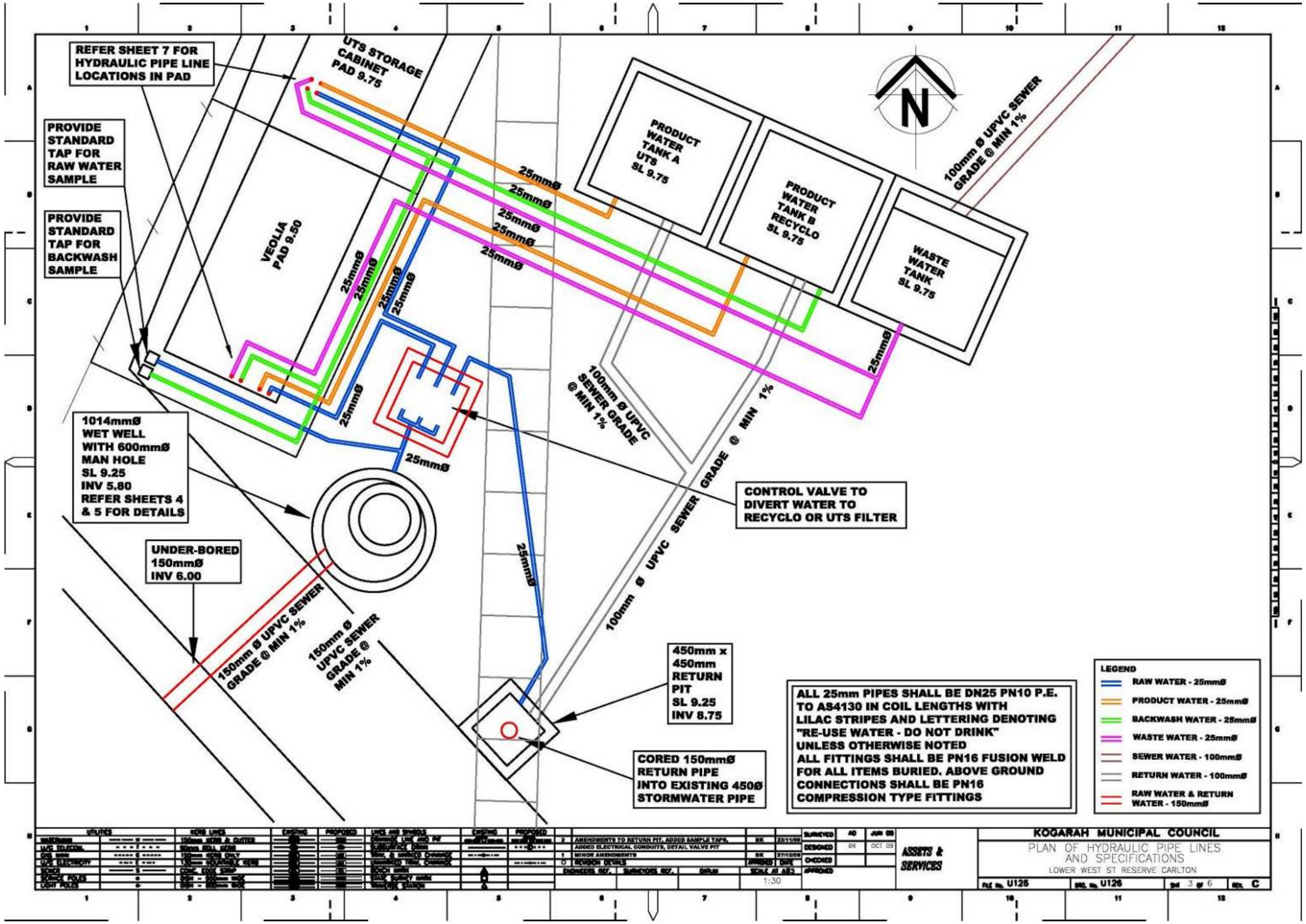


Figure B.11 Construction issue documentation concerning the stormwater harvesting plant



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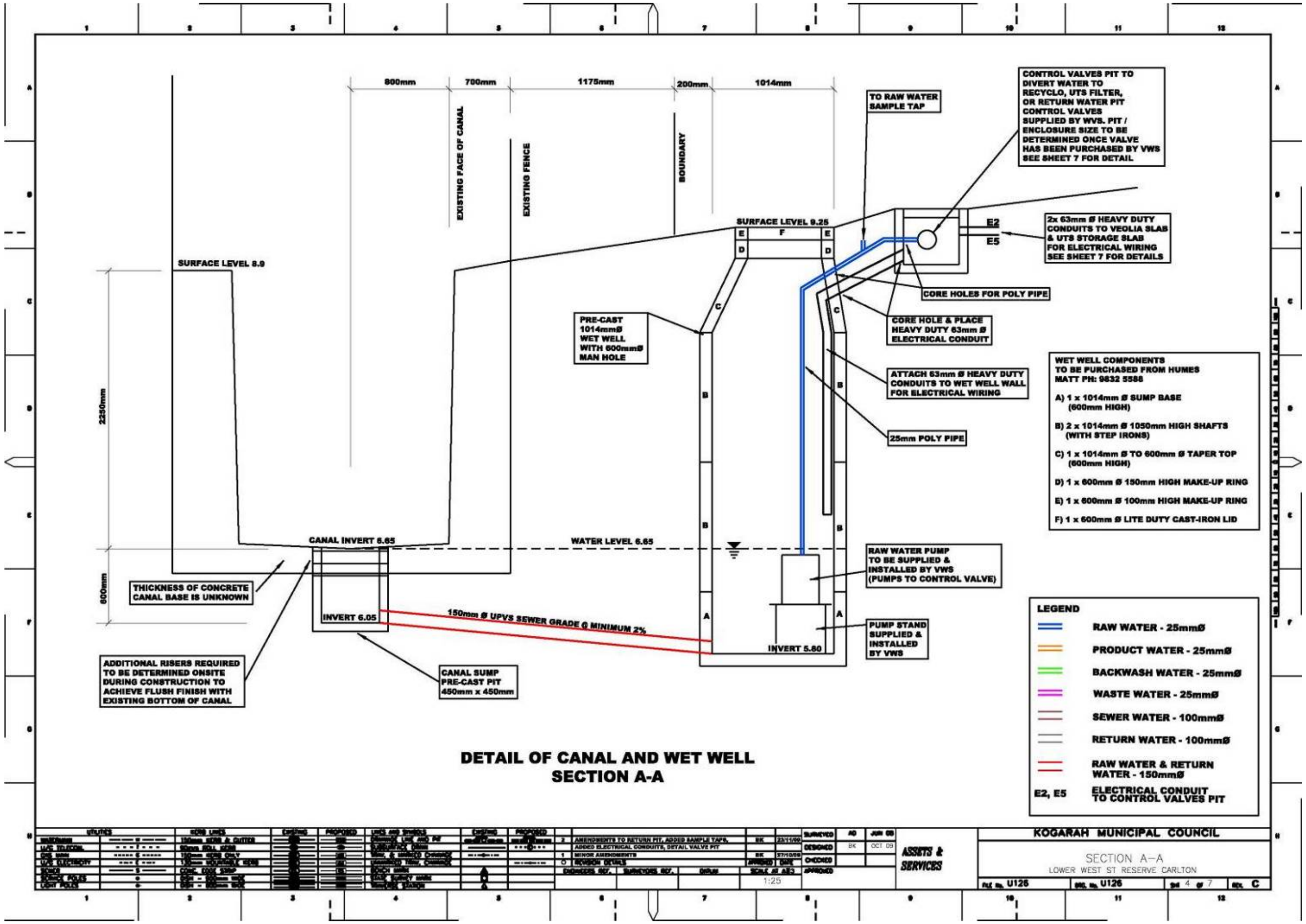

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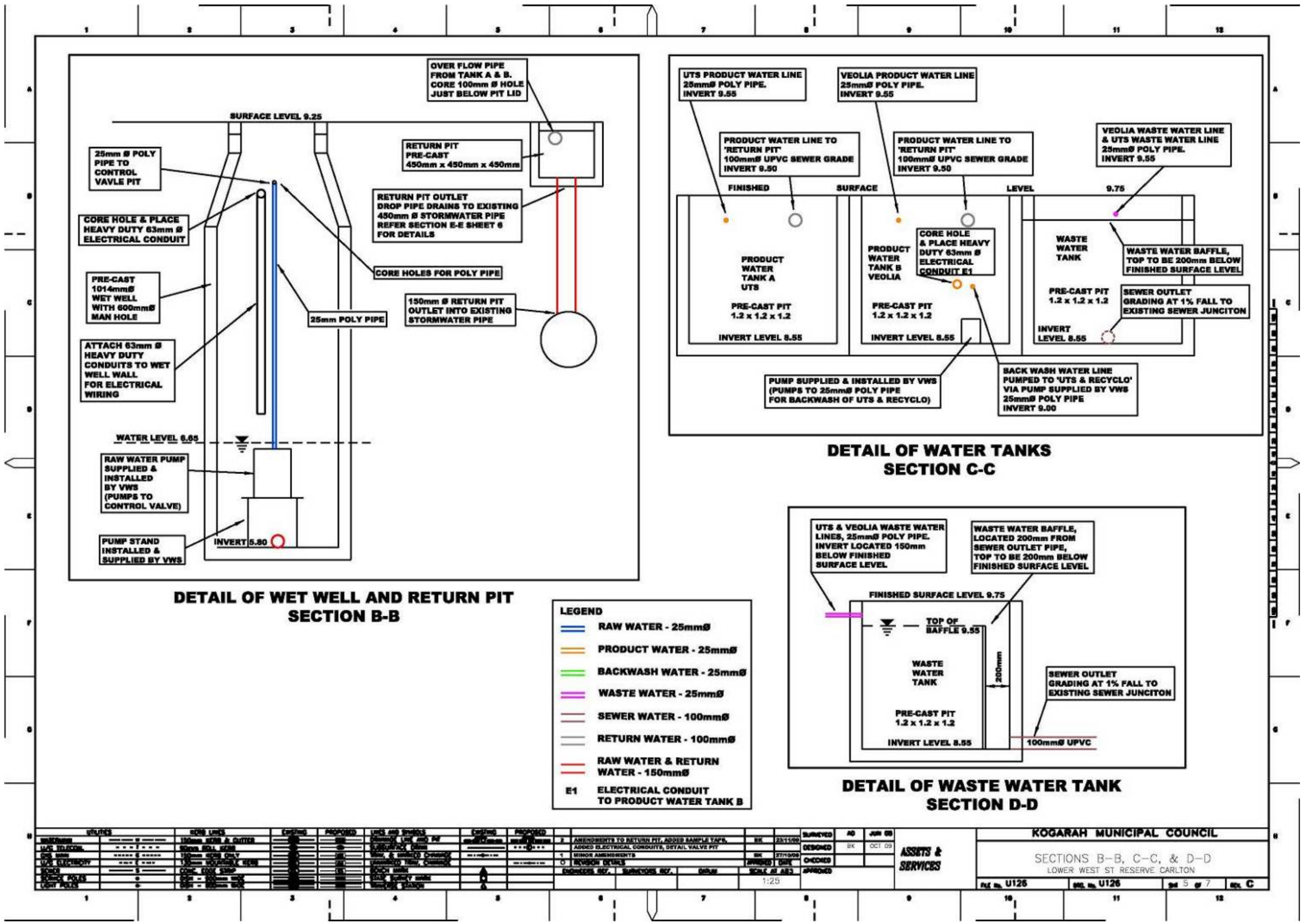

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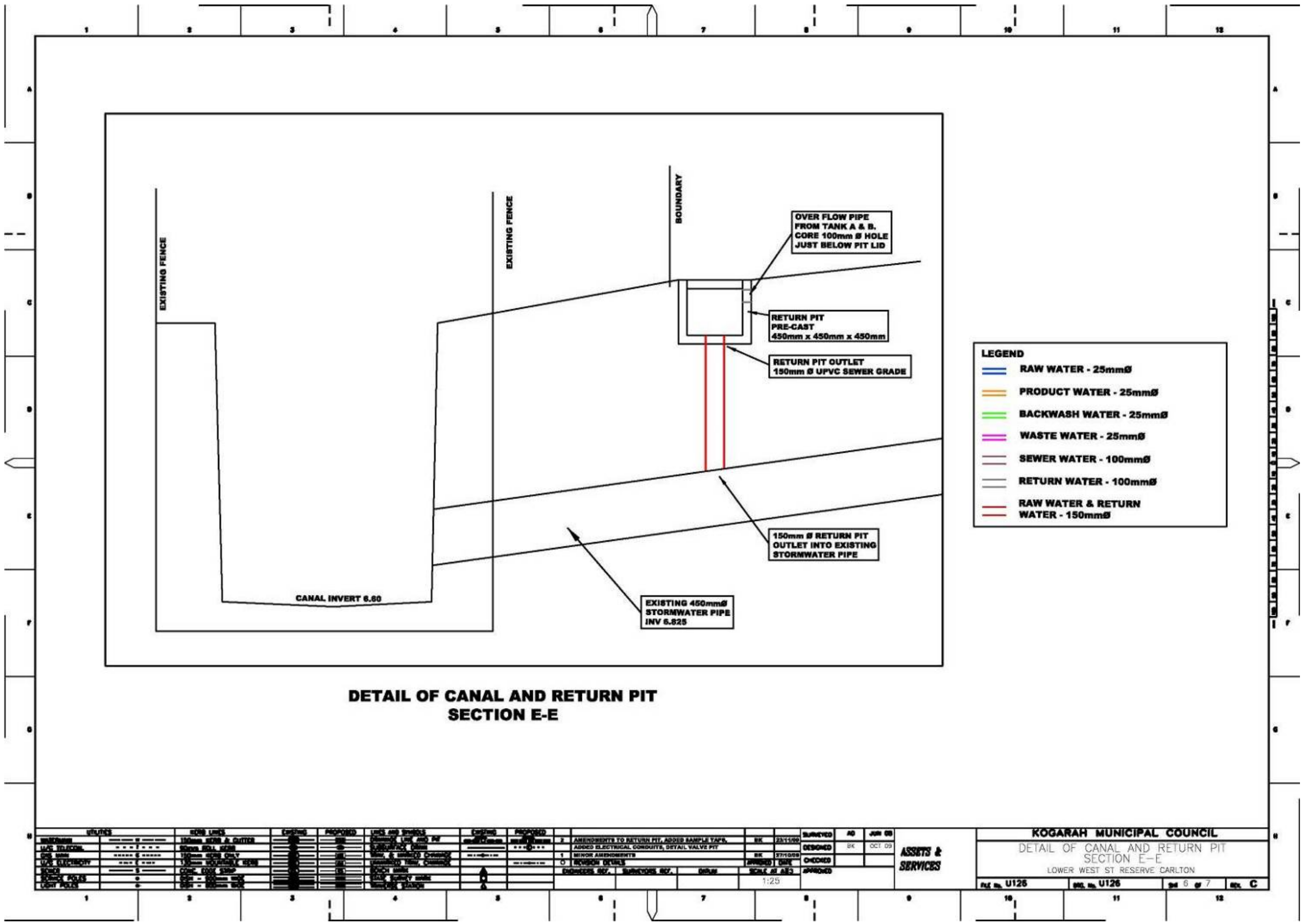

Figure B.15 Construction issue documentation concerning the stormwater harvesting plant

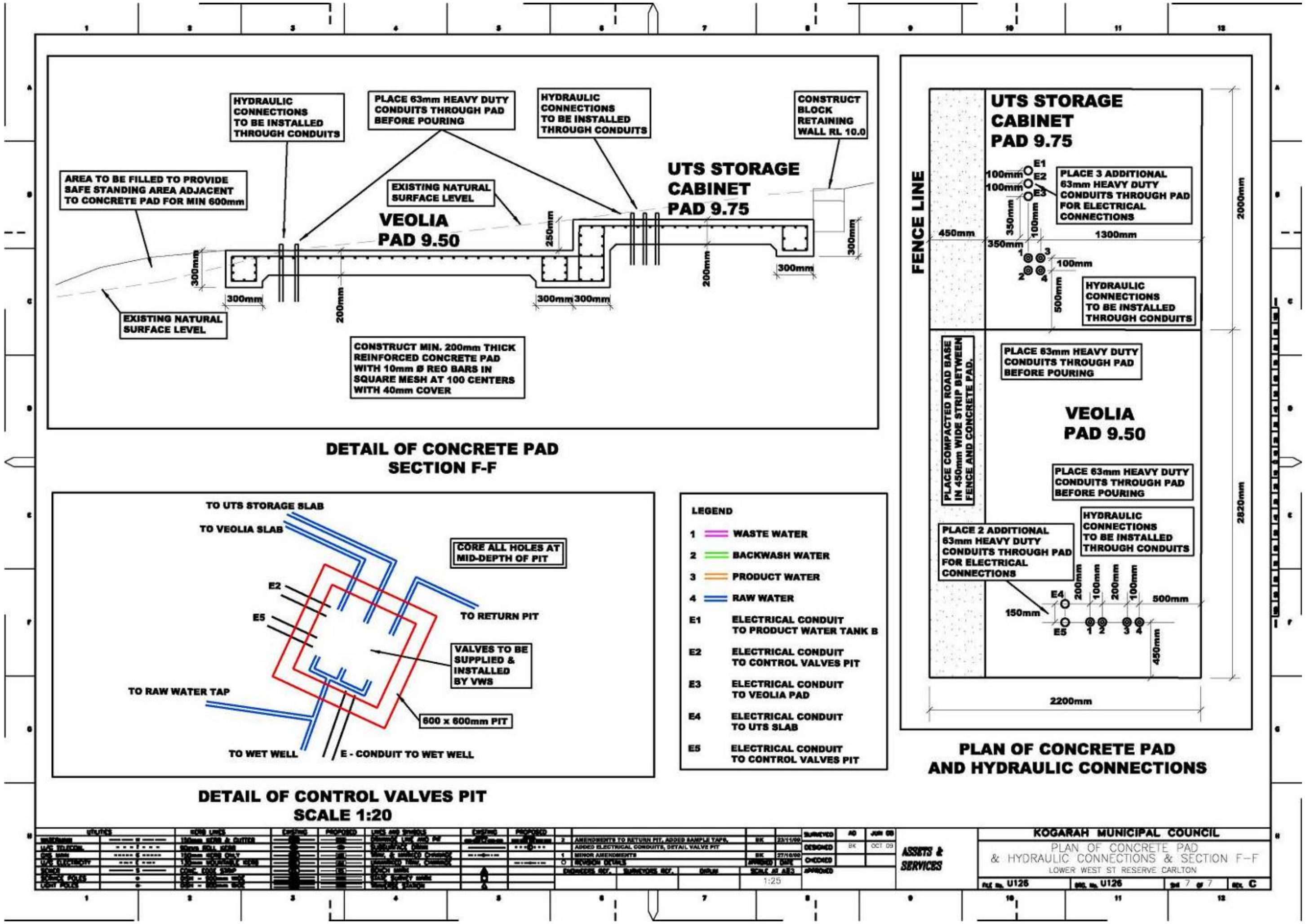

Figure B.16 Construction issue documentation concerning the stormwater harvesting plant

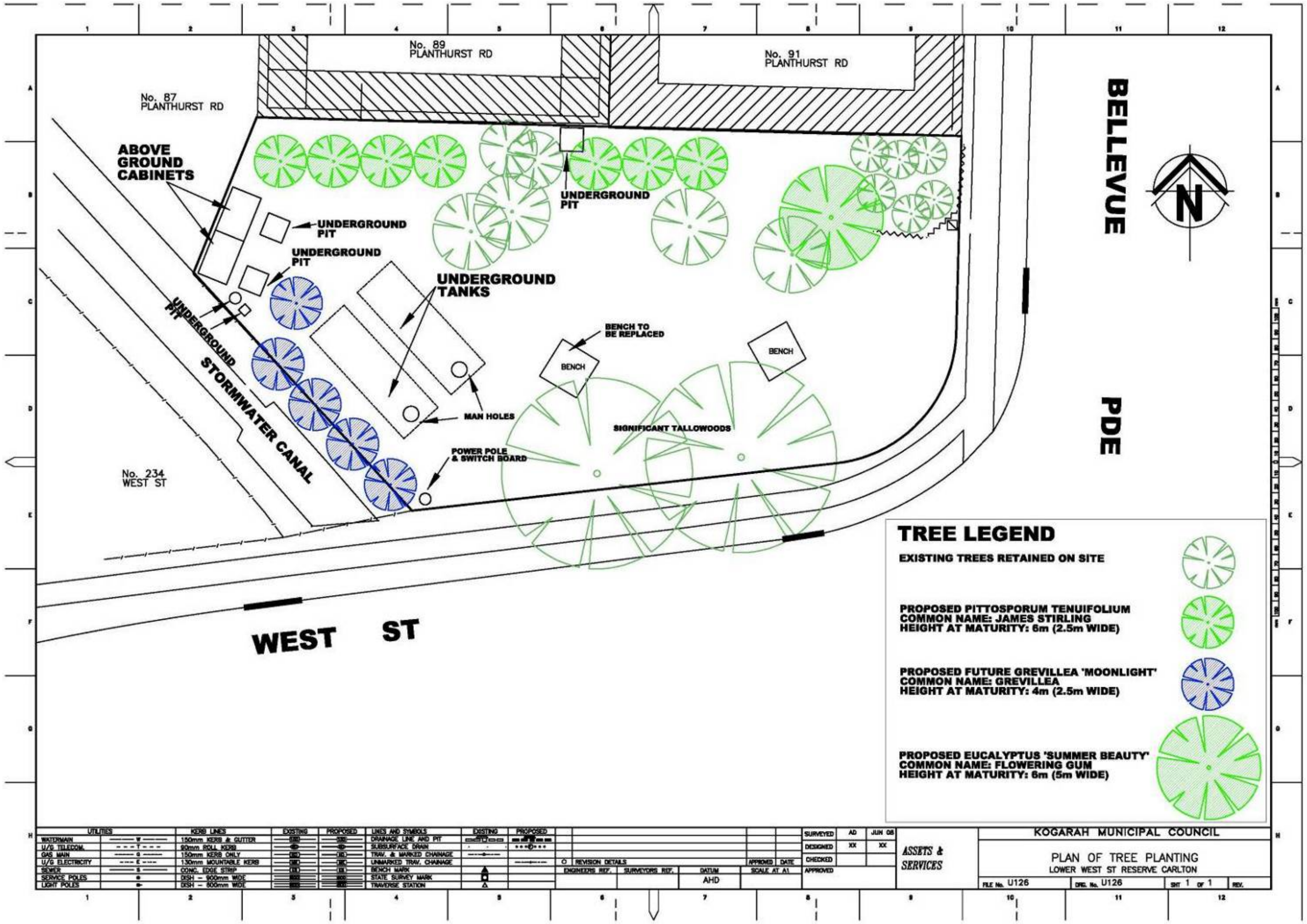

Figure B.17 Construction issue documentation concerning the stormwater harvesting plant

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