CIV6000Z: DISSERTATION FOR THE DEGREE OF DOCTOR OF PHILOSOPHY

The viability of rainwater and stormwater harvesting in the residential areas of the Liesbeek River Catchment, Cape Town

Prepared by:

Lloyd Norman Fisher-Jeffes BSc. (Eng), MSc (Eng)

Supervised by:

Professor Neil P Armitage Pr Eng, PhD

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Department of Civil Engineering
University of Cape Town, Private Bag Rondebosch, 7700
South Africa 7700

'Don't let anyone look down on you because you are young, but set an example for the believers in speech, in life, in love, in faith and in purity.'
(1 Tim 4 vs. 12 NIV)

'No company would stay in business long if its management did not know how much product was being produced, how much it cost to produce it, or the market price for the product... Why should we treat our natural capital – capital that sustains life on the planet – any differently?'
(Olewilder, 2004)

'I shall pass through this world but once. Any good therefore that I can do or any kindness that I can show to any human being, let me do it now. Let me not defer or neglect it, for I shall not pass this way again.' – Stephen Grellet

Mine eyes have seen the glory of the coming of the Lord; He is trampling out the vintage where the grapes of wrath are stored; He hath loosed the fateful lightning of His terrible swift sword: His truth is marching on.

Glory, glory, hallelujah! Glory, glory, hallelujah! Glory, glory, hallelujah!
His truth is marching on.
(Julia Ward Howe, 1861)

Declaration

I, Lloyd Norman Fisher-Jeffes, know the meaning of plagiarism and declare that all the work in this document, save for that which is properly acknowledged, is my own. A list of data sources (along with the associated copyrights and disclaimers) for figures making use of geographic images and/or shapefiles is included in Appendix P.

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Date: 08/12/2015

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Abstract

The sustainable provision of water to South African citizens is a significant challenge facing the country. In order to avert a crisis, municipalities will need to reduce their reliance on traditional water sources. Rainwater harvesting (RWH) and stormwater harvesting (SWH) are two alternative water resources that could supplement traditional urban water supplies. To date, the potential benefits of RWH and SWH within an urban setting have not been adequately considered or investigated in South Africa.

The only way to quantify the benefits and potential viability of rainwater and stormwater harvesting was to select and model a representative catchment – the Liesbeek River Catchment, Cape Town South Africa was selected. An Urban Rainwater Stormwater Harvesting Model was developed to model the use of RWH and SWH in the catchment. Additionally, a Storm Water Management Model (SWMM) of the catchment was developed to investigate the stormwater management benefits of RWH and SWH.

The study found, *inter alia*, that: RWH was viable for only a minority of property owners; climate change would have limited impact on the performance of RWH systems; and RWH is an unreliable – even for small storm events – means of attenuating peak flows. On the other hand, SWH has the potential to reduce potable water demand in the Liesbeek River Catchment by up to 20%. However, for SWH to be viable there would need to be a high level of adoption by residents, at least for non-potable uses such as flushing toilets and outdoor irrigation. SWH is also of benefit in the attenuation of peak flows during storm events. Finally, the research found that the implementation RWH and SWH together would be unwise, as both are most cost-effective under conditions of maximum demand.

The study concluded that SWH could be a viable alternative water resource for urban residential areas in South Africa – depending on the scale at which it is implemented, the end use for which it is utilised, and the population density that drives the water demand. RHW, on the other hand, has limited potential – depending on climatic conditions; it may, for example, be viable in areas with year-round rainfall.

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Glossary

The definitions below refer to the use of terms in this thesis and are based on the definitions in Armitage et al. (2013), the South African Guidelines for Sustainable Drainage Systems.

Aquifer is a porous, water-logged sub-surface geological formation. The description is generally restricted to media capable of yielding a substantial supply of water.

Attenuation means the reduction of peak flow.

Catchment refers to the area contributing runoff to any specific point. This could refer to a roof draining to a down pipe, or an urban area draining to a wetland.

Climate change is a continuous phenomenon and refers to the change in global climatic conditions.

Conveyance is the transfer of runoff from one location to another.

Depression storage refers to precipitation stored in surface depressions.

Drainage system refers to the network of channels, drains, hydraulic control structures, levees, and pumping mechanisms that drain land or protect it from potential flooding.

Impervious surface refers to surfaces which prevent the infiltration of water. Roads, parking lots, sidewalks and rooftops are examples of impervious surfaces.

Interception storage refers to precipitation stored on vegetation.

Lag time is defined as the time from the centroid of the excess rainfall to the peak of the associated runoff hydrograph.

Life Cycle Cost refers to the costs of a structure / asset throughout its operating life.

Permeability refers to the ability of a material to allow water to flow through when fully saturated and subjected to an unbalanced pressure.

Precipitation is the water received from atmospheric moisture as rainfall, hail, snow or sleet, normally measured in millimetres depth.

Rainwater harvesting is the collection, storage of runoff from the roof/s present on an individual property and the subsequent use within that property – including both indoor and outdoor uses.

Recurrence interval or return period is the average interval between events exceeding a stated benchmark. The recurrence interval is usually expressed in years and is the reciprocal of the annual probability – that is, the event having an annual probability of occurrence of 2% (0.02) has a recurrence interval of 50 years. This does not imply that such an event will occur after every 50 years, or even that there will necessarily be one such event in every 50 years, but rather that over a very long period (e.g. 1000 years), assuming no climate change, there will be approximately 20 events of greater magnitude (1000/20 = 50 years).

Runoff generally refers to the excess water that flows after precipitation.

Sedimentation is the deposition of soil particles that have been carried by flowing waters, typically during flood peaks as a consequence of a decrease in the velocity of flow below the minimum transportation velocity.

Stormwater drainage system is constituted by both the constructed and natural facilities including: stormwater pipes, canals, culverts, overland escape routes, 'vleis', wetlands, dams, lakes, and other watercourses, whether over or under public or privately owned land, used or required for the management, collection, conveyance, temporary storage, control, monitoring, treatment, use and disposal of stormwater.

SuDS is the abbreviation for sustainable urban drainage systems or sustainable drainage systems, which are a sequence of management practices and/or control structures or technologies designed to drain surface water in a more sustainable manner than conventional techniques.

Stormwater harvesting is the collection and storage of runoff from an urban area, and the subsequent redistribution for use by one or more independent users for any appropriate purpose. For example: garden irrigation and/or toilet flushing.

Surface runoff is that part of the runoff that travels over the ground surface and in channels to reach the receiving streams or bodies of water.

Time of concentration is the time required for water to flow from the most hydraulically remote point of the basin to the point / location of analysis.

Acronyms and abbreviations

AADD	Average Annual Daily Demand
AHMC	Australian Health Ministers' Conference
AMDD	Average Monthly Daily Water Demand
AR	Artificial Recharge
Aus	Australia
AWRMS	Atlantis Water Resource Management Scheme
BCA	Benefit Cost Analysis
CEA	Cost Effectiveness Analysis
CIDB	Construction Industry Development Board
CoCT	City of Cape Town
CoR	Cost of Replacement
CPAF	Contract Price Adjustment Formulae
CSIR	Council for Scientific and Industrial Research
CSM	Catchment Stormwater Model
CSRM	Catchment, Stormwater and River Management Branch
CUA	Cost Utility Analysis
CVM	Contingent Valuation Method
DAFF	Department of Agriculture, Forestry, and Fisheries
DCD	Department of Community Development
DEA	Department of Environmental Affairs
DECNSW	Department of Environment and Conservation New South Wales
DEM	Digital Elevation Model
DLGP	Department of Local Government and Planning
DMF	Decision Making Framework
DoCGTA	Department of Cooperative Governance and Traditional Affairs
DOE	Department of Energy
DPLG	Department of Provincial and Local Government
DVD	Digital Video Disc
DWAF	Department of Water Affairs and Forestry
ECM	Economic Calculation Module
EGS	Ecosystem Goods and Services
EMC	Event Mean Concentration
EV	Equivalent Volume

GI	Green Infrastructure
GIS	Geographic Information System
IPCC	Intergovernmental Panel on Climate Change
IUWM	Integrated Urban Water Management
IWRM	Integrated Water Resource Management
LCC	Life Cycle Cost
LID	Low Impact Development
LiDAR	Light Detection and Ranging
LIUDD	Low Impact Urban Design & Development
MAR	Managed Aquifer Recharge
MBWCP	Moreton Bay Waterways and Catchments Partnership
MDG	Millennium Development Goals
MUSIC	Model for Urban Stormwater Improvement Conceptualisation
MWD	Monthly Water Demand
NOAA	National Oceanic and Atmospheric Administration
NPV	Net Present Value
NRMMC	Natural Resource Management Ministerial Council
NRW	Non-Revenue Water
NWA	National Water Act
NWRS	First National Water Resources Strategy
NWRS2	Second National Water Resource Strategy
NWSA	National Water Services Act
OF	Objective Function
PGWC	Provincial Government of the Western Cape
QMRA	Quantitative Microbial Risk Assessment
RCPs	Representative Concentration Pathways
REUM	Residential End Use Model
RSA	Republic of South Africa
RTC	Real-Time Control
RWH	Rainwater Harvesting
SABS	South African Bureau of Standards
SAWS	South African Weather Service
SEQ	South East Queensland
SLAMM	Source Loading And Management Model
StatsSA	Statistics South Africa

SuDS	Sustainable Drainage Systems / Sustainable Urban Drainage Systems
SUSTAIN	System for Urban Stormwater Treatment and Analysis Integration
SWH	Stormwater Harvesting
SWMM	Storm Water Management Model
TARWR	Total actual renewable water resources
TBL	Triple Bottom Line Analysis
TCM	Travel Cost Method
TWCM	Total Water Cycle Management
TWD	Total Water Demand
UKRHA	United Kingdom Rainwater Harvesting Association
UN	United Nations
UNDP	United Nations Development Programme
UNEP	United Nations Environmental Program
UNESCO	United Nations Educational, Scientific and Cultural Organization
URSHM	Urban Rainwater/Stormwater Harvesting Model
USA	United States of America
USEPA	United States Environmental Protection Agency
UV	Ultra-violet
UVQ	Urban Volume And Quality
UWC	Urban Water Cycle
WBMOEE	WBM Oceanics and Ecological Engineering
WC/WDM	Water Conservation and Water Demand Management
WGMS	World Glacier Monitoring Service
WHO	World Health Organisation
WRC	Water Research Commission
WSC	Water Sensitive Cities
WSDP	Water Services Development Plan
WSS	Water Sensitive Settlements
WSSD	World Summit on Sustainable Development
WSUD	Water Sensitive Urban Design
WWAP	World Water Analysis Partnership
WWTW	Waste Water Treatment Works
YAS	Yield After Spillage
YBS	Yield Before Spillage
ZAR	South African Rand (Currency)

Symbols

A	Area (m^2)
a	Coefficients of evaporation determined by regression analyses or by visual fitting
AA_c	Adjusted catchment area
AC	Annualised cost
$A_{c/r}$	Area of the catchment/effective roof area
AD_{imp}	Adjusted impervious depression storage
ae	Presence of pool filter
$AImp_c$	Adjusted fraction of catchment that is impervious
$APERC_{perv}$	Adjusted percentage of impervious runoff routed to pervious
b	Event filtering volume
CC	Capital cost
CCK	Capital Cost per kilolitre
c	Coefficient for frequency of use
CR	Runoff coefficient
d	Days
Df	Discount factor
$D_{imp/r}$	Depression storage of impervious area / roof (mm)
e	End use
ER	Evaporation from an open water surface / reservoir
ER_t	The effective runoff from the catchment (after initial and runoff losses) in time t
E_t	The evaporation from the storage unit during time t
ET_h	Evapotranspiration calculated using Hargreaves Method (mm)
ET_o	Evapotranspiration (mm)
EV	Equivalent volume
FCR	Filter coefficient
FF_t	Volume of water not bypassing the first
FF_{vol}	The volume of storage available in the first
FI_{Rain}	The filter inflow in time t
$f_{m,e}$	Pool cover factor / garden irrigation factor
FO_t	The filter outflow in time t
FV	Future value
i	Discount rate

IE	Irrigation efficiency factor
IL_t	Depth of initial losses in time t
Imp_c	Fraction of catchment that is impervious
I_t	Runoff volume (m^3)
k_{CET}	Coefficient for catchment evapotranspiration
$k\ell$	Kilolitre
$k\ell/hh.mnth$	Kilolitres per household a month
$k_{m,e}$	Crop factor or pool / reservoir / open water body factor
LC	Effective roof area loss factor (%)
L_t	Seepage and/or leakage losses during time t
m	Monthly
m^3	Cubic meters
$M\ell$	Mega litres
$M\ell/yr.$	Mega litres per year
MWD	Monthly water demand
n	Number of discount periods
NPV	Net present value
$PAMWC$	Property's average municipal water cost
$PERC_{perv}$	Percentage of impervious runoff routed to pervious
$p_{m/d}$	Pan evaporation for month or day (mm)
P_t	Incidental rainfall during time t
PV	Present value
R	Monthly rainfall in mm/month
R^2	Coefficient of determination
RCi	Roof depression storage capacity (mm)
RE_t	Evaporation during time step (mm)
r_m	Effective monthly rainfall (mm)
R_t	Rainfall depth in time t
s	Surface area of vegetation type or pool
S_p	Symonds pan evaporation (mm)
S_t	The overflow / spillage during time t
SV	Storage volume
t	Time (seconds, minutes or hours)
TWD	Total water demand
VR	Volumetric reliability

V_{Rain}	Volume of rainfall in a specific time period
V_{Ru}	Volume of runoff in a specific time period
V_t	The storage volume at the end of the current time step t
WSE	Water saving efficiency
YAS	Yield after spillage
YBS	Yield Before spillage
yr.	Year
Y_t	The yield / water demand during time t
ΔRVR	Required change in Volumetric Reliability
$\Delta VRSV$	Change in volumetric reliability as the storage volume increases
2013ZAR/kℓ	South African Rands per kilolitre at 2013 values

1. Introduction

1.1 Background

South Africa (RSA) is a water stressed, developing country facing a range of challenges with respect to water management, *inter alia,* resource shortages, environmental degradation, fragmented institutional structures and basic services backlogs (Kok & Collinson, 2006; Turton, 2008; DEA, 2010; UNEP, 2010; RSA, 2011a, 2011b; Fisher-Jeffes *et al.*, 2012; DWA, 2013). Currently, more than 60% of South Africa's population lives in urban centres (RSA, 2011a). Urbanisation, which is expected to continue in South Africa (RSA, 2011a), further adds to the difficulty in addressing the challenges as it '*affects many resources and components of the environment in urban areas and beyond* (Marsalek *et al.*, 2006)'[1]. Urbanisation results in the natural water cycle being altered. The differences between the natural and urban water cycles may be broadly summarised as: an increase in surface imperviousness, changes in runoff conveyance networks and an increase in water demand (Vörösmarty & Sahagian, 2000; Hoban & Wong, 2006; Marsalek *et al.*, 2006). The result has been, and continues to be, an increasing demand for all resources. Water is just one, albeit an important one, of the resources affected.

In 1992, in response to the realisation that the scarcity and misuse of fresh water posed a serious and growing threat to sustainable development and the environment (UN, 1992b), the delegates at the International Conference on Water and the Environment adopted the 'Dublin Principles' (discussed further in Appendix B) which recognise that increasing water scarcity is the result of the different and conflicting uses and overuses of water. These realisations were once again brought to international attention in the 2006 Human Development Report (UNDP, 2006), which warned that the world was approaching a '*global water crisis*'. The RSA, based on the total actual renewable water resources (TARWR) per person per year, is estimated to be the 29th driest country out of 193 countries (Muller *et al.*, 2009). The first National Water Resources Strategy (DWAF, 2004) indicated that, by 2050, the RSA will have exceeded the limits of its economically usable, land-based water resources. Addams *et al.* (2009), however, predict that, by 2030, South Africa will already be facing significant water resource shortages, with an average supply shortfall of 17%. The RSA has already developed and utilised most of the economically available yield from surface water resources (DWA, 2013). The potential economic consequences of water shortages in the RSA have been highlighted in many publications (Ashton, 2000; Scholes, 2001; Turton, 2008; Addams *et al.*, 2009; Muller *et al.*, 2009) and recognised in national strategy and policy documents (DWAF, 2004; RSA, 2011b; DWA, 2013). The RSA's latest National Water Resource Strategy 2 (NWRS2) (DWA, 2013) – a strategy document that guides water management in the RSA – has identified desalination as a means of supplying '*unlimited*' water. It also notes that large-scale desalination is 'imminent', while simultaneously noting that it will be expensive at the coast and too costly to use inland. The NWRS2 does recognise the need to reduce non-revenue water (NRW) water losses through water conservation and demand management (WC/WDM) approaches. The NWRS2 further recognises rainwater harvesting (at the household level) as a potential

[1] An overview of the South African context is provided in Appendix A

contributor, but suggests its use is appropriate '*for domestic purposes where communities do not have a reliable source of potable water*'. It is worth questioning the logic that rainwater only be used in communities with unreliable water supplies. The RSA is a water-scarce country where all water should be conserved and attitudes changed so the use of water may be maximised. However, there exists a historically entrenched paradigm supporting the centralised provision of water in urban areas coupled with the view that domestic RWH is not viable, a view that has been reinforced by research findings that have focused on rainwater in urban areas as a means of supplementing outdoor demand only (e.g. Jacobs *et al.*, 2011).

Internationally, it is becoming increasingly accepted that a new approach to urban water management is needed (Jacobsen *et al.*, 2012). The World Water Analysis Partnership (WWAP, 2012) notes that '*water management in urban areas can benefit from more comprehensive urban planning and integrated urban water management*'. Integrated Urban Water Management (IUWM) is an approach to urban water services that considers water supply, drainage and sanitation as components of an integrated physical system known as the urban water cycle (Mitchell, 2006). The concept of IUWM has been developed further, and many different approaches to urban water management have evolved. These approaches include, *inter alia*, Water Sensitive Urban Design (WSUD) with the goal of developing water sensitive cities (WSC) in Australia (Aus) and green infrastructure (GI) in the United States of America (USA). In the RSA, the concept of water-sensitive settlements (WSS) (broadly based on the Australian WSC approach) has been proposed (Armitage *et al.*, 2014). WSC and WSS are both developed around three principles proposed by Wong & Brown (2008): '*access to a diversity of water sources underpinned by a diversity of centralised and decentralised infrastructure; provision of ecosystem services for the built and natural environment; and socio-political capital for sustainability and water sensitive decision making and behaviours*'. An in-depth discussion of the alternative approaches to, and paradigm shifts within, urban water management is included in Appendix B.

Rainwater and stormwater harvesting for residential use is potentially one means of ensuring that a diversity of water resources is used within an urban area. Technologically, it is relatively easy to harvest rainwater and stormwater – it has been done for centuries in different countries across the world using a variety of different collection storage and distribution methods (Pacey & Cullis, 1986; Gould & Nissen-Petersen, 1999; Pandey *et al.*, 2003; Hamdan, 2009; Mwenge Kahinda, 2010).

1.2 The research need

The existing situation with respect to water scarcity in the RSA is aggravated in that freshwater resources are unevenly distributed and disproportionally available relative to demand (UNDP *et al.*, 2000; Blignaut & Heerden, 2009; UNEP, 2010; Carden, 2013). Rainwater and stormwater are underutilised water resources that could potentially be used as sources of non-potable water to supplement the water supply in urban areas. They are increasingly being used around the world as alternative water resources in urban areas. Site-scale rainwater harvesting (RWH) has been widely promoted as a water demand and conservation strategy in the RSA, but with a specific focus on rural and poor communities. International experience has indicated that site-scale rainwater harvesting is an expensive alternative, but that stormwater harvesting on a broader scale is more economical (e.g. Marsden Jacobs Associates, 2006). For rainwater and stormwater harvesting to be a viable resource in the South African context, it is necessary to: understand the potential financial implications (for cities and individuals); investigate the reliability of rainwater and stormwater systems in the RSA; understand the potential risks; ensure that, where schemes are implemented, they are designed sustainably (socially, economically and environmentally); and ensure that stormwater harvesting (SWH) schemes are designed in a manner that accounts for local factors. There has been limited research and understanding of whether RWH is viable in the RSA. For example, whilst Mwenge Kahinda *et al.* (2010) considered the viability of RWH around the country, their focus was not on residential urban areas, and the methods employed have been shown in other studies (See Neumann *et al.*, 2011) to lead to significant errors in estimates of yield. Meanwhile, there appears to have been no major studies of the viability of urban rainwater and stormwater harvesting for residential use in the RSA.

The NWRS2 is considering the use of desalination as one way of augmenting water resources, but this will likely result in the continuation of the current silo management of the urban water cycle in the RSA. Recent developments, such as the potential sale of desalination works in Australia (See Walton, 2014), indicate that there is a need to carefully consider other alternatives before selecting desalination as the solution to the gap between supply and demand. There have been extensive investigations around the RSA considering where and how best to implement different water resources including, *inter alia*, new dams, raising dam walls, exploiting aquifers and desalination facilities. However, none has seriously considered whether rainwater and stormwater harvesting would be alternative sources of water to address the issue of water scarcity.

The RSA is a water-scarce country, yet there is currently no local evidence to suggest that urban rainwater and stormwater harvesting is, or is not, a viable urban water resource. The Water Research Commission (WRC, 2012), amongst others, has identified stormwater as being a potential water resource worth investigating. This research, therefore, aims to be at the forefront of examining the practicality and viability of SWH in the RSA.

Due to the limited availability of appropriately detailed data in the RSA, it was decided to focus on a single urbanised catchment. The Liesbeek River Catchment in the City of Cape Town was selected for this study as it incorporates a diversity of land uses and there is a larger than normal amount of data available for the effective development of the detailed models required for the accurate simulation of catchment-wide rainwater and stormwater harvesting. It is notable that, while the catchment represents a range of wealth levels, it does not contain any informal settlements / slums. This makes the analysis easier as rainwater harvesting is hard to model in informal settlements. The results from this study are – to the best of the author’s knowledge – the first detailed analysis of the potential viability of urban rainwater and stormwater harvesting for residential use in the RSA. The viability of rainwater and stormwater harvesting is expected to vary in other areas of the RSA, however, the methods developed and employed in this study could be used in future studies to assess the viability of rainwater and stormwater harvesting elsewhere in the RSA.

1.3 Objectives of this research

This research aims to investigate whether, and under what conditions, rainwater and/or stormwater may be considered a viable alternative water resource for residential use in the Liesbeek River Catchment, Cape Town, RSA. The hypothesis is thus: '*Stormwater harvesting is a viable water resource that offers the potential to improve water security in the residential areas of the Liesbeek River Catchment, Cape Town.*'

1.4 Key research questions

The viability of rain- and stormwater as a resource is dependent on practical (quantity and quality), economic, environmental, social and political factors. This study focuses primarily on the practical, economic and environmental aspects of rain- / stormwater harvesting. The research seeks to test and analyse whether stormwater is a viable resource in residential areas of the Liesbeek River Catchment, Cape Town, RSA. The results can then be used to motivate social studies, while informing the political sphere. The following key research questions, have therefore been addressed in this study:

- Should rainwater harvesting and stormwater harvesting be promoted for residential use in the Liesbeek River Catchment?
- Under what conditions is rainwater and/or stormwater harvesting and reuse economically viable in the Liesbeek River Catchment?
- What is the potential volume of rainwater and stormwater that may be harvested for use in the Liesbeek River Catchment?
- What are the benefits (e.g. peak flow attenuation), and are they quantifiable in the Liesbeek River Catchment?
- How reliable (quantity and quality) and for what purposes is stormwater use '*fit for purpose*' as an alternative water supply?

The answers to these key questions could then be used to infer the potential viability of RWH and SWH in the RSA.

1.5 Thesis structure

This thesis consists of eight chapters, a reference list, and 17 appendices. A brief overview of Chapters 1 through 6 is provided below.

Chapter 1 provides a brief overview of the South African context, motivation for this study, the aim and objectives of this research.

Chapter 2 comprises a review of the literature relating to: rainwater and stormwater harvesting, the modelling of rainwater and stormwater harvesting and the economic modelling of rainwater and stormwater harvesting.

Chapter 3 provides an introduction to the case study site, the Liesbeek River Catchment. It provides a brief history of the catchment, an overview of the climate, the current water demand and an overview of socio-economic trends / factors within the catchment.

Chapter 4 begins by discussing the scope and limitations of this research. It then details the research method, including a description of the data requirements for, and the development of, the models used to analyse the viability of rainwater and stormwater harvesting in the residential areas of the Liesbeek River Catchment, Cape Town, RSA.

Chapter 5 summarises the results of the analysis of the viability of RWH and SWH in the Liesbeek River catchment. It further discusses the potential impacts of different modelling methods, including the potential errors.

Chapter 6 presents a concise summary of the key findings of this research, notes how this research has contributed to knowledge, and provides a list of recommendations for further research, both in the RSA and internationally.

The **Appendices** provide supporting documentation for the main thesis, including, *inter alia*, a review of literature relating to integrated urban water management, a review of the South African water management context, additional background information on the Liesbeek River Catchment, key inputs and additional results.

2. Literature review

This chapter provides a discussion of the relevant literature including, *inter alia*, aspects of engineering, economics, environmental and social sciences. It is focused on rainwater and stormwater harvesting. A brief overview of the 'South African context' and a detailed literature review of urban water management, the need for a paradigm shift in how urban water is managed, and what alternatives to conventional urban water management are being proposed and implemented have been included for readers not familiar with the topics or terminology in Appendix A and Appendix B respectively.

The literature review begins with a discussion of the difference between the terms 'rainwater harvesting' and 'stormwater harvesting', and defines how they are used in this study. Section 2.2 highlights the challenges and opportunities associated with rainwater and stormwater harvesting, while Section 2.3 provides a brief history of the use of RWH and SWH in the RSA and elsewhere in the world. It then focuses on a detailed discussion of rainwater harvesting (Section 2.4) and stormwater harvesting (Section 2.5), including how rainwater and stormwater harvesting may be modelled in an urban context. Since there are a number of similarities between rainwater and stormwater harvesting – as will be fleshed out in the literature review – certain aspects of modelling these systems are the same. In order to prevent repetition, these aspects are discussed in Section 2.6. The Literature Review concludes with a summary in Section 2.7.

2.1 Rainwater vs stormwater harvesting

The terms 'rainwater harvesting' (RWH) and 'stormwater harvesting' (SWH) are used in the literature to refer to the collection, storage and use of runoff. Within the urban context, SWH is generally defined as the collection, storage and use of runoff from urban surfaces (e.g. roads, drains) that would otherwise drain to a water body (DECNSW, 2006; O'Connor *et al.*, 2007; NRMMC *et al.*, 2009a; Akram *et al.*, 2014). On the other hand, RWH is typically considered to be the collection, storage and use of runoff from roofs (Thomas, 1998; Hassell, 2005; Roebuck, 2007). There is, however, a level of confusion in the literature, with some authors using the term 'RWH' to refer to the collection of any runoff and specifying different sub-categories of RWH, e.g. Domestic RWH, as referring to the collection of runoff from residential roofs (Mwenge Kahinda *et al.*, 2008; Hamdan, 2009; Helmreich & Horn, 2009). Other authors suggest RWH is the direct capture of stormwater, typically from roofs (Armitage *et al.*, 2013). This seems mostly to be as a result of whether the original focus of the literature was on rural or urban environments.

The RSA's NWRS2 (DWA, 2013) suggests RWH is a technology more appropriate in rural areas and implies a broader definition than just the collection of runoff from a roof. Furthermore, DECNSW (2006 p.13) notes that, within an urban context, '*stormwater harvesting schemes and the systematic installation of rainwater tanks across a catchment can have broadly similar benefits in reducing pollution loads, downstream stormwater flows and demand for mains water. However, there are distinct differences in costs, stakeholders, and* *maintenance and health risks between these approaches – each has potential advantages and disadvantages*'. As such, it is important to define what RWH and SWH refer to within the context of urban water management in the RSA and within this thesis specifically, as follows:

- **Rainwater Harvesting (RWH)** is the collection and storage of runoff from the roof/s present on an individual property and the subsequent use within that property – including both indoor and outdoor uses.
- **Stormwater Harvesting (SWH)** is the collection and storage of runoff from an urban area, and the subsequent redistribution for use by one or more independent users for any appropriate purpose; for example: garden irrigation and/or toilet flushing.

The above definitions also have important implications in that RWH systems are systems that are typically owned, operated and maintained by individuals. Conversely, SWH systems are typically owned, operated and maintained by a collective such as a municipality.

2.2 Opportunities and challenges for RWH and SWH

2.2.1 Drivers and potential benefits of stormwater harvesting

RWH and SWH have been shown to offer a range of benefits, including the reduction of peak flows, total runoff volumes and associated pollutants (Chiu *et al.*, 2008; Fletcher *et al.*, 2008). RWH and SWH should also reduce potable water demand – if they are being used in place of another water source. However, Roebuck (2007) points out that claims made by many authors relating to RWH are unproven. This section briefly considers some of the potential benefits and drivers for RWH and SWH.

2.2.1.1 Peak flows and runoff volume

RWH and SWH have been promoted as a means of attenuating peak flows, reducing runoff volumes and mitigating flood risk (Woods-Ballard *et al.*, 2007; Fletcher *et al.*, 2008, 2013; Huang *et al.*, 2009; Armitage *et al.*, 2013). While Burns *et al.* (2010) note that to attenuate peak flows to predevelopment levels will require more than simply RWH, they suggest that RWH may be able to reduce peak flows by between 10% and 20% (even up to the 100 year recurrence interval). However, the literature increasingly suggests that in general RWH only attenuates the peak flows of minor storm events, and not to predevelopment levels – as highlighted in the results from three recent studies:

- Burns *et al.* (2014) monitored 12 voluntarily installed RWH systems for 2 years and found that the majority of systems failed to offer stormwater retention approaching predevelopment conditions. This was attributed to a combination of limited demand and small tank capacity which meant that overflow from the tanks frequently occurred because they were not emptied often enough.

- Petrucci *et al.* (2012) found that rainwater tanks '*affect the catchment hydrology for usual rain events,* (but) *are too small and too few to prevent sewer overflows in the case of heavy rain'*.
- Campisano *et al.* (2014) concluded a study which investigated the 'Potential for Peak Flow Reduction by Rainwater Harvesting Tanks' by noting that: '[the] *results show that significant reduction of the flow peak may be obtained with the use of rainwater tanks depending on the tank size and on the household water demand patterns.*' The study further showed that at the system scale RWH has the potential to attenuate peak flows, under specific conditions.

Burns *et al.* (2014) and Petrucci *et al.* (2012) seem to contrast with the finding by Burns *et al.* (2010) and this may be as a result of: the density of the catchments on which these studies were undertaken; local climate; and the fact that, while Burns *et al.* (2010) modelled a hypothetical catchment, Petrucci *et al.* (2012) modelled an actual catchment and Burns *et al.* (2014) monitored actual RWH systems. Campisano *et al.* (2014) also indicate that once the RWH storage is full, RWH offers very little attenuation. While possibly obvious, this is in line with Burns *et al.* (2012) who showed that the larger the roof area, the less efficient the RWH system was at attenuating runoff, unless a proportionally larger tank is used. This will likely make the system uneconomical / impractical in densely developed urban areas. The fact that RWH systems offer little attenuation once the storage is full is significant in the case of larger catchments where the time of concentration may be longer than the time it takes to fill the RWH storage. In such cases, at a catchment scale, RWH will likely not offer peak flow attenuation.

The attenuation of peak flows has been noted as a benefit of stormwater harvesting (Hatt *et al.*, 2006; Fletcher *et al.*, 2008, 2013); however most of the studies of these impacts rely on modelling, with little or no monitoring data available. Modelling has shown that '*reductions of around 40 to 50% in the 3 month recurrence interval peak flow, dropping to around 5 to 10% for the 100-year recurrence interval event* (Fletcher *et al.*, 2008)' are possible. The same reductions in pollutant loads are to be expected but there is a risk of over-abstraction. Over-abstraction happens when excessive amounts of water are harvested, resulting in the environment not receiving adequate water for its healthy functioning. Therefore, in order to prevent over abstraction, it is necessary to evaluate the required environmental flows on a catchment-by-catchment basis (Fletcher *et al.*, 2008).

2.2.1.2 Water quality benefits

Urbanisation also leads to decreasing stormwater quality (Duncan, 1995; Makepeace *et al.*, 1995; AMEC *et al.*, 2001; Marsalek *et al.*, 2006; Lee *et al.*, 2010). Table 2-1 summarises the pollutants typically conveyed by stormwater and their effect on water quality. Buys & Aldous (2009) noted that stormwater runoff is a major contributor to deteriorating water quality in the urban water systems of cities in RSA. Conventional drainage systems are generally focused on managing local flooding and largely ignore the need to preserve or improve water quality (AMEC *et al.*, 2001; Woods-Ballard *et al.*, 2007; Burns *et al.*, 2010; Armitage *et al.*, 2013).

Table 2-1: Summary of urban stormwater pollutants (AMEC *et al.*, 2001)

Constituents	**Effects**
Sediment – Suspended Solids, Dissolved Solids	• Stream turbidity • Habitat changes • Recreation / aesthetic loss • Contaminant transport • Filling of lakes and reservoirs
Nutrients – Nitrate, Nitrite, Ammonia, Organic Nitrogen, Phosphate, Total Phosphorus	• Algae blooms • Eutrophication • Ammonia and nitrate toxicity • Recreation / aesthetic loss
Microbes – Total and Faecal Coliforms, Faecal Streptococci Viruses, *E.Coli*, Enterococci	• Ear / Intestinal infections • Shellfish bed closure • Recreation / aesthetic loss
Organic Matter – Vegetation, Sewage, Other Oxygen-demanding Materials	• Dissolved oxygen depletion • Odours • Fish kills
Toxic Pollutants – Heavy Metals (cadmium, copper, lead, zinc), Organics, Hydrocarbons, Pesticides / Herbicides	• Human & aquatic toxicity • Bioaccumulation in the food chain
Thermal Pollution	• Dissolved oxygen depletion • Habitat changes
Trash and debris	• Recreation / aesthetic loss

In an attempt to reduce the impact on receiving water bodies, many approaches have been developed, including WSUD – further discussed in Appendix B. Where RWH/SWH is used in conjunction with treatment (whether conventional or alternative), the use of harvested rainwater / stormwater will reduce, potentially towards predevelopment levels, pollutant loads that would otherwise be discharged to receiving water bodies (Mitchell *et al.*, 2005; Wong *et al.*, 2012).

2.2.1.3 Water demand

RWH and SWH should reduce potable water demand if they are being used in place of another water source, and this is without exception found to be true (e.g. Roebuck, 2007; Maheepala *et al.*, 2011; Neumann *et al.*, 2011). The degree to which potable water demand is reduced will depend on what the harvested water is used for.

A significant benefit of reducing potable water demand is the potential to delay the development of new water resources – e.g. new dams and desalination works. For example, Coombes *et al.* (2002b) showed that the use of RWH to meet outdoor, hot water and toilet flushing demand could potentially delay the construction of new water supply head works infrastructure by up to 34 years in New South Wales, Australia. This could have significant economic implications for the local water authority.

2.2.1.4 Economic considerations

As demand for potable water exceeds available supply, it becomes necessary to look at options for meeting or managing the demand. Figure 2-1 shows the direct costs of different water supply options in Australia. The cost of stormwater harvesting is relatively low in comparison to other supply options, yet the cost of rainwater harvesting – a widely accepted approach (Hatt *et al.*, 2006) – is relatively expensive. Figure 2-1 also shows significant variability in the cost per kilolitre of harvested rainwater, which is an important consideration. The variability is the result of a range of local and climatic factors. This variability, and the associated uncertainty with respect to the cost, can be a barrier to households adopting RWH, and undermines the 'driver' of reduced potable water demand. It is therefore important to provide upfront information on the local costs of installing and operating such a system (Leonard *et al.*, 2014).

The fourth Dublin Principle is: '*water has an economic value in all its competing uses and should be recognised as an economic good'* (UN, 1992b). Therefore, direct and indirect costs and benefits should be considered, including the opportunity cost and the value of benefits derived from ecosystem goods and services – benefits arising from healthy ecosystems (Mwenge Kahinda *et al.*, 2008; Liang & Van Dijk, 2010). SWH has a range of benefits that are difficult to quantify including, *inter alia*: reduced stormwater runoff and reduced pollutant loads entering receiving water bodies (Philp *et al.*, 2008; Scholes & Shutes, 2007). Costanza *et al.* (1997) further highlight the importance of considering the value of ecosystem goods and services. In their study, they estimated the value of ecosystem goods and services for the entire Earth to be approximately US$33 trillion per year, when at the same time the global gross national product was estimated to be around US$18 trillion per year. '*Efficient stormwater management will demonstrate many benefits such as groundwater replenishment, energy conservation, carbon sinking, air quality, efficient land use planning and robust urban development'* (Dharmaratna & Gangadharan, 2011, p. 3).

Philp *et al.* (2008) note that investigating the economics of SWH is difficult due to '*issues in defining the true costs and benefits of stormwater harvesting in comparison to traditional urban water supplies'*. For example, there is significant variation in the cost of RWH/SWH. In some cases the cost of RWH and SWH has been found to be more than traditional water sources (Goonrey, 2005; Marsden Jacobs Associates, 2006; Philp *et al.*, 2008). However, there are additional benefits – as discussed in previous sections – that should be considered in an economic analysis (Philp *et al.*, 2008). The ability to undertake such an analysis is limited due to a lack of data on the life-cycle costs of SWH systems, limited quantitative data on the benefits associated with SWH, and no accepted approach to undertaking economic analyses of

SWH systems (Hatt *et al.*, 2004a; Philp *et al.*, 2008; Goonrey *et al.*, 2009; Akram *et al.*, 2014). A discussion of economic analysis methods is provided in Section 2.6.6.

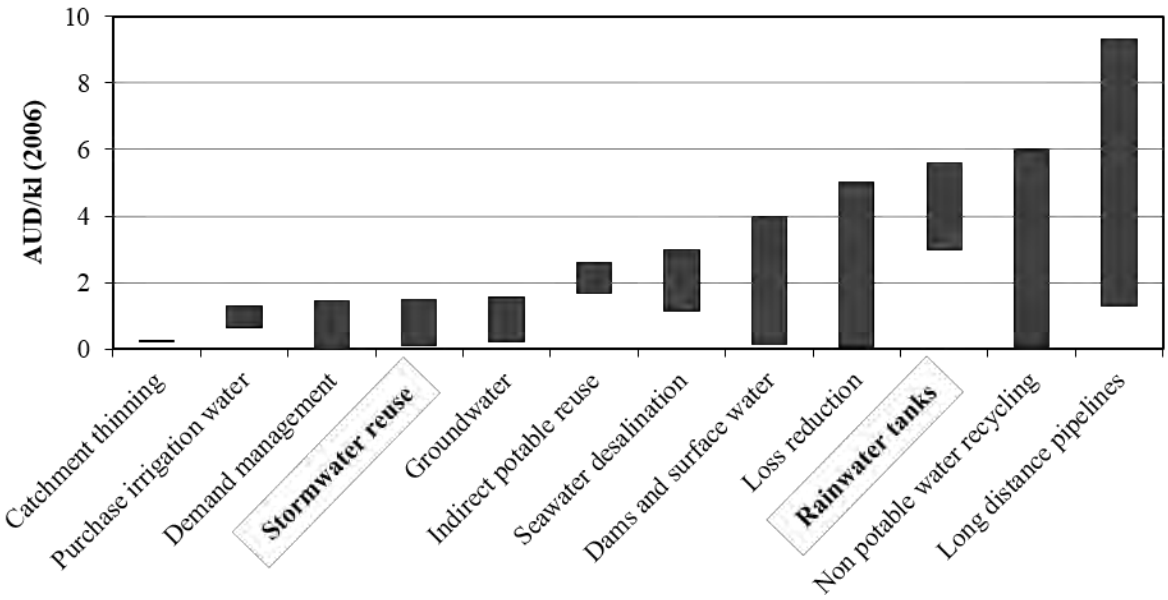

Figure 2-1: Direct costs of water supply / demand options in Sydney, Adelaide, Perth and Newcastle (Marsden Jacobs Associates, 2006)

2.2.2 Barriers to the uptake of rainwater and stormwater harvesting

'*Water represents many values to society and it contributes to a complex system of services'* (Zenani & Mistri, 2006). Social, cultural and religious views of RWH and SWH have the potential to derail any scheme – if no-one will use the water, the scheme will fail. Even if potential users understand the need for a reuse scheme, this does not mean that they will be willing to make use of it (Ilemobade *et al.*, 2009). For example, owing to perceived risks and the 'disgust' factor (revulsion or deep-seated negative response to an idea or something), the use of alternative water sources to supplement potable water supplies has not always been possible in Australia (Alexander, 2010). There is, however, a general preference for stormwater over wastewater reuse due to the different perceptions regarding its 'history' (Coombes & Mitchell, 2006; Fletcher *et al.*, 2008; Marks *et al.*, 2008; Mankad & Tapsuwan, 2011). Rainwater and stormwater may be used in a range of applications, as shown in Table 2-2, but how it is used is context-specific and likely dependant on the public's perception of the acceptability of rainwater and stormwater as water resources.

Table 2-2: Potential uses of harvested rainwater and stormwater
(Scholes & Shutes, 2007)

Residential uses	Industrial uses	Agricultural uses
Garden irrigation	Toilet flushing	Grazing land irrigation
Toilet flushing	Cooling towers	Golf course irrigation
Hot water	Cleaning processes	Crop irrigation
Car washing	Electricity generation	Parks irrigation

2.2.2.1 Public perception and acceptability

In South Africa there has been limited research into the public perceptions of RWH and no known research into the public perceptions of SWH. There have, however, been studies of the general perceptions surrounding water and wastewater recycling. Wilson & Pfaff (2008) found, *inter alia*, that: there were no apparent religious grounds preventing the possible reuse of wastewater; religious views surrounding responsibility and sustainability could support wastewater recycling; potable recycling could be acceptable politically; schemes should be implemented in as equitable and just manner as possible; costs must be fairly distributed; and environmental concerns are an important consideration. On the other hand Ilemobade *et al.* (2009) showed that there is a sense of '*it's a good idea, but someone else should do it*'. For example, they found 94% of respondents supported recycling during a drought, yet 64% were not willing to use recycled water. The same response might be found for RWH and SWH. It is, therefore, important that the social, cultural, political and religious factors form an integral part of the assessment of whether a stormwater harvesting scheme is viable.

In the first study focussing on social perceptions of RWH in the RSA, Dobrowksy *et al.* (2014) found that all respondents who had been supplied with RWH systems wanted to keep them. The two main reasons were the financial savings (reduced municipal water demand) and, when there were disruptions in the municipal supply, the RWH system continued to supply water. It is worth noting that the Dobrowksy *et al.* (2014) study was conducted in a low-income housing area, the respondents had not paid the capital costs of the RWH system, the systems were gravity fed (i.e. not pumped) and only 66% of the people indicated they would fix the RWH system if it broke. Dobrowksy *et al.* (2014) also found that harvested rainwater was used for multiple domestic uses, as shown in Table 2-3. The findings of Dobrowksy *et al.* (2014) are encouraging and indicate that, if RWH is economical, people may invest in it.

Research such as Coombes & Mitchell (2006), Dobbie *et al.* (2012) and Wu *et al.* (2012), has indicated that public support for using stormwater for a particular end use is closely related to how personal the end use is or how much human contact it has. The more personal (e.g. bathing), the lower the acceptability of stormwater – no matter the level of treatment. Stormwater is also generally perceived to be more acceptable than recycled wastewater (Coombes & Mitchell, 2006; Dobbie *et al.*, 2012), although it is important to recognise that perceptions change over time as a result of experiences and exposure to information (Coombes & Mitchell, 2006).

Table 2-3: Uses of harvested rainwater (Dobrowksy *et al.*, 2014)

Use	%
Laundry	92
Cleaning	70
Gardening	46
Bathing	44
Drinking	24
Cooking	19

2.2.2.2 Education

In Australia, public perception has largely been negative towards the use of recycled water in households. In comparison, Singapore embarked on a very strong public engagement programme, including having approximately 60% of the population visit a water recycling plant, which has led to the widespread acceptance of water reuse (Scholes & Shutes, 2007). This ties in with findings by Leonard *et al.* (2014) that a lack of community consultation is a barrier to the adoption of WSUD, including RWH and SWH.

A recent study in Australia which considered the perceptions of risk related to stormwater harvesting indicated that urban water management professionals in Australia generally '*associated moderate benefits with them* [SWH and stormwater treatment systems]*, generally agreed that the technologies were proven in contributing to sustainable urban water management and were moderately confident that the systems could deliver the intended water service.'* (Dobbie *et al.*, 2012). The research did however note differences in support for specific uses of treated stormwater and which stormwater systems were appropriate in different developments, and that the practioners' background influenced their assessment of the perceived risks (Dobbie *et al.*, 2012).

Education and knowledge about RWH is important, as most urban water management professionals in Australia trust individual homeowners to manage the risk associated with operating and maintaining their system (Dobbie *et al.*, 2012). In the RSA it is unlikely that urban water management professionals would be as trusting.

Another complicating factor is the historical acceptance or rejection of systems. For example, in some countries, rainwater harvesting used to be illegal and/or discouraged (Mankad & Tapsuwan, 2011). In the RSA, this was also the case; however, since the adoption of the current National Water Act (NWA) in 1998, there has been legislative authority supporting rainwater harvesting (see Section 2.2.4). Nevertheless there could be a residual rejection based on historical experience.

2.2.2.3 Maintenance and operation

Water Sensitive Urban Design (WSUD) is an approach to urban water management (discussed in Appendix B) that seeks to ensure that urban development and redevelopment addresses the sustainability of water – but the associated systems (which include RWH and SWH) are prone to failure as a result of poor maintenance practices. As a result, developers, residents and councils may become reluctant to invest in them (Leonard *et al.*, 2014). Leonard *et al.* (2014) noted that uncertainty about maintenance and operation and the associated costs was often a barrier to the acceptance of alternative systems, including RWH and SWH, and that it was necessary to develop and implement long-term maintenance plans.

Another potential reason for the failure of these systems, especially SWH systems, is that current practice is ahead of research. In essence this means that urban water management professionals are experimenting and hoping for the best. This does not necessarily mean that individuals have accepted the risks associated with SWH – they may simply be unaware of them. In some cases the system may work, in other cases the system may fail. The reason for the failure is often as a result of a failure to maintain the system properly. Hatt *et al.* (2004b) warn that *'Just one high profile case of public health or environmental failure of a re-use project ... could undermine public confidence in re-use nationally, costing our society time and money in the much needed adoption of future water re-use technologies'*.

2.2.3 Risk management

'Risk', which may be defined as a measure of the probability and the severity of adverse effects (Lowrance, 1976), is crucial to determining the feasibility of stormwater harvesting and reuse schemes (Goonrey, 2005; DECNSW, 2006). The viability of stormwater as a resource is dependent on the associated risks being actively managed (Hatt *et al.*, 2006), and failure to do so could create a barrier to its use. The main categories of risk that need to be managed are public safety, environmental health and public health (DECNSW, 2006; Kruger, 2007; NRMMC *et al.*, 2008). NRMMC *et al.* (2008) developed Australia's guidelines for stormwater harvesting and reuse based on a risk-management framework that focuses on public and environmental health, as summarised in the following sections.

2.2.3.1 Public safety

A key purpose of stormwater management is to '*protect the health, welfare and safety of the public, and to protect property from flood hazards'* (CSIR, 2005b). Stormwater harvesting schemes need to manage risks through implementing mitigating measures. For example, open water bodies pose a potential risk for drowning that may be managed through limiting access or considering the embankment slopes. Another important public safety risk associated with stormwater harvesting schemes is the potential flooding of urban areas as a result of overtopping and failing embankments (DECNSW, 2006; NRMMC *et al.*, 2008).

2.2.3.2 Environmental risk

Coombes & Mitchell (2006) note that there might be competing objectives when making use of alternative water sources such as rainwater or stormwater. It is important that harvesting stormwater does not negatively affect the progress towards the vision it aims to help achieve. There are many potential environmental risks, such as the negative impacts on soil quality, localised flooding, over abstraction of water, and the negative impact on groundwater quality (as a result of recharging poor quality water) (DECNSW, 2006; NRMMC *et al.*, 2008). In the RSA, these risks are predominantly managed through ensuring that the standards set in the South African Water Quality Guidelines (DWAF, 1995, 1996a, 1996d) are met.

2.2.3.3 Public health

As noted in Section 2.2.2, the perceptions surrounding stormwater quality and the willingness to use harvested stormwater is largely related to perceptions around the quality and health risks of using stormwater. It is therefore crucial that public health risks are well managed, as this may encourage or discourage the general willingness to use the harvested water.

While stormwater is widely considered to be polluted and will typically require treatment prior to use (Coleman, 2001; Fletcher *et al.*, 2008; Aryal *et al.*, 2010), studies of harvested rainwater quality have produced contradictory conclusions (DeBusk & Hunt, 2014). For example, while Abdulla & Al-Shareef (2009) suggested that RWH is a safe source of drinking water in Jordan, the only major study to consider the water quality of RWH in the RSA to date found that harvested water typically met national and international standards for chemical parameters but not microbiological parameters (Dobrowksy *et al.*, 2014) – suggesting that it is necessary to treat harvested rainwater prior to potable use. Abbasi & Abbasi (2011) provide a comprehensive review of the results from many studies that investigated the quality of harvested rainwater. It is apparent that the quality of harvested rainwater is a result of numerous factors and is location specific (Abbasi & Abbasi, 2011; DeBusk & Hunt, 2014). Abbasi & Abbasi (2011) suggested that, even where RWH water quality is expected to be of potable standard, in order to manage risk, it is advisable to disinfect the water prior to its use for potable purposes.

Until recently, risks within the water sector have been managed using end-use standards – water quality standards based on what the water is used for. The South African Water Quality Guidelines (DWAF, 1996b, 1996c, 1996d) reflect this approach. This is beginning to change with a move towards the use of risk assessment as a more effective approach (Howard *et al.*, 2006). This has been endorsed by the World Health Organisation (WHO) (Howard *et al.*, 2006; WHO, 2008). Howard *et al.* (2006) note that quantitative microbial risk assessment (QMRA) – an approach that makes use of the dose-response relationship of a pathogen to assess the risk of exposure – is considered by '*the WHO as a valuable tool for setting health-based targets and for validation of water safety plans'*.

NRMMC *et al.* (2006, 2008, 2009) have adopted the QMRA approach used by the WHO for use in Australia when assessing the risk of using recycled wastewater and stormwater for a range of purposes. The problem with the use of the QMRA approach in the RSA is that the pathogens (e.g. *Cryptosporidium, Campylobacter, Rotavirus*) on which the QMRA is based are not commonly tested for in stormwater in the RSA. On the other hand, *E. Coli* is regularly tested for, and as Howard *et al.* (2006) showed, could potentially be used as a proxy. The use of *E. Coli* as a proxy in the RSA does, however, come with many problems that would need to be considered (Barnes, 2012). There is a need for further research into how best to assess the risks to public health associated with RWH and SWH.

2.2.4 Policy, regulation and guidance: rainwater and stormwater harvesting

The National Water Act (NWA) (RSA, 1998) recognises that water is a scarce and unevenly distributed resource in RSA. The Constitution (Schedule 4 – Part B) (RSA, 1996) determines that the provision of potable water, sanitation and stormwater services in urban areas is the responsibility of the local municipality. As is common across the world, municipalities across the RSA frequently separate the management of stormwater from that of water and sanitation, with the former often being assigned to roads departments. This has resulted in the silo management of urban water services (Fisher-Jeffes *et al.*, 2012). The National Water Services Act (NWSA) places a duty on municipalities to develop water services development plans (WSDP) detailing '*existing water services*'. While this should encourage integrated management, unfortunately it has not. Instead, because stormwater is frequently considered together with the provision of roads, it is often neglected in municipalities' WSDPs and only mentioned as a result of problems relating to the ingress of stormwater into the sewage system – which overloads the wastewater treatment works and, in some cases, is a cause of water pollution (see CoCT, 2011b).

The NWA (Schedule 1 (1a)) (RSA, 1998) states that a person may '*store and use run-off water from a roof*'. Significantly, the act also states in Schedule 1 (1f) that a person may discharge '*run-off water, including stormwater from any residential, recreational, commercial or industrial site, into a canal, sea outfall or other conduit controlled by another person authorised to undertake the purification, treatment or disposal of waste or water containing waste'*. These two points are significant in that the first explicitly allows rainwater harvesting at the household scale and the second implies stormwater should at some stage be treated, whether by the individual or municipality (following discharge into the system) – although this is not happening. If a municipality is to treat polluted stormwater, it would need to recoup the cost, which could possibly, in part or in full, be accomplished through harvesting stormwater; and then charge users for harvested stormwater which could be used, *inter alia*, for irrigation. Currently the provision of stormwater management in the RSA is generally funded from property rates, which means stormwater departments have to compete with other departments with many more pressing needs when advocating for funding. No municipality in South Africa currently charges directly for the provision of stormwater services – aside from a possible once-off connection fee (Fisher-Jeffes & Armitage, 2013).

The City of Cape Town's Stormwater Management By-law (CoCT, 2005) and the associated Management of Urban Stormwater Impacts Policy (CSRM, 2009b) are, arguably, the RSA's most advanced stormwater legislation. In themselves, they are an important first step in that they encourage an alternative approach to stormwater management that might include RWH and SWH. However, there is a need for the development of legislation at all levels of government that encourages the management of the whole urban water cycle in an integrated manner – and not just components of it.

2.3 History of use of RWH/SWH in the RSA and elsewhere

Pacey & Cullis (1986), Gould & Nissen-Petersen (1999), Pandey *et al.* (2003), Hamdan (2009) and Mwenge Kahinda (2010) have provided a comprehensive history of rainwater harvesting. They highlight that rainwater harvesting (RWH) has been practised for thousands of years around the world. Pandey *et al.* (2003) document the use of RWH dating back over the past 8,000 years. There are examples of RWH from every continent (except Antarctica), across a range of climates and using a range of techniques (Table 2-4). Gould & Nissen-Petersen (1999) also highlight that rainwater harvesting remains a primary source of water for many isolated households on islands in the Pacific. Mwenge Kahinda (2010) notes that, as a result of increased pressure on freshwater resources, there has been an interest in alternative water sources, and consequently, there is a '*revival of this old practice*'.

The history of urban SWH is not as well documented, and it is difficult to separate historical RWH and SWH practices in the literature. Gould & Nissen-Petersen (1999) highlight that communal rainwater harvesting (similar to modern-day stormwater harvesting) has been practised for centuries. Goonrey (2005) suggests that the Commonwealth Environment Protection Agency (1993) and Dowsett (1994) were at the forefront of proposing that stormwater should be considered a resource rather than a problem and thus should be harvested. Hamdan (2009) cites examples from China of well storage tanks constructed between 1970 and 1974. Philp *et al.* (2008) highlight many case studies from the 1970s to the present. Lim *et al.* (2011) detail Singapore's transition to harvesting urban stormwater, which began in the 1970s and was actively pursued by the 1980s. SWH, as defined in this thesis, appears to have started in the late 1970s and early 1980s in many countries. However, as Philp *et al.* (2008) point out, the application of SWH outside of Australia has been limited. While the RSA does have examples of both RWH and SWH, these examples are isolated and do not represent case studies where stormwater harvesting forms part of the urban water system.

Table 2-4: Selection of examples of rainwater and stormwater harvesting from different geographic regions around the world

Continent	Country*	City	Stormwater / Rainwater harvesting	Source
Africa	South Africa	Cape Town	Rainwater, stormwater	DWAF (2010); CoCT (2011a)
	Namibia	Omdel	Stormwater (Floods)	Zeelie (2004)
North America	United States of America	Nationally	Rainwater and stormwater	USEPA (2012c)
	Canada	Nationally	Stormwater	Exall *et al.* (2006)
South America	Brazil	Petrolina	Rainwater	Scholes & Shutes (2007)
Middle East	Israel	Nationally	Stormwater	Tal (2006)
	Gaza	Khan Younis city	Stormwater	Hamdan (2012)
Australasia	Australia	Melbourne	Stormwater, rainwater	Anderson (2003); Wong *et al.* (2012)
		Adelaide		
Europe	Denmark	Århus	Rainwater	Stockholm Environment Institute (2009)
	United Kingdom	Nationally	Rainwater	UKRHA (2012)
	Malta	Nationally	Stormwater	Gatt & Farrugia (2012)
	Germany	Nuremburg	Rainwater	Stockholm Environment Institute (2009)
Asia	India	Nationally	Rainwater	Stockholm Environment Institute (2009)
	Singapore	Nationally	Rainwater, stormwater	Lim *et al.* (2011)
	Japan	Okinawa	Rainwater	Kawasaki *et al.* (2005)

*Only a selection have been shown, as countries may have numerous schemes.

2.3.1 Rainwater harvesting in South Africa

Rainwater harvesting (RWH) is not new in the RSA. The Department of Water and Sanitation (previously the Department of Water Affairs) has historically, and continues to, promote RWH for rural and poor communities. As a consequence, research in the RSA has largely focused either on the use of rainwater within the rural environment and for supplying food gardens (Denison & Wotshela, 2009; Helmreich & Horn, 2009; Mwenge Kahinda *et al.*, 2010; Viljoen *et al.*, 2012; DWA, 2013; Enninful, 2013; Dobrowksy *et al.*, 2014). The NWRS2 (DWA, 2013) states that rainwater '*will also be used for domestic purposes where communities do not have a reliable source of potable water*'. Dobrowksy *et al.* (2014) note that the DWA has earmarked RWH '*as a short-term intervention to provide water*' for dispersed settlements with inadequate water supply. It is worth questioning the logic behind considering that rainwater only be used in communities with unreliable water supply. The RSA is a water-scarce country where all water should be conserved in all contexts – urban or rural. If harvested rainwater is (temporarily or permanently) acceptable for some citizens, why is it not acceptable for all? If concerns are related to managing the risks associated with water quality, shouldn't the causes of poor water quality be addressed rather than limiting the technology's use?

There has been relatively little notable research into the impacts, whether positive or negative, of urban domestic RWH in the RSA, although interest in RWH has been increasing recently. Jacobs *et al.* (2011) showed that RWH for garden irrigation in the Western Cape is not viable due to the climate (winter rainfall), but never considered alternative uses such as toilet flushing. Viljoen (2013) showed that RWH in commercial buildings could lead to substantial financial savings; however, the results failed to consider life-cycle costs and significantly overestimated the yield due to the use of a monthly time step for modelling the changes in storage (see Section 2.6.3). SASOL, a major petroleum producer, has commissioned reports to consider the viability of investing in rainwater harvesting on their sites or investing in RWH for others as a means of possibly achieving water off-setting credits (Socio-Technical Interfacing, 2013). These studies have focused on commercial / industrial facilities and have shown that RWH is a potentially viable option in these situations.

As a result of initiatives such as the Green Building Rating system, many commercial buildings have installed RWH systems to obtain the rating they desire; for example, the Aurecon head office in Cape Town (Aurecon, 2011). Furthermore, there are many private companies that promote RWH systems and claim to show that they are financially viable at the domestic scale. However, no research has tested these claims in the RSA. Similar claims were made in the UK, but Roebuck (2007) showed these claims to be largely untrue.

Currently in the RSA, there is limited uptake of RWH in urban areas, especially at a domestic scale. While individual companies might incorporate RWH into new buildings, there is no requirement to do so. The potential benefits of RWH within an urban setting are not being considered, investigated or realised. Whether RWH proves to be financially viable at the site scale, there are other benefits (e.g. flood attenuation) that could be realised at the catchment scale. A particularly relevant benefit for municipalities may be that, through reducing demand for potable water, it may be possible to delay the need for new infrastructure. These aspects need to be properly examined.

2.3.2 Stormwater harvesting in South Africa

Wright (1996) appears to be the first to moot the possibility of the widespread use of stormwater as a resource in the RSA. The Atlantis Water Resource Management Scheme (AWRMS) is an example of SWH in the RSA dating back to 1979 (DWAF, 2010). The establishment of AWRMS was initially in response to the need to find an alternative to marine wastewater discharge (DWAF, 2010). After more than 30 years in operation, the AWRMS – shown schematically in Figure 2-2 – is seen internationally as an exemplar of a wastewater and stormwater reuse scheme, making use of managed aquifer recharge (where water is temporarily stored in the unconfined aquifer that underlies the area) (Philp *et al.*, 2008). An important aspect of the harvesting system's design entailed the designing of the catchment – the town of Atlantis. Atlantis was thus planned with fully separated residential and industrial areas. This has allowed for the separation of high- and low-quality wastewater effluent and stormwater. The low-quality water is disposed through recharge near the coast in such a way as to create a hydraulic barrier between the cleaner groundwater and the seawater (Murray & Tredoux, 2004).

The AWRMS is an example of large scale stormwater harvesting in the RSA. While SWH schemes, such as AWRMS, could provide a valuable resource in the RSA, to date, the use of SWH (of any quality of water) has not been widely used around the country (Tredoux *et al.*, 2002; Murray *et al.*, 2007b). The AWRMS was the only example of a large scale, operational, stormwater recharge facility identified in a major study of the potential for MAR in the RSA (see Murray *et al.*, 2007a). It is interesting that this large scale scheme started off as an interim solution while a 'conventional' pipeline was developed (DWAF, 2010). While the AWRMS has ensured the town of Atlantis has had a sustainable supply of water for over 30 years (Murray & Tredoux, 2004), it has not led to the widespread uptake in SWH in the RSA.

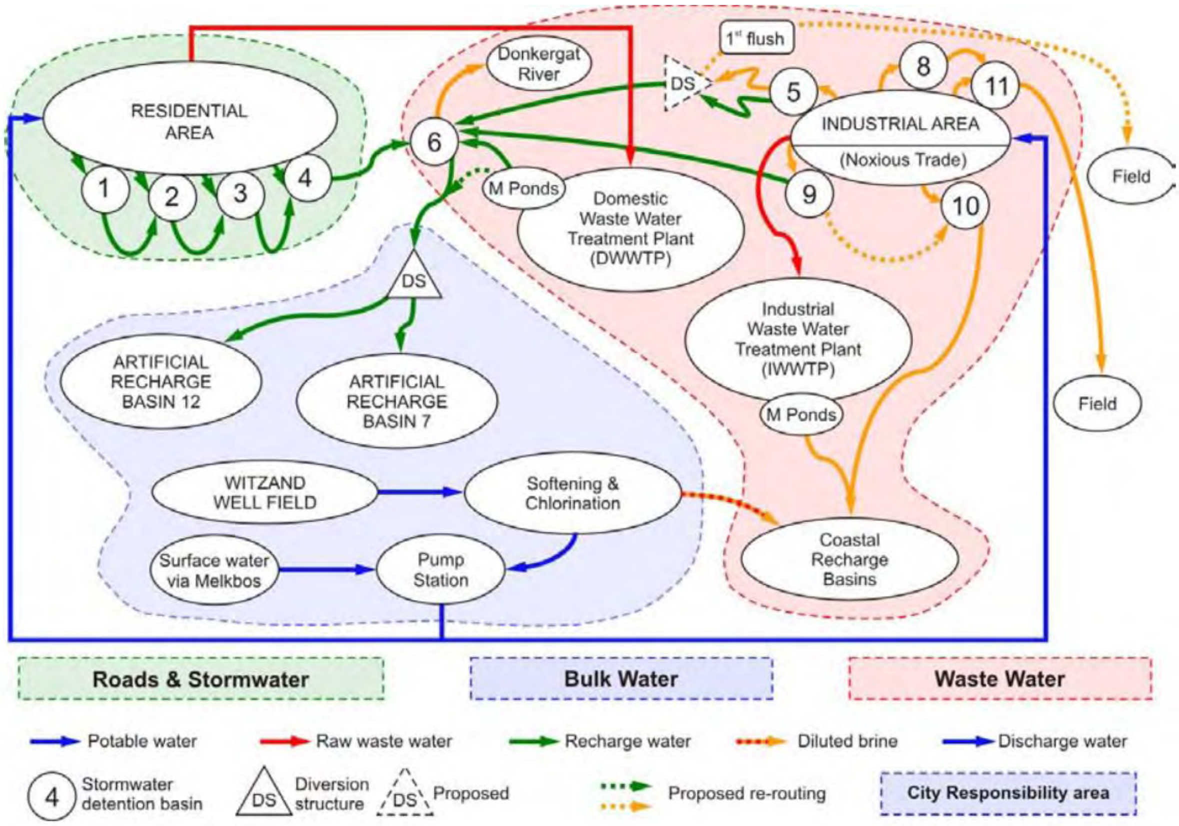

Figure 2-2: Schematic of the Atlantis Water Resource Management Scheme
(DWAF, 2010)

2.4 Rainwater harvesting systems

Rainwater Harvesting (RWH) is the collection and storage of runoff from the roof/s present on an individual property and the subsequent use within that property for domestic purposes – including indoor and outdoor uses. As highlighted in Section 2.2, RWH has been used for centuries as a form of water supply. It has also been the focus of significant research in a number of countries. Fewkes (2006) and Roebuck (2007) provide similar extensive reviews of rainwater harvesting systems and different approaches to modelling such systems at the site / individual system scale. DeBusk & Hunt (2014) provide a review of the literature relating to RWH with a focus on the outcomes of studies, rather than the design and modelling of RWH systems. Together these reviews provide a comprehensive 'state of the art'. This section, thus, aims to highlight the aspects most relevant to the research and refers back to the above sources for further details.

2.4.1 Types of rainwater harvesting systems

Legget *et al.* (2001), Roebuck (2007), Woods-Ballard *et al.* (2007) and Armitage *et al.* (2013) define three types of RWH systems: gravity-fed systems, directly pumped systems and indirectly pumped systems.

A gravity-fed system is any RWH system where the runoff is collected and distributed via gravity. Roebuck (2007) described a typical gravity-fed system in the United Kingdom as: a system where the '*main storage tank is located within the roof void of the building. Rainwater is collected from the roof, filtered and then piped directly to the storage (header) tank. Water is delivered to appliances via gravity'*. In the RSA and much of the developed world, a gravity system would refer to a system where the tank is on the ground and water is abstracted from the tank in basins / buckets for indoor use, and outdoor use may be distributed through a pipe network by gravity (Figure 2-3). These systems are commonly supplied to low-income communities (see Pacey & Cullis, 1986; RainWater Cambodia, 2011; Dobrowksy *et al.*, 2014). Fewkes (2006) notes that the main advantage of gravity systems is that they do not require a pump and an electrical supply. However, the disadvantage is that the pressure may be too low for some appliances such as washing machines and dishwashers (Roebuck, 2007). In lower-income communities in the RSA where these systems have been implemented, this may not be a problem, as many don't have such appliances. Another disadvantage of gravity-fed systems where the tank is stored in the roof structure, is that the roof structures must be capable of carrying the load. This can limit the size of storage systems (Fewkes, 2006).

Directly pumped systems (Figure 2-4) use pumps to supply harvested rainwater at pressure at the point of use. Rainwater is stored in a tank (above or below ground) and then pumped directly to the point of use when required. The advantage of such a system is that water is provided at an adequate pressure for required use; e.g. garden hoses and washing machines (Fewkes, 1999; Roebuck, 2007; Woods-Ballard *et al.*, 2007). However, should the pump fail (including, for example, as a result of an electrical outage) no water can be supplied directly to the point of use.

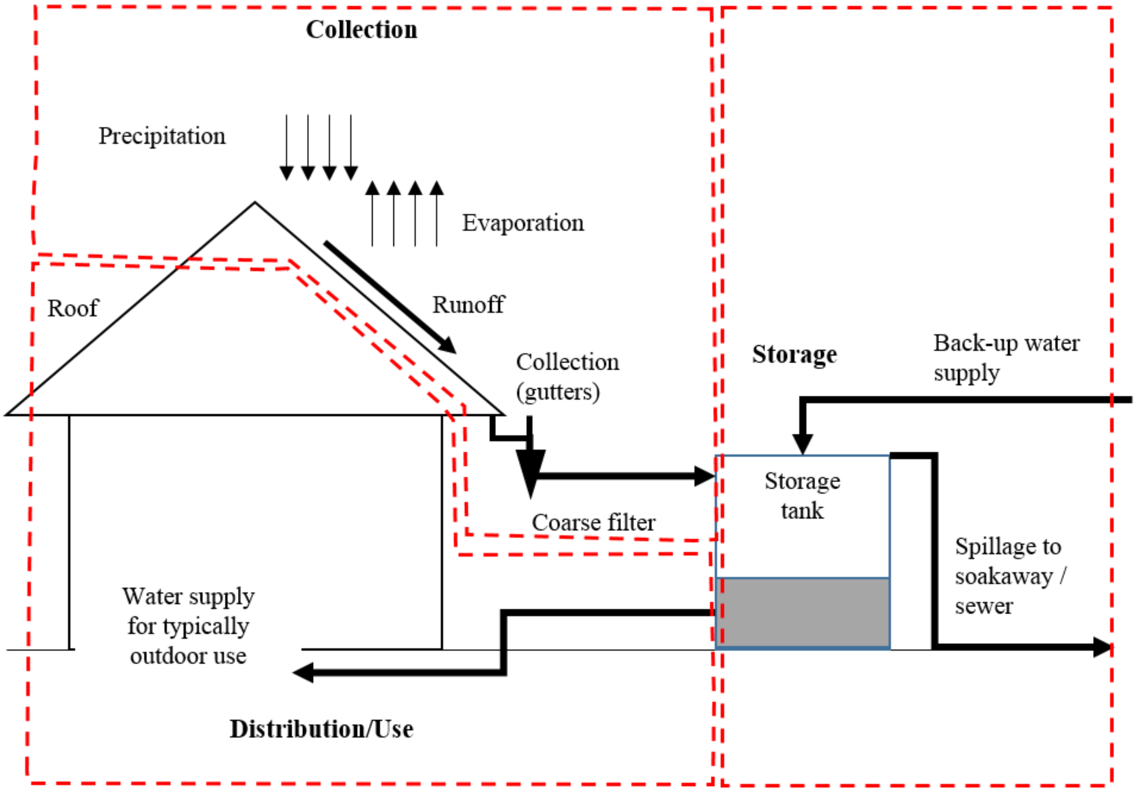

Figure 2-3: Gravity-fed RWH system
(After Legget *et al.*, 2001; Roebuck, 2007; Water Rhapsody, 2014)

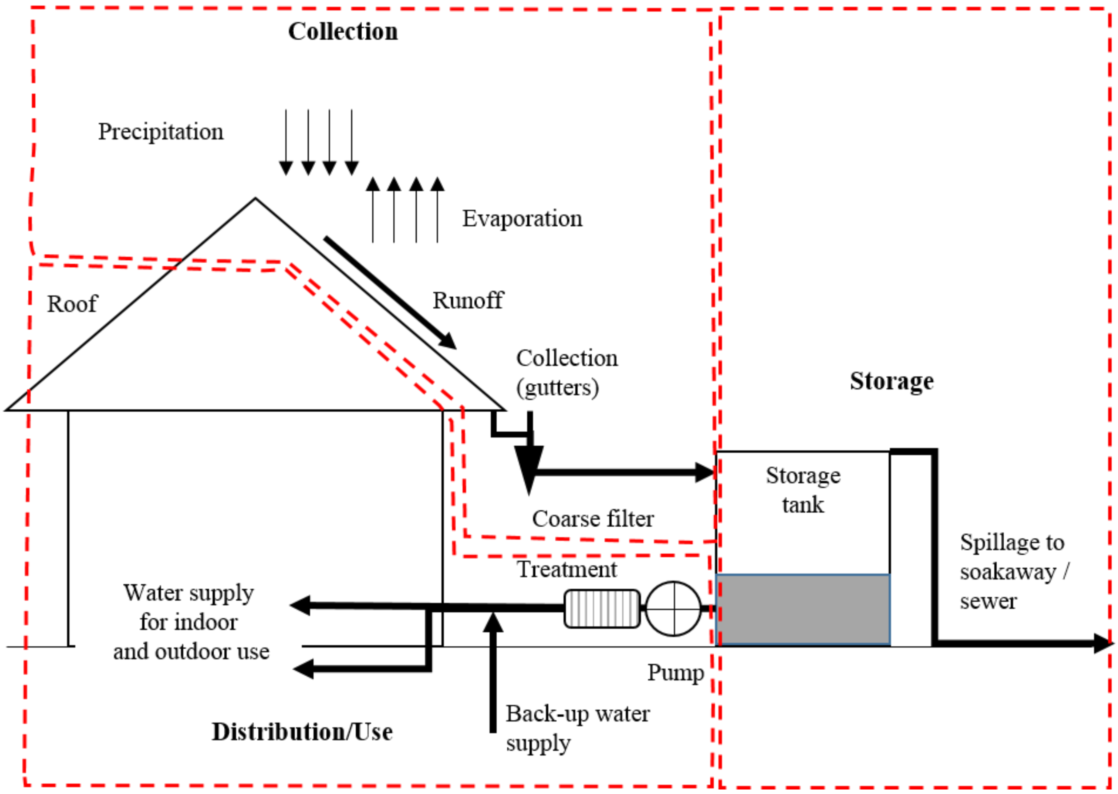

Figure 2-4: Directly pumped RWH system
(After Legget *et al.*, 2001; Roebuck, 2007; Water Rhapsody, 2014)

Indirectly pumped systems (Figure 2-5) are hybrids of directly pumped and gravity systems. Rainwater is collected in a storage tank (at or below ground level) and then pumped to a header tank – located within the roof as with gravity systems. The header tank then supplies water by gravity. When the water in the header tank drops to a predetermined level, the pump turns on, and it is refilled (Roebuck, 2007; Woods-Ballard *et al.*, 2007). Indirectly pumped systems have the advantages of being able to store more water than gravity systems where the tank is situated in the roof, simpler pump control mechanisms and, in the event of a pump / electrical failure, water will still be supplied to appliances until the header tank is empty. The disadvantage is that the pressure may be too low for some appliances (Roebuck, 2007).

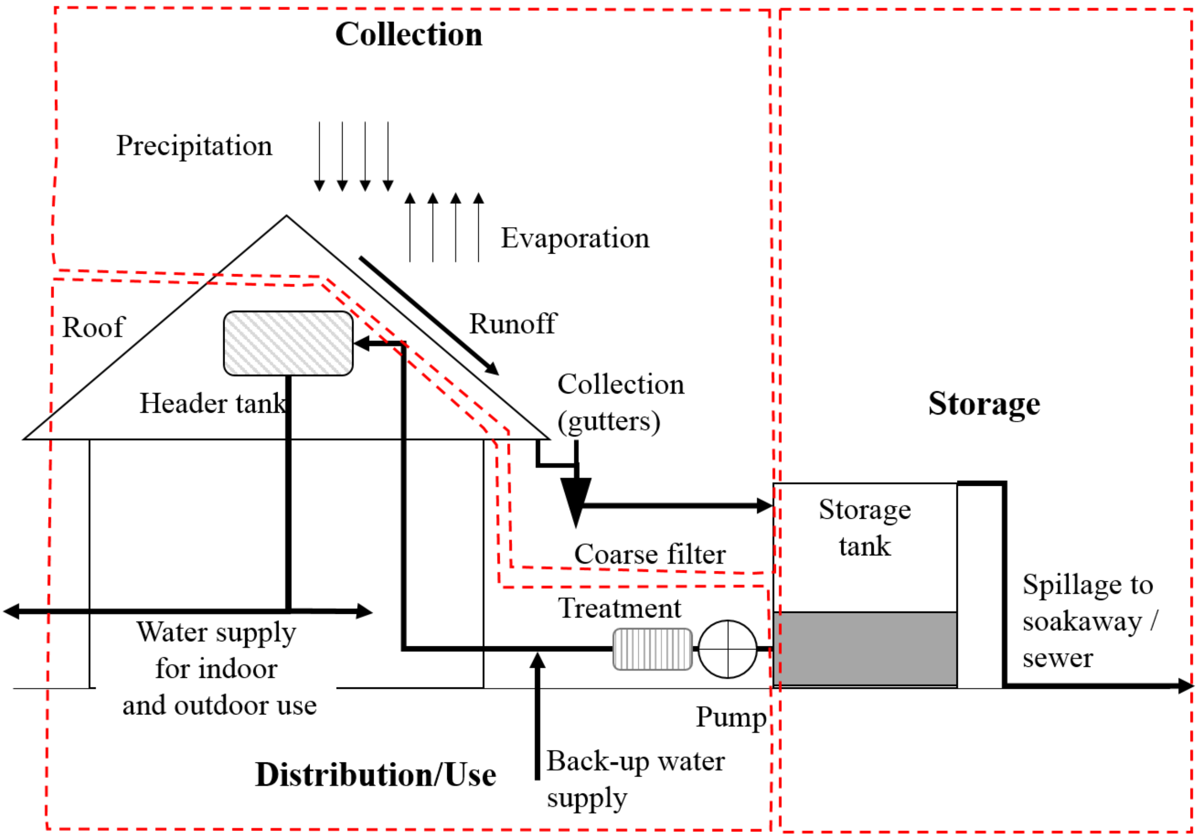

Figure 2-5: Indirectly pumped RWH system
(After Legget *et al.*, 2001; Roebuck, 2007; Water Rhapsody, 2014)

The selection of a system has significant financial and economic impacts. Directly pumped systems are usually recommended for use in domestic properties (Roebuck, 2007; Taylor, 2013). This is due to the fact that certain appliances need a required flow and/or pressure of water (Roebuck, 2007; Taylor, 2013). Gravity systems (whether in the roof or on the ground) could be appropriate if harvested rainwater is to be exclusively for low pressure end-uses; e.g. for filling swimming pools. Once water is required to be supplied at pressure, it is often necessary to use a pumped system (Taylor, 2013). Roebuck (2007) recommends that, in commercial situations, it is best to install an indirectly pumped system, as peak demands tend to be relatively high, increasing the demands on the pump. Furthermore, the indirectly pumped system can continue to supply water during pump or electrical failures.

2.4.1.1 Typical rainwater harvesting system components

RWH systems may comprise a range of components, which varies depending on the type of system. A typical RWH system comprises the following components (Figure 2-6): catchment surface (roof); collection (gutters) and pipework; first-flush diverters and filters; storage device/s; pump; treatment; control and management systems; back-up supply; and distribution pipework (Roebuck, 2007). Important aspects of the different components are discussed in the following sections.

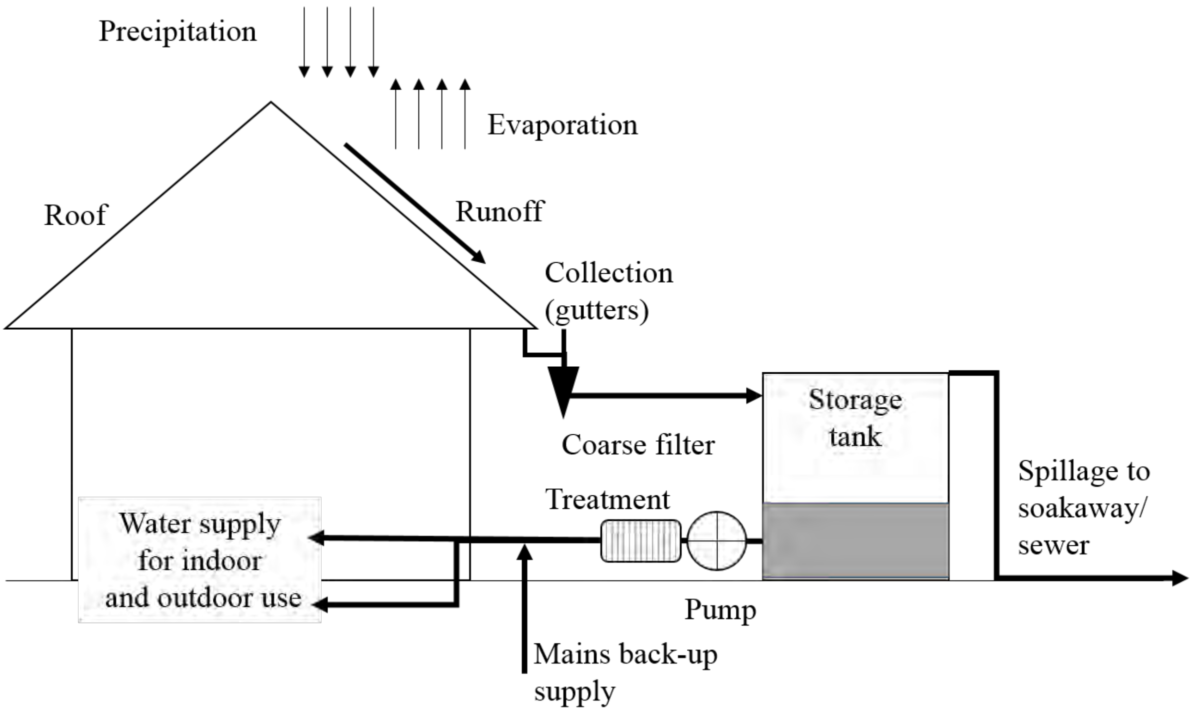

Figure 2-6: Typical rainwater harvesting system

2.4.2 Collection

Rainwater can be harvested from many different surfaces (Roebuck, 2007), but as noted in Section 2.1, RWH in urban areas is generally considered to be the collection, storage and use of runoff from roofs (Thomas, 1998; Hassell, 2005; Fewkes, 2006; Roebuck, 2007). Therefore, this section focuses on the harvesting of runoff from roofs. The harvesting of runoff from other surfaces is dealt with in Section 2.4.5.

The volume of runoff will depend on the depth of rainfall, the area of the roof (catchment area) and the volume of losses. Roebuck (2007) highlights that not all of the rain will runoff from a roof's surface. The proportion of rainwater that becomes runoff may be influenced by, *inter alia*, the roofing material, ponding, absorption and the amount of evaporation. The difference between the volume of rainfall and volume of runoff is considered to be 'runoff losses'. Two main approaches are used to account for the runoff losses: first, the use of a runoff coefficient and, second, by defining a minimum depth of rainfall before runoff occurs – referred to as 'initial losses' (Mitchell, 2007; Roebuck, 2007) or 'depression storage' (Mitchell *et al.*, 2008a).

2.4.2.1 Roof area

To model RWH systems, it is necessary to obtain estimates of the effective roof areas. Many recent studies (see Liaw & Tsai, 2005; Ghisi *et al.*, 2006; Mitchell, 2007; Mitchell *et al.*, 2008a; Maheepala *et al.*, 2013) have made use of roof areas derived from an analysis of existing properties. Mwenge Kahinda *et al.* (2008) and Jacobs *et al.* (2011), two RSA studies on rainwater harvesting, used roof areas of 20 m^2, 40 m^2 and 200 m^2 respectively. These estimates were used to represent typical low-income houses and a large house in a low-density suburban area. However, it appears that there are no major studies in the RSA that suggest typical roof areas for different regions, suburbs or income levels.

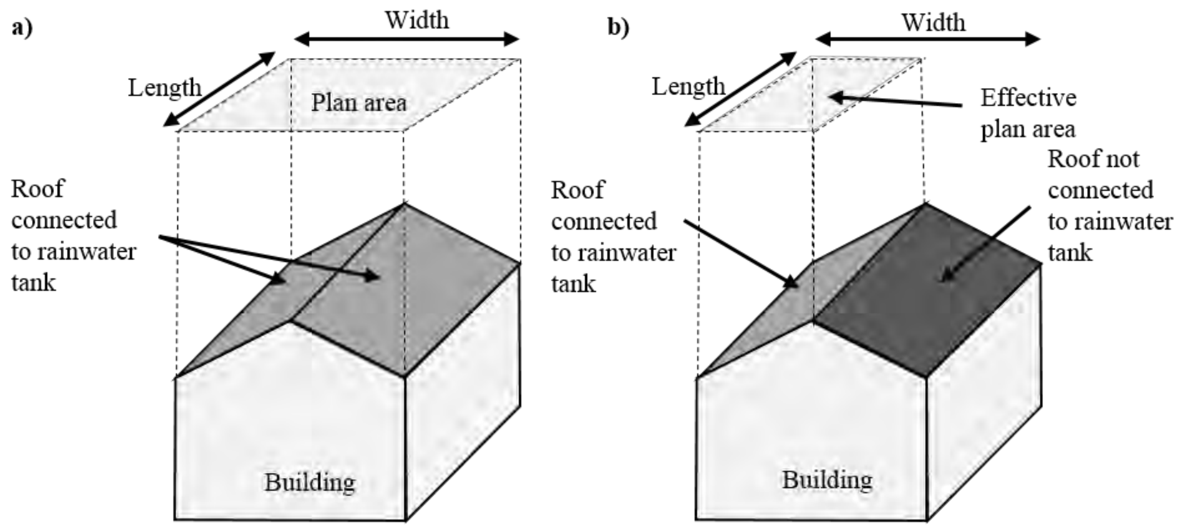

Figure 2-7: a) Total roof area (maximum potential catchment area), b) Effective roof area (actual RWH catchment area) (After Roebuck, 2007)

The roof area is generally calculated as a plan area, not as a total surface area. It is also important to differentiate between the total roof area and the effective area – that which is connected to the RWH storage unit (Figure 2-7), since the volume of captured runoff is directly related to the catchment (roof) size. In South East Queensland (SEQ), Australia, it is required that rainwater tanks be connected to at least the lesser of, 50% of the total roof area or 100 m^2

of the roof area (DLGP, 2008; Biermann *et al.*, 2012). Biermann *et al.* (2012) showed that, in SEQ, on average 118 m^2 of the roof area was connected to the RWH system, exceeding the 100 m^2 requirement. This, however, represented an average of 39% of the total roof area. Table 2-5 highlights that the roof areas in SEQ are relatively large compared to those investigated by Jacobs *et al.* (2011), who considered 200m^2 a 'relatively' large roof area in the RSA. When considering the potential for RWH to manage urban flood risk, Burns *et al.* (2010) assumed 100% of a roof area to be connected and the maximum total roof area used in the study to be 230m^2. Burns *et al.* (2010) used data from Mitchell *et al.* (2005) that was based on studies focused on Melbourne, Australia. Had Burns *et al.* (2010) used data from the Biermann *et al.* (2012), the study's results would no doubt have been different – probably significantly so – since roughly only 39%, and not 100% of the roof area would be connected to the RWH system. This highlights the need for local data for key modelling inputs such as the roof area. Furthermore, if local factors, such as the roof area have an impact on the performance of a RWH it is necessary to question whether it is appropriate to apply the results – such as the potential to mitigate floods – of studies from other areas.

Table 2-5: Total vs. connected roof area (Biermann *et al.*, 2012)

Local Government Area (LGA)	Total roof area (m^2)	Connected roof area (m^2)	Percentage connected (%)
Caboolture	310	119	38
Gold Coast	326	136	42
Pine Rivers	281	110	39
Redland	294	113	38
Average for all LGAs	300	118	39

2.4.2.2 Runoff coefficients

The runoff coefficient is defined as the proportion of rainfall that becomes runoff (Fewkes, 2006; Roebuck, 2007; Butler & Davies, 2010). Runoff coefficients have been estimated by collecting data over a period of months or years, representing many storm events. The average difference between rainfall and runoff is then reported as the runoff coefficient (Roebuck, 2007). Table 2-6 shows a range of typical runoff coefficients for a range of different types of roofs. It is interesting to note the variation between the different sources, especially for flat roofs.

The volume of runoff is generally calculated using Equation 2-1, where V_{Ru} is the volume of runoff in a specific time period; C_R is the runoff coefficient; and V_{Rain} is the volume of rainfall in a specific time period.

$$V_{Ru} = C_R \times V_{Rain} \qquad \textbf{2-1}$$

Table 2-6: Runoff coefficients for different types of roofs (After Roebuck, 2007)

Reference	Surface Type	High	Average	Low
(Pacey & Cullis, 1986)	Tile	0.9		0.8
	Corrugated metal sheet	0.9		0.7
(Dharmabalan, 1989)	Roof tiles	0.9		0.8
	Corrugated sheets	0.9		0.7
	Plastic sheets	0.8		0.7
	Thatched roof	0.6		0.5
(BS EN 752-4, 1998)	Steeply sloping roofs	1		0.9
	Large flat roofs (>10,000m2)		0.5	
	Small flat roofs (<100m2)		1.0	
(Fewkes & Wam, 2000)	Pitched roof covered with tiles or slates (Total Flow type)	1.0		0.9
	Pitched roof covered with tiles or slates (Diverter Flow type)	0.95		0.75
	Flat roof with impervious membrane	0.5		0.0
	Flat green roof with vegetation	0.5		0.0
(Leggett *et al.*, 2001)	Pitched roof tiles	0.9		0.75
	Flat roof, smooth surface		0.5	
	Flat roof with gravel layer or thin turf (<150mm)	0.4		0.4
(Woods-Ballard *et al.*, 2007)	Pitched roof tiles		0.8	
	Flat roof		0.5	
	Flat roof, gravel		0.4	
	Extensive green roof		0.3	
	Intensive green roof		0.2	

2.4.2.3 Initial losses

Fewkes (1999) showed that using only a runoff coefficient to account for runoff losses yields acceptable results, but that the results can be improved by first adjusting for initial losses. Initial losses are those that result from depression storage, absorption and wind effects, and are usually modelled by defining a minimum depth of rainfall before runoff will occur in the model (Roebuck, 2007). Modelling the initial losses improves the accuracy of a simulation model. Table 2-7 shows a range of typical initial losses for different types of roofs.

The effective rainfall is then calculated using Equation 2-2, where RE_t is the effective rainfall in time t, R_t rainfall depth in time t, and IL_t depth of initial losses in time t.

$$RE_t = Max\begin{cases} R_t - IL_t \\ 0 \end{cases} \qquad \textbf{2-2}$$

Table 2-7: Initial losses for different types of roofs (After Roebuck, 2007)

Reference	Surface type	Initial losses (mm)
Pratt & Parkar (1987)	Bungalow roofs, combination of pitched and flat roofs	0.32
Fewkes (1999)	Pitched roofs, concrete tiles	0.25
Li *et al.* (2004)	Asphalt-fibre glass	0.1
	Plastic film	0.2
	Gravel-covered plastic film	0.9
Mitchell (2007)	Typical urban roof (assumed)	1.0
Mitchell *et al.* (2008a)	Spatially averaged (normally distributed)	0 – 1.0

There does not seem to have been much interest in investigating the range of initial losses since Roebuck (2007) was published. There has, however, been research that indicated that the depth of initial losses used in modelling needs to be cognisant of the time step. Mitchell *et al.* (2008a) showed that, as the depth of initial losses increases, the difference in modelled volumetric reliability would increase between the model using a six minute time step and a model using a daily time step (Figure 2-8). In this case, the use of 1 mm as depression storage with the six minute time step model was equivalent to a 1.7 mm depression storage when using a daily time step model. The additional initial storage depth required for modelling using the daily time step model is in order to account for losses caused by wetting and drying of the roof between storm events within a time step (e.g. a day) (Mitchell *et al.*, 2008a).

2.4.2.4 First-flush diverters and filters

It has been widely shown that the first flush (initial) runoff from roofs is typically more polluted than subsequent runoff (Fewkes, 2006; Roebuck, 2007; Mendez *et al.*, 2011; Gikas & Tsihrintzis, 2012). To harvest good quality rainwater, it is suggested that systems include a first-flush diverter that captures or diverts the first 1-3 mm of rainfall that tends to collect the majority of atmospheric particulates, debris and bird / animal droppings that have built up on the roof during the preceding dry period (Martinson & Thomas, 2005; WHO, 2008; Abdulla & Al-Shareef, 2009; DeBusk & Hunt, 2014). Recent research (e.g. Mendez *et al.*, 2011; Gikas & Tsihrintzis, 2012) has, however, shown that while a first-flush diverter improves the quality in the storage unit, the rainwater will still require treatment before it is suitable for indoor use; the first flush filter improves the physio-chemical quality, but cannot prevent microbial contamination of the stored water (Gikas & Tsihrintzis, 2012).

Roebuck (2007) notes that '*limited evidence was found for the use of first flush diverters in the UK and none of the proprietary system suppliers provide them as a standard part of their package systems*'. This is equally true in the RSA, although the inclusion of a first-flush filter is advised by some suppliers (e.g. Jojo Tanks, 2013b). Instead, systems in the RSA (proprietary or otherwise) generally have a cross-flow / coarse filter that contains a mesh screen to remove larger debris. Most systems are designed to be self-cleaning (Roebuck, 2007). Particles smaller than the mesh will still pass through into the storage unit. Therefore, the microbial and small / dissolved contaminants in the first flush will still reach the storage unit.

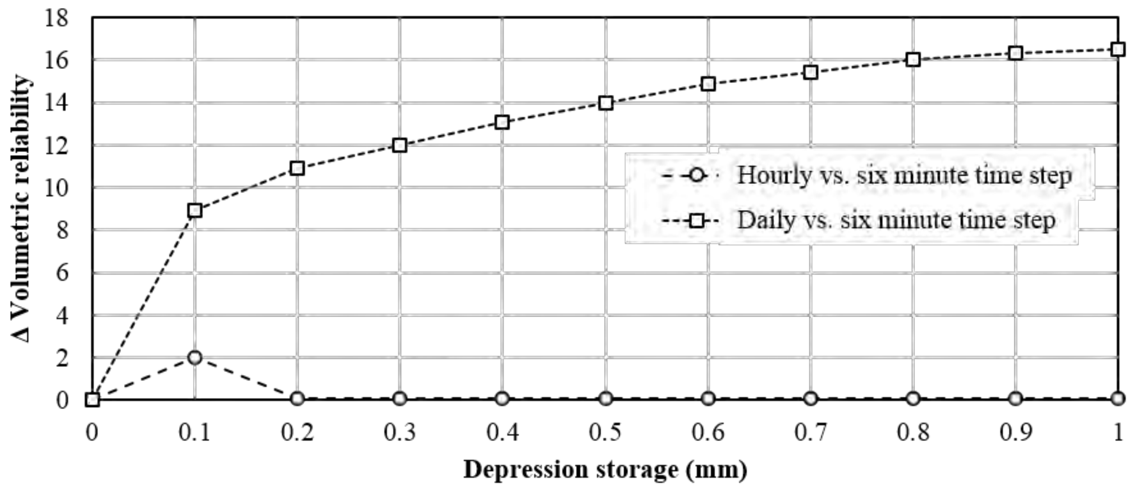

Figure 2-8: Relationship between initial losses and volumetric reliability depending on time step (from Mitchell *et al.*, 2008a)

The reduction in harvestable runoff as a result of the use of a first-flush filter is typically calculated using Equation 2-3. Essentially, a predefined volume is removed. Cross-flow filters might result in splashing / spillage as a result of their design, but these processes continue throughout the storm event, and so the harvestable runoff is typically calculated using a runoff coefficient, as in Equation 2-4.

$$FF_t = Max\begin{cases} ER_t - FF_{vol} \\ 0 \end{cases} \quad \textbf{2-3}$$

$$FO_t = FC_R \times FI_{Rain} \quad \textbf{2-4}$$

Where: FF_t is the volume of water bypassing the first-flush filter in time t; ER_t is the effective runoff from the catchment (after initial and runoff losses) in time t; FF_{vol} is the volume of storage available in the first-flush filter; FO_t is the filter outflow in time t; FC_R is the filter coefficient; and FI_{Rain} is the filter inflow in time t.

2.4.3 Storage

'*Rainfall events occur more erratically than system demand*'; therefore, the storage of rainwater is an essential part of a functioning rainwater harvesting system (Fewkes, 2006). While storage can be in the form of tanks (made from a variety of materials), permeable pavements, ponds, roofs and local aquifer recharge (Fewkes, 2006; Hatt *et al.*, 2006; Roebuck, 2007; Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013) tanks (between 1 and 10 m^3) are the most commonly used means of storage for domestic purposes (Roebuck, 2007). Other storage options are generally used at a greater scale – i.e. for harvesting stormwater – and are discussed in Section 2.4.5.

Storage tanks can be above or below ground. Hassell (2005) suggests that it is most common to make use of an underground tank, and Thomas (1998) notes that underground tanks are generally cheaper than above ground tanks. In the RSA, however, the cost of underground tanks is roughly three times that of aboveground tanks (Jojo Tanks, 2013a). Underground tanks are reported in the United Kingdom to have the advantages that '*water will be cold, so hazardous bacteria should not develop, cold water is able to store oxygen longer; and support (beneficial) aerobic development in the storage tank; algal growth will be minimized due to the lack of sunlight'* (Woods-Ballard *et al.*, 2007). In other words, storage conditions are important (Amin *et al.*, 2013). Underground tanks do have the disadvantage in that they require pumps to extract water; the integrity of the tank is difficult to monitor; it is difficult to identify and fix leaks; and they pose a greater drowning risk if their covers are not strong enough (Thomas, 1998). Above ground tanks can, however, be kept dark (colour selection) and sited in a shaded area to keep the water cool and prevent algal growth (Dashora *et al.*, 2013).

A certain level of treatment takes place within the tank (Coombes & Mitchell, 2006; Fewkes, 2006; Abbasi & Abbasi, 2011; Spinks *et al.*, 2014) through settlement, flotation and the development of biofilms. Particles such as pollen will float on the surface, and therefore, Fewkes (2006) suggests the tank should be designed to overflow at least twice a year.

The selection of the size of the storage tank is important as it impacts the capital cost of the system, the volume of water that can be stored, and the performance of the system as a whole (Fewkes, 2006). The size of the rainwater storage unit needs to be selected in relation to the catchment size and water demand (Guo & Baetz, 2007). It is also necessary that a balance between cost and performance is maintained (Roebuck, 2007). The appropriate sizing of rainwater tanks forms an important component of this study and is discussed further in Section 2.4.5 and Chapter 4.

2.4.4 Distribution

The distribution components of RWH systems comprise pumping, post-storage treatment and controls to manage back-up water supply. These are briefly discussed below.

2.4.4.1 Pumps

RWH systems, excluding gravity-fed systems, require a pump to provide water at the point of use (Roebuck, 2007). In systems where a pump is used, installation needs to be carefully considered. Biermann *et al.* (2012) showed that the total volume of a tank could effectively be reduced as a result of the installation of a pump – depending on the height of the pump outlet and the height of the pump cut off switch (if one is used) – by between 10% and 20%.

Hydraulically, pumps may be modelled simply by considering the volume of water to be pumped per unit of time (Roebuck, 2007). The modelling of a pump's electricity requirements is potentially more complicated, but is important as it is a driver of cost (Marsden Jacobs Associates, 2007). While the majority of studies that have considered the life cycle cost of a RWH system have included the cost of pumping (electricity) (Domènech & Saurí, 2011; Farreny *et al.*, 2011; Roebuck *et al.*, 2011), there are exceptions (Liaw & Tsai, 2004). Many studies considered the energy demands in a simple manner by assuming that a pump uses electricity at a constant rate per kilolitre, which is then used for estimating the electricity demand and, consequently, the cost (Ward, 2010). Legget *et al.* (2001) monitored electricity demand for pumping in RWH systems and found that it ranged between $1kWh/m^3$ and $3kWh/m^3$. In reality, energy demand may vary significantly over a period of use; i.e. when the pump starts up, as the pump re-pressurises a system, and through continuous pumping (Gardner *et al.*, 2008; Ward, 2010).

2.4.4.2 Post-storage treatment

Post-storage treatment can consist of a combination of the following to address the physical, chemical and microbiological quality of the water: in-line sediment filters / cartridge filters, slow sand filtration, in-line filters, cartridge filters, flocculation and disinfection (Abbasi & Abbasi, 2011; DeBusk & Hunt, 2014).

The disinfection of harvested rainwater is an important consideration in the management of health risks. Rainwater may be disinfected, *inter alia*, through heating, boiling, chlorination, ultra-violet (UV) irradiation, reverse osmosis and ozonation (Jordan *et al.*, 2008; Abbasi & Abbasi, 2011; DeBusk & Hunt, 2014). Each of these has its own advantages and disadvantages. The choice of disinfection method considered in this thesis is discussed further in Chapter 4.

2.4.4.3 Back-up supply

'Given the intermittent nature of rainfall it is rare that a RWH system can be designed such that a constant supply of harvested water can be guaranteed' (Roebuck, 2007). Therefore, it is important to either have a back-up supply (typically mains supply) for the rainwater tank or to be able to switch to another source of water. This is an important part of the design of a RWH, and it can be accomplished using many approaches, including a float control valve, electronic controls etc.

2.4.4.4 Distribution pipework

The distribution pipework conveys water from the storage unit to the point of use (Roebuck, 2007). In the RSA, the design, selection of materials and construction is prescribed by the following standards:

- National Building Regulations and Building Standards Act (RSA, 1997)

- South African National Standards, for example SABS 252-2 (SABS, 1993) and SANS 10252-1 (SABS, 2012), which includes the National Building Regulations, for example SANS 10400-A (SABS, 2010)

The distribution network is seldom considered in RWH studies except for the cost of installation when the economics of RWH is considered (e.g. Liaw & Tsai, 2004; Palla *et al.*, 2012). This is likely because it is a site-specific consideration (Roebuck, 2007).

2.4.5 Modelling rainwater harvesting systems

As noted previously, Fewkes (2006) and Roebuck (2007) provide in-depth reviews of rainwater harvesting systems and approaches to modelling such systems at the site / individual system scale. DeBusk & Hunt (2014) provide a state-of-the-art review of the literature relating to RWH with a focus on the outcomes of studies and highlight a range of regional scale studies. None of the studies highlighted by DeBusk & Hunt (2014) consider spatial and temporal variation in the use of RWH systems. Maheepala *et al.* (2013) provide a review of the impact that spatial and temporal factors might have at the regional scale and highlight the importance of considering these factors in order to avoid significant errors that arise from extrapolating property scale results to the catchment scale. This section aims to broadly review common approaches to modelling RWH systems at a site scale, and then at the regional scale.

2.4.5.1 Site-scale harvesting

Many approaches to modelling RWH systems have been proposed. These include: graphical methods, statistical models, probabilistic models and behavioural / continuous simulation (herein referred to as behavioural simulation) models (Liaw & Tsai, 2004; Basinger *et al.*, 2010; DeBusk & Hunt, 2014). Graphical and mass curve methods are most appropriate for rapid assessment as part of a preliminary design (Liaw & Tsai, 2004). Statistical models may be used to determine the relationship of the capacity of a reservoir and its inflow / outflow (Liaw & Tsai, 2004). Probabilistic models derive '*probability distributions for the dependent variables such as runoff and overflow from meteorological distribution functions by using hydrological relationships*' (Kim *et al.*, 2012). Behavioural simulation models make use of long-term input data (historical data or stochastically generated) to simulate, *inter alia*, the operation of a RWH system (Fewkes & Wam, 2000; Liaw & Tsai, 2004; Mitchell, 2007). They require a large amount of data and computational effort, but they are simple to develop and easy to understand as they mimic the physical system (Fewkes & Butler, 2000; Kim *et al.*, 2012). They are also considered the most accurate (Kim *et al.*, 2012). It is not surprising therefore that behavioural simulation models are common (e.g. Fewkes & Butler (2000), Mitchell (2004), Roebuck (2007), Mitchell *et al.* (2008a), Palla *et al.* (2011) etc.) and widely used by researchers (Liaw & Tsai, 2004; Mitchell, 2007).

While different models may require less (e.g. Thomas, 2002) or more data (Mitchell *et al.*, 2008a), Roebuck (2007) suggests that the minimum data required to perform a basic analysis of a RWH system is: rainfall data, catchment area and water demand. However, typically RWH models also require the following: evaporation data, roof area, initial and continuous loss factors, tank size and demand data (Thomas, 2002; Maheepala *et al.*, 2013).

2.4.5.2 Modelling roof runoff and collection

Figure 2-6 shows that RWH systems can be separated into three unit processes, namely, collection, storage and distribution. The modelling of the storage component is common among most RWH behavioural simulation models (DeBusk & Hunt, 2014). It is also common with the modelling of the storage component of SWH systems and, to prevent repetition, they are discussed together in Section 2.6.3. The modelling of the distribution system (treatment and pumping) is typically focused on the cost of treating and pumping to the point of demand and is discussed in Sections 2.4.4.1 and 2.4.4.2. The modelling of the collection of runoff is specific to RWH and is discussed in this section.

Many authors have presented models for simulating RWH at the site scale. Figure 2-9 highlights three of these models and the increasing complexity associated with each model. 'Basic models' (e.g. Ghisi *et al.*, 2007) typically only make use of a runoff coefficient (using Equation 2-1) to account for losses. They typically do not consider initial losses and filter losses separately, but assume the runoff coefficient accounts for all these losses. While Fewkes (1999) showed that this would yield acceptable results, the results can be improved by modelling the initial losses – as is done in intermediate (e.g. Mitchell *et al.* 2008a) and complex (e.g. Roebuck 2007) models.

Mitchell *et al.* (2008a) presents a model where data requirements and complexity falls between the other two models. Mitchell *et al.* (2008a) first models initial losses (using Equation 2-2), then models effective roof runoff (using Equation 2-1). The coefficient (1 - effective roof area loss factor) used for calculating the effective roof runoff accounts for '*continuous losses due to splashing, gutter overflow, etc.*' The inflow into storage is the remaining runoff after all losses have been accounted for.

Roebuck (2007) presents the most complex and data-demanding of the three models. Roebuck (2007) first models initial losses (using Equation 2-2), models effective runoff (using Equation 2-1), models losses resulting from the use of a first-flush filter (using Equation 2-3) and then models losses resulting from the use of a coarse filter (using Equation 2-3). The inflow into storage is the remaining runoff after all losses have been accounted for.

In the application of his model, Roebuck (2007) does not model the use of a first-flush filter because the limited use of first flush devices in the UK. Instead, Roebuck (2007) uses a runoff coefficient of 0.9 and a coarse filter coefficient of 0.9, which equates to reducing the runoff to 0.81 (Runoff × 0.9 × 0.9) of what it was after initial losses had been accounted for.

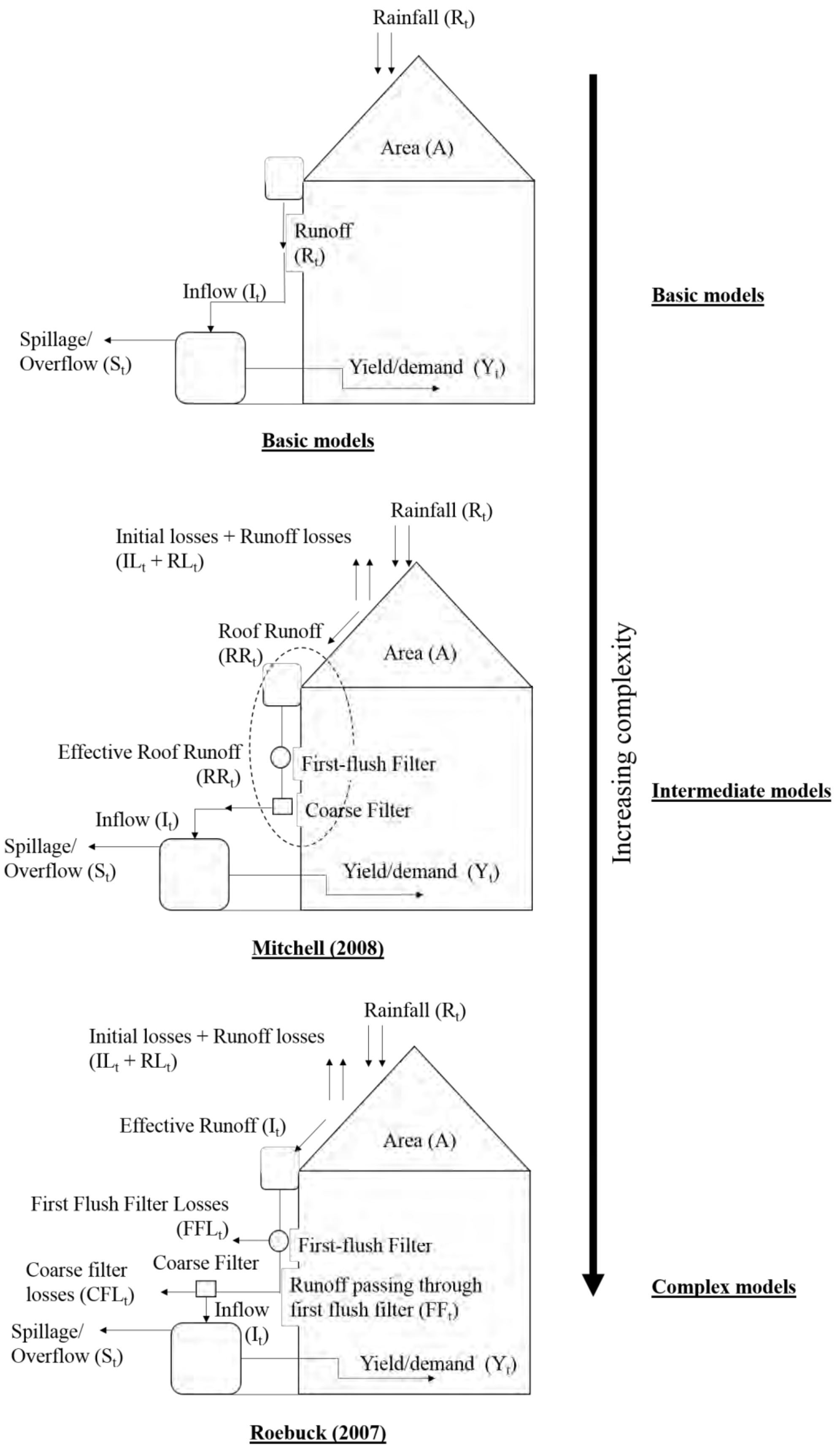

Figure 2-9: Three approaches to modelling runoff

Conversely, Mitchell *et al.* (2008a) used an 'effective roof area loss factor' (runoff coefficient) of 0.85, as '*based on the surprisingly limited amount of literature documenting roof losses total losses (the combined effect of depression storage and effective roof area loss factor) of 15-20% appears to be typical*'. While Mitchell *et al.* (2008a) do not expressly state that the 'effective roof area loss factor' accounts for the filter losses, the value used is based on total losses and would likely take this into account. However, even if that is not the case, in terms of modelling the runoff factor, a single runoff factor (selected with due consideration of the climate, roof material etc.) could be used to account for roof runoff losses and coarse filter losses – if a first-flush filter is not being modelled – in which case the models, as applied by Roebuck (2007) and Mitchell *et al.* (2008b), are essentially the same. This significantly simplifies the modelling calculation processes.

Urban drainage modelling software – for example *XP-SWMM* or *MIKE URBAN* – is in general more commonly associated with the modelling of stormwater systems, and can be used to model site-scale RWH systems (e.g. van der Sterren *et al.*, 2014). However these models are typically limited to assessing the hydraulic and water quality aspects of a RWH system and generally do not incorporate economic considerations, which models such as Roebuck (2007) do. Further, in order to analyse regional-scale RWH (thousands of systems) using such software, each property would need to be spatially modelled.

2.4.5.3 Regional scale harvesting

Modelling RWH at a site scale is relatively simple. Section 2.4.5.1 highlighted the abundance of models available for undertaking such studies. When modelling the performance of RWH systems at a regional scale, however a common approach has been to linearly extrapolate the quantity and quality implications obtained from modelling a single RWH system with average storage and water demand characteristics (Neumann *et al.*, 2011; Maheepala *et al.*, 2013). Such an approach assumes that every RWH system in the area of study provides water of equivalent quality; every RWH system has the same level of water demand, at the same rate; and factors (such as roof area) that affect supply and overflow from RWH systems are equivalent (Maheepala *et al.*, 2013). While the above assumptions are clearly not true, studies have and continue to make such broad assumptions. In a study titled '*Rainwater harvesting: a comprehensive review of literature*', DeBusk & Hunt (2014) discuss '*municipal-scale analysis*' in which many studies (summarised in Table 2-8) are listed, which all used linear extrapolation to evaluate the performance of RWH at a regional scale. DeBusk & Hunt (2014) note that site-scale studies produce higher estimates (45% to 89%) of the amount of potable water demand than could be met through RWH when compared to regional scale studies (5.6% to 68%). The difference between the results for site and catchment scale studies could be a consequence of using linear extrapolation to estimate regional impacts of RWH, and requires further investigation.

Sekar & Randhir (2007) and Mwenge Kahinda (2010) have made use of GIS to spatially assess the potential of RWH at a regional or national scale. However, both have made significant simplifications that do not account for the spatial and temporal variation in demand, such as assuming a 'standard property and house size'. Recent research, such as Steffen *et al.* (2013), used *SWMM* to investigate the stormwater management benefits of RWH. Steffen *et al.* (2013) modelled each property in the catchment in detail, including the individual RWH systems. However, Steffen *et al.* (2013), failed to account for the spatial variability in water demand, instead assuming an average demand pattern for all properties.

Table 2-8: Selection of recent RWH studies that use linear extrapolation to determine the potential impacts of RWH at a regional scale.

Reference	Method
Meng *et al.* (2005)	Used averaged input values
Ghisi *et al.* (2007)	Made use of typical roof areas, typical household size and typical per-capita demand. The results at a regional scale were then calculated by linearly extrapolating the results of the site-scale analysis.
Abdulla & Al-Shareef (2009)	'*To accomplish the objectives specified above, it was necessary to obtain rainfall data, potable water supply, population and number and area dwellings in each governorate. Then, the total roof area in each governorate was calculated based on the average area of different dwellings and their number. The potential rainwater harvesting volume is estimated based on the total roof area, the average annual rainfall, and the runoff coefficient*'.
Domènech & Saurí (2011)	The results from an average / typical property were linearly extrapolated to provide the results at a regional scale.
Kim & Furumai (2012)	'*The average area was used to conduct the analysis of RWHU for different scenarios of each building type*'.
Lange *et al.* (2012)	Undertook detailed modelling of RWH systems in one area and extrapolated the results for a significantly larger region.

Mitchell *et al.* (2008a), Xu *et al.* (2010), Coultas *et al.* (2011), Maheepala *et al.* (2011), Maheepala *et al*. (2013), Mashford *et al.* (2011) and Neumann *et al.* (2011) have all shown that the use of linear extrapolation of the benefits of RWH at an individual property scale to the regional scale can lead to significant errors in estimates of yield, overflow and quality. A summary of the results and potential error is given in Table 2-9. The degree of potential error, is significant. It is worth noting that this issue does not appear to have been reported in journal articles; all of the above references are either completed research reports or reviewed conference papers.

Coombes & Barry (2012) considered the impact of spatial and temporal averages in predicting water security using systems analysis. They warned that the '*use of average water demands that replace spatial and temporal variation in analysis of regional water systems generates dramatic reductions in certainty about system behavior that leads to incorrect understanding of the performance of the system. ... Moreover, the use of averages cannot capture the substantial spatial and temporal variation of the majority of parameters, including climate, demographics, urban form and socio-economics that drive the behavior of any urban settlement. The use of global averages in simulation of regional water systems is unlikely to* *describe the spatial and temporal contribution provided by WSUD approaches* [such as RWH] *that generate water resources or reductions in water demands within a metropolis'*.

Further support of Coombes & Barry's (2012) findings can be found in studies that have shown that the volumetric reliability of, and overflow from, a RWH system is dependent on factors such as local rainfall, catchment size and water demand (Maheepala *et al.*, 2013; DeBusk & Hunt, 2014). The significant spatial and temporal variation that can be found in each of these factors is highlighted in Sections 2.6.4, 2.4.2.1 and 2.6.5 respectively.

Table 2-9: Results of studies evaluating the impact of linear extrapolation to upscale the effects of RWH

Reference	Conclusions
Mitchell *et al.* (2008a)	This study used data from Melbourne and indicated that linear upscaling resulted in an overestimation of volumetric reliability (8%–14%) and yield (8%–24%).
Maheepala *et al.* (2011)	This study used data from Canberra and showed that linear upscaling resulted in an overestimation of volumetric reliability (15%) and yield (18%).
Coultas *et al.* (2011)	This study used data from Brisbane and showed that linear upscaling resulted in an overestimation of volumetric reliability (14.7%) and yield (14.8%). The study further indicated an underestimation of overflow volume (6.3%) and pollutant loads in the overflow (15%–27%).
Neumann *et al.* (2011)	This study used data from Melbourne and showed that linear upscaling resulted in an overestimation of volumetric reliability (16%) and underestimation of overflow by 37%. The study also indicated that pollutant loads are potentially underestimated by up to 30%.
Poustie & Deletic (2014)	Made use of *UVQ,* with averaged inputs, to model the urban water cycle.

In light of the above, the results of the studies presented in Table 2-9 seem plausible. Furthermore, the results might be explained by the following example: a household using 750 ℓ/day will empty their tank quicker than a household using 250 ℓ/day. When the average data are used to model a catchment containing many households, it effectively results in a situation where the households using 750 ℓ/day are regarded in exactly the same way for the purposes of the storage tank design as households using 250 ℓ/day. The same principle will apply to differences in roof area. The linear upscaling of the corresponding impacts of RWH of an average system is thus not recommended, and some authors suggest that analyses should rather consider a stochastic approach in order to represent the uptake of RWH (Mitchell *et al.*, 2008a; Xu *et al.*, 2010; Coultas *et al.*, 2011; Maheepala *et al.*, 2011, 2013; Mashford *et al.*, 2011; Neumann *et al.*, 2011). These authors do not address the question of how to analyse the stormwater (e.g. attenuation of peak flow) impacts of RWH within the context of an actual catchment as a whole. Their findings indicate that the potential reduction of overflow – widely reported with limited supporting evidence (Section 2.2) – could be significantly overstated (see Table 2-9). Consequently, the potential attenuation of peak flows is also likely to be less than expected due to less runoff being detained. This could potentially explain the findings by Petrucci *et al.* (2012) that rainwater tanks '*affect the catchment hydrology for usual rain events,* (but) *are too small and too few to prevent sewer overflows in the case of heavy rain'.* In the context of this research, this is a significant factor. How this was dealt with is discussed in Chapter 3.

2.5 Stormwater harvesting systems

Stormwater Harvesting (SWH) is the collection and storage of runoff from an urban area, and the subsequent redistribution for use by one or more independent users for any appropriate purpose; for example: garden irrigation and/or toilet flushing. SWH can potentially provide an alternative water resource for cities. It is thus unsurprising that interest in stormwater harvesting has significantly increased in recent years (Fletcher *et al.*, 2013). Unlike RWH, SWH is still a developing field. Hatt *et al.* (2004a), Goonrey (2005), DECNSW (2006), Fletcher *et al.* (2008), Philp *et al.* (2008) and Akram *et al.* (2014) together provide the 'state of the art' with respect to stormwater harvesting. Due to the variability in physical stormwater characteristics, the water demand patterns, costs and public perception, the design of a SWH system will vary from site to site (DECNSW, 2006; Philp *et al.*, 2008).

2.5.1 Collection

Stormwater management systems in the RSA are typically focused on eliminating local flood nuisances. Stormwater systems in the RSA can be separated into 'conventional systems' and 'alternative systems'. Conventional systems typically are thoses systems that make use of an underground pipe network to collect runoff and convey it to a nearby receiving watercourse (Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013). SWH collection systems include stormwater from streams, conventional stormwater pipe networks, and alternative stormwater management systems (e.g. SuDS) (Goonrey, 2005; Philp *et al.*, 2008).

Duncan (1995), Makepeace *et al.* (1995) and Minton (2002) have all highlighted how urbanisation impacts on the physical (e.g. dissolved solids, suspended solids, temperature, colour etc.), chemical (e.g. heavy metals, nitrate, nitrite, phosphate, etc.) and microbiological (e.g. *E.coli*, *Salmonella*, *Shigella* etc.) quality of stormwater. The quality can be highly variable, depending on many factors, but stormwater is often highly polluted and may be a public health and environmental hazard. Land use and surface characteristics, amongst other factors, affect the quality of stormwater runoff (Duncan, 1995) and are important when modelling stormwater quality (Mannina & Viviani, 2010). Lim *et al.* (2011) note that SWH has not been widely practiced as there are concerns that, in general, stormwater pollution levels are unreasonably high. In the RSA, stormwater is generally considered highly polluted (Wright, 1996), especially microbiologically (Wright, 1993).

In Singapore, where urban stormwater is harvested for potable end uses, the design of the collection systems and land use planning has formed an important part of their SWH system plans. Lim *et al.* (2011) note land use restrictions were put in place to protect potential catchment areas, and that stormwater from areas which were expected to have poor water quality (e.g. industrial areas) was not harvested but released directly into the ocean. Additionally, stormwater management approaches which not only manage quantity but also quality (e.g. WSUD/SuDS such as filter trenches – discussed further in Appendix B), were implemented within the SWH catchment areas to minimise pollutants entering the collection system. Interestingly, this is similar to how the Atlantis Water Resource Management Scheme (AWRMS) has operated (DWAF, 2010). What is particularly noteworthy about Singapore's experience is that over 20 years of water quality data showed that the reservoirs storing harvested stormwater were of a comparable standard to those storing water harvested from upland (natural) catchments and within '*the limits stipulated by USEPA and WHO for drinking water quality (except for microbiological parameters which will be effectively removed by water treatment at the waterworks)* (Lim *et al.*, 2011)'. This is largely because the stormwater pollution levels were lower than is typically reported in the literature – which Lim *et al.* (2011) attribute to the design of the collection system and land use planning.

While Hatt *et al.* (2004a) found conventional piped systems to be the most common form of collection system for SWH, SuDS – comprehensively discussed in Woods-Ballard *et al.* (2007) and Armitage *et al.* (2013) – provide an alternative that can attenuate peak runoff, improve runoff water quality, offer amenity and enhance biodiversity. SuDS options such as swales may be used to treat and convey stormwater in an attempt to mitigate the impacts of urbanisation on receiving water bodies (Woods-Ballard *et al.*, 2007; Fletcher *et al.*, 2008). One potential concern relating to the use of SuDS to convey runoff for SWH schemes is the potential for evapotranspiration and infiltration losses. Mitchell *et al.* (2006) and Mitchell *et al.* (2007a) indicate that the quantity of these losses is dependent on many factors, including local climate, local soil types and catchment imperviousness. For a SWH system to be successful, it is important that the collection system be carefully designed – considering the advantages and disadvantages of different options – while taking into account the different land uses present in a catchment.

2.5.2 Treatment

The treatment of stormwater is an important part of a SWH system (Akram *et al.*, 2014). Water should be treated to a quality that meets but not necessarily exceeds end-use requirements (Mitchell *et al.*, 2007a). To minimise pollution, and the required degree of treatment, it is important to consider land-use planning and the design of the collection system – as discussed in Section 2.5.1. Currently, one of the major obstacles to the widespread implementation of SWH is a paucity of reliable and affordable treatment technologies (Hatt *et al.*, 2004b; Philp *et al.*, 2008). Hatt *et al.* (2004b) conclude with a warning that: '*Existing practice is far ahead of research, which may pose a danger to the future adoption of such measures. Just one high profile case of public health or environmental failure of a re-use project (conducted without sound scientific backing) could undermine public confidence in re-use nationally, costing our society time and money in the much needed adoption of future water re-use technologies'*. The users have not necessarily accepted the risks associated with SWH – so much as simply being unaware of them. Should a failure happen it may result in a negative attitude developing towards a specific technology e.g. SWH.

The treatment of stormwater that is to be harvested can be separated into two broad categories: SuDS treatment and advanced treatment (including disinfection).

2.5.2.1 Sustainable Drainage Systems treatment

Sustainable Drainage Systems – termed SuDS – were developed to protect receiving water bodies (Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013). SuDS aims to attenuate peak flows, treat water quality, and provide amenity and biodiversity. Such systems are typically designed to ensure that the treatment reduces the event mean concentration. However, if stormwater is to be harvested for use, '*a higher level of uniformity in treated water quality due to public health and safety considerations'* is required (Philp *et al.*, 2008). SuDS have commonly been used to treat stormwater prior to, and in some cases (e.g. retention ponds and wetlands) during, storage as part of an integrated SWH system (Hatt *et al.*, 2004b).

SuDS stormwater systems generally make use of number of SuDS treatment options that are arranged in a 'treatment train' (Armitage *et al.*, 2013). There are four key intervention points (corresponding to the scale of intervention) in the treatment train, namely: good housekeeping, source controls (including RWH), local controls, and regional controls – these are discussed in Armitage *et al.,* (2013). At each of these intervention points slightly different combinations of SuDS options are used to manage the stormwater. The selection of SuDS treatment options, and the arrangement of the treatment train, is an important design consideration due to the different treatment processes (physical, chemical, and biological) that take place by way of the different SuDS options. Many guidelines exist for the design of each technology, and more detailed information on their pollutant removal methods and capabilities are provided in, for example, AMEC *et al.*, 2001 and Woods-Ballard *et al.*, 2007. Hatt *et al.* (2004b) found that larger SWH systems typically make use of wetlands to treat the harvested stormwater. Mitchell *et al.* (2007a), however, noted that if wetlands are the only treatment method (i.e. a treatment train is not used), then such a wetland will likely need to be substantially larger than one which forms part of a 'treatment train', in order to ensure adequate and effective treatment.

2.5.2.2 Advanced treatment and disinfection

While SWH has been largely used for providing water for irrigation (Philp *et al.*, 2008), it may be used for a number of potable and non-potable purposes – as is the case in Singapore (Lim *et al.*, 2011). Mitchell *et al.* (2007a) note that disinfection may be required if harvested stormwater is to be used in ways that may result in human contact. The need for, and level of treatment, is based on a risk assessment, as laid out for example in NRMMC *et al.* (2006, 2008, 2009b), and depends on the end use – as discussed in Section 2.2.3.

Hatt *et al.* (2004b) note that the types of advanced treatment techniques found in SWH were characteristic of those used in potable water and wastewater treatment plants. These include: coarse and fine screening microfiltration, reverse osmosis, dissolved air flotation, electrolytic flocculation, aeration and biological treatment (Hatt *et al.*, 2006; Philp *et al.*, 2008).

Although SuDS treatment options and some advanced treatment methods will reduce pathogenic organism loads, it is generally still necessary to disinfect the water prior to use where human contact is likely (Philp *et al.*, 2008). Disinfection is essential if the intended end use involves human contact (Hatt *et al.*, 2004b). Table 2-10 provides an overview of common disinfection methods.

Table 2-10: Advantages and disadvantages of disinfection techniques (Philp *et al.*, 2008)

Method	Advantages	Disadvantages
Chlorination	• Stable and continuous disinfection • Well-developed technology • Low-cost, widely available	• Chlorine products are highly toxic and corrosive and therefore require special transport, storage and handling procedures • Strength of NaOHCl decays during storage. CaOHCl may crystallise and clog lines. Tends to be ineffective against viruses and protozoa • Requires mixing tanks to meet 10 - 25 min. contact time. May produce toxic by-products • May require post-disinfection de-chlorination, depending on the end use
Ultraviolet Radiation	• Chemical free • Small footprint • Instantaneous disinfection • No toxic by-products or residuals • Higher virus inactivation efficiency than chlorination	• Efficiency reduced by turbidity and suspended solids • Requires electricity which potentially adds to the operational costs • No residual disinfection, potential for photo-reactivation and mutation of the microbial population • Can be difficult to verify correct calibration of UV reactors in unattended locations
Oxidation (Ozonation)	• Reduces colour and odour • No dissolved solids production • May increase dissolved oxygen concentration • Reduces organic matter	• Ozone is toxic, highly unstable and must be produced on-site • Requires mixing tank to meet 5 -15 min. contact time • May form harmful by-products (e.g. bromates) • Poor water quality increases the required ozone dosage • High cost
Membrane Filtration	• Prevents bacterial regrowth • No toxic by-products • Produces high water quality	• High capital cost, Moderate to high operating cost • Backwash may be significantly contaminated by microbes • Requires chemicals for cleaning

2.5.3 Storage

There are a range of options for storing harvested stormwater. The options apply at different scales and depend on the intended applications for the harvested water. All designs need to consider: how the water will be collected, where it will be stored, the need and options for treatment and how it will be distributed to its end use (DECNSW, 2006). The design of the storage component of a SWH system is a trade-off between maximising volumetric reliability and minimising the required storage size and associated costs (Mitchell *et al.*, 2007a). This section provides an overview of the alternative options, which are discussed in more detail in, amongst others, Hatt *et al.* (2004a), Dillon (2005) and Philp *et al.* (2008). For this review, the alternative storage options have been divided into three categories: closed storage, open storage and managed aquifer recharge.

2.5.3.1 Closed storage

'Closed storage' refers to all forms of storage where water is stored in a storage unit that is sealed and in which incident precipitation and evaporation will not increase or decrease the stored volume. This could, *inter alia*, include: tanks and underground vaults (e.g. pipes).

Tanks are a widely used form of storage, but are generally used for RWH – Section 2.4. Tanks can, though, be used to store rainwater that runs off a number of roofs or properties (Hatt *et al.*, 2006; Begum *et al.*, 2008) – which is defined in this thesis as SWH (See Section 2.1). Most commonly, tanks are used in SWH to collect the runoff from a small catchment, for example, a number of roofs, a permeable pavement or a combination of runoff from an urban area. It is, therefore, unsurprising that, as the catchment size increases, the use of tanks decreases (Hatt *et al.*, 2006). A selection of the major advantages and disadvantages of using tanks to harvest stormwater are provided in Table 2-11.

Table 2-11: Advantages and disadvantages of tank systems for SWH

Advantages	Disadvantages
Mosquitoes can easily be managed if proper screens are installed (NRMMC *et al.*, 2008).	Roof materials may leach toxins (NRMMC *et al.*, 2008; RainWater Cambodia, 2011).
Managing at source helps mitigate the negative impacts of urbanisation on water quality and flow (Fletcher *et al.*, 2008; NRMMC *et al.*, 2008).	'Anaerobic conditions can develop in stormwater storage tanks where the stormwater has high levels of organic matter and the residence time is long…can lead to odour problems (NRMMC *et al.*, 2008)'.
Tanks are widely available in South Africa (Armitage *et al.*, 2013).	Relatively expensive means of harvesting and reusing stormwater (Marsden Jacobs Associates, 2006; Armitage *et al.*, 2013).

Permeable pavements are an example of a storage system that can also be designed to treat and store runoff for use at a later stage (Pratt, 1999; Beecham *et al.*, 2010). Permeable pavements designed for SWH typically harvest incident precipitation, and evaporation is limited.

2.5.3.2 Open storage

Open storage stormwater includes '*ponds, dams, constructed lakes and open water bodies such as lakes, rivers, streams and creeks'* (Goonrey, 2005). The use of natural water bodies such as natural wetlands should be discouraged to prevent irreparable damage as a result of pollutants (Armitage *et al.*, 2012). Figure 2-10 shows the conceptual design of an open storage system, e.g. a retention pond.

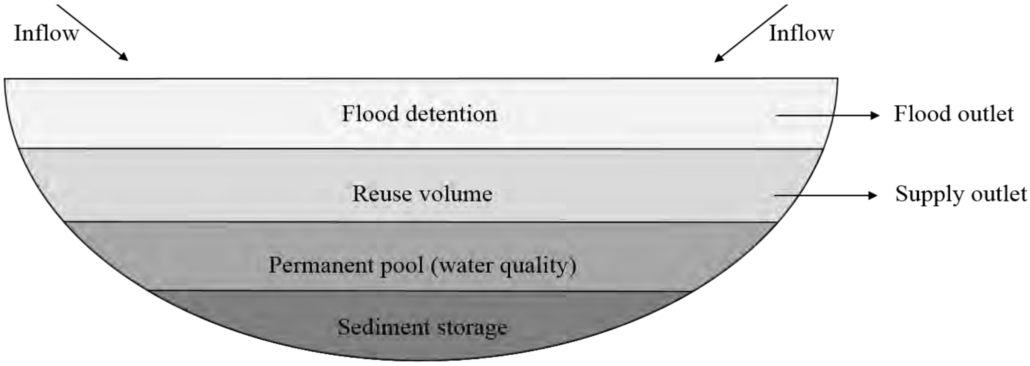

Figure 2-10: Conceptual design of an open storage stormwater harvesting system
(After DECNSW, 2006)

Open storage systems are attractive to a range of fauna, including water birds, the faeces of which may result in increased pathogen levels and thus a public health concern (DECNSW, 2006; Armitage *et al.*, 2013). On the other hand, open storage systems such as retention ponds are known to offer a range of benefits such as increased property values, recreational areas etc. A selection of advantages and disadvantages of open storage systems is highlighted in Table 2-12.

Table 2-12: Advantages and disadvantages of open storage

Advantages	Disadvantages
Low capital and maintenance costs, ease of construction (Goonrey, 2005)	Public safety (Philp *et al.*, 2008)
Provides ecosystem goods and services (Armitage *et al.*, 2013)	Public health – mosquitoes (Philp *et al.*, 2008)
Aesthetics: if properly maintained, may be an advantage, if not maintained, will be a disadvantage (Philp *et al.*, 2008; Armitage *et al.*, 2013)	

2.5.3.3 Managed Aquifer Recharge

Managed Aquifer Recharge (MAR) is the '*intentional banking and treatment of water in aquifers* (Dillon, 2005)'. In the past the term 'artificial recharge' (AR) was used to refer to

MAR. Dillon (2005) noted that the use of the term 'artificial' had a negative connotation when dealing with community participation and suggested MAR be used instead of AR.

MAR may be used for storing water for future reuse, treating water or maintaining the ecological reserve (NRMMC *et al.*, 2009b). There are many different approaches that may be used to recharge an aquifer; which approach is the most appropriate will depend on the characteristics of the aquifer, *inter alia*, whether it is confined or unconfined. The range of alternative approaches is illustrated in Figure 2-11. In the RSA, The Atlantis Water Resource Management Scheme is a well-known example of infiltration ponds having been used to recharge an aquifer for use at a later stage (DWAF, 2010). It has also been cited internationally as a '*a complex and large-scale urban stormwater collection system'* (Philp *et al.*, 2008).

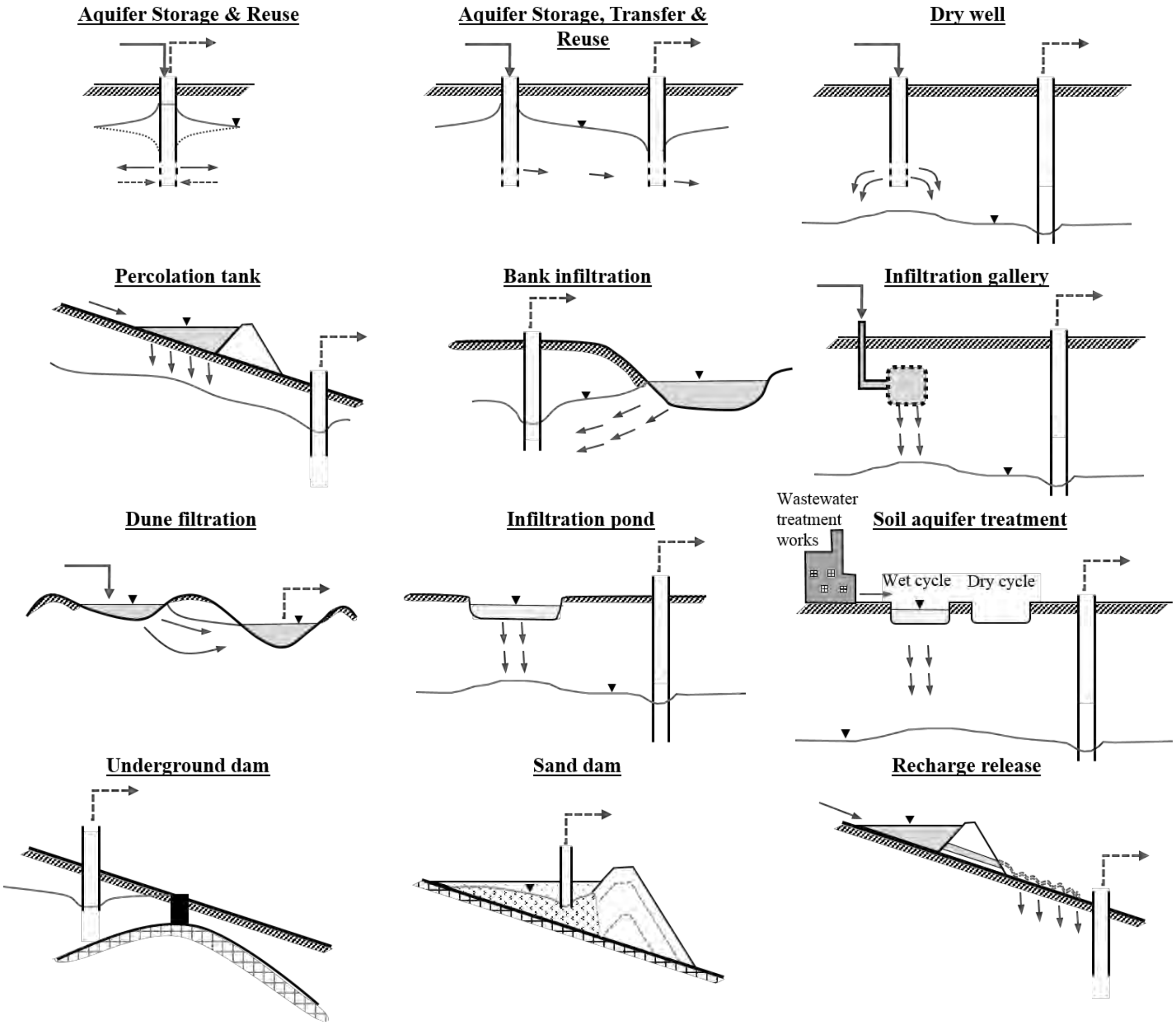

Figure 2-11: Types of managed aquifer recharge
(Dillon, 2005; NRMMC *et al.*, 2009b)

Due to the extensive options for aquifer recharge, and since MAR was not a viable option in the catchment used as a case study in this thesis, the different options are not reviewed in detail. Table 2-13, however, highlights some of the many advantages and disadvantages of MAR.

Table 2-13: Advantages and disadvantages of MAR

Advantages	Disadvantages
Limited space required (DECNSW, 2006).	Can potentially pollute aquifers (DECNSW, 2006; MBWCP & WBMOEE, 2006; NRMMC *et al.*, 2009b).
Prevents salt water intrusion resulting from over abstraction (DECNSW, 2006).	
Increased water supply security flood mitigation (NRMMC *et al.*, 2009b).	'For many managed aquifer recharge projects, the level of some risks can only be estimated before full-scale implementation and validation monitoring occurs' (NRMMC *et al.*, 2009b).
Extended retention times and filtration result in the removal of many pathogens (NRMMC *et al.*, 2009b).	
Generally, the most cost-effective option of harvesting stormwater when the geology is suitable (Dillon, 2005; Wong *et al.*, 2012).	Requires suitable geology (DECNSW, 2006; MBWCP & WBMOEE, 2006; NRMMC *et al.*, 2009b).
MAR makes it possible to harvest and reuse significant quantities of stormwater (NRMMC *et al.*, 2009b).	Cannot be used in areas with shallow unconfined aquifers (NRMMC *et al.*, 2009b).
Offers many ecosystem goods and services, which may result in increased property values, decreased downstream flooding etc. (NRMMC *et al.*, 2009b)	

2.5.4 Distribution

There are generally two categories of SWH distribution system: open space irrigation systems (e.g. drip or sprinkler irrigation system for a park, golf course or public open space) and non-potable distribution systems / dual reticulation systems (where a property is supplied with two water supply connections, one for potable water, and the other for non-potable water) (Mitchell *et al.*, 2007a). The type of distribution system is determined by many factors, including the spatial scale of the system, the number of end users and the desired end uses for the harvested water (Mitchell *et al.*, 2007a). Hatt *et al.* (2004a) showed that, in Australia, 50% of SWH systems were used for irrigation and 35% were dual reticulation systems. Irrigation systems were typically limited to a catchment smaller than 200 ha. Philp *et al.* (2008) note that stormwater harvesting in Australia rarely considers end uses beyond irrigation.

Mitchell *et al.* (2007a) note that '*much of the experience* [gained] *in designing, operating and maintaining a potable or recycled wastewater distribution system is transferable to a stormwater harvesting distribution system'*. While it may be necessary to gain some experience in designing and operating SWH distribution systems in the RSA, there are already established guidelines and local experience in designing potable water reticulation systems that could be leveraged for designing SWH systems. In the RSA, the design of potable water distribution systems is typically guided by the 'Guidelines for Human Settlement and Planning' (CSIR, 2005b).

2.5.5 Modelling stormwater harvesting systems

There has been a significant amount of research into the impact of computational methods on reservoir storage-yield predictions – focusing on sizing large, over-year dams or reservoirs for high levels of supply reliability (Mitchell *et al.*, 2008b). However, only Wanielista *et al.* (1991) and Mitchell *et al.* (2008b) have expressly reported on the impact of computational methods used for modelling SWH (Section 2.6.3). A SWH system is different from a RWH system in that, while SWH typically has a catchment with both pervious and impervious areas, a RWH system typically has a near 100% impervious catchment (roof). Furthermore, SWH storage may comprise open storage, closed storage, or underground aquifer storage, whereas RWH typically makes use of closed storage (a tank).

2.5.5.1 Modelling catchment runoff

The modelling of the storage component of a SWH system is, in process, common with RWH behavioural simulation models and, to prevent repetition, they are both discussed in Section 2.6.3. The modelling of the treatment and distribution is typically focused on the cost of treating and pumping to the point of demand. There are established guidelines for the design of such systems in the RSA (e.g. 'Guidelines for Human Settlement and Planning'; CSIR, 2005b) – as highlighted in Sections 2.5.2 and 2.5.4. The modelling of runoff from an urban catchment is very different from, and more complicated than that of a roof due to the catchment not being 100% impervious and is discussed below. It is easiest to make use of a widely used and accepted runoff module from existing stormwater modelling software – as was the case in Mitchell *et al.* (2008b). A number of relevant models have been highlighted in Section 2.6.1 and Table 2-14.

Mitchell *et al.* (2008b) made use of the *MUSIC* (eWater, 2009) rainfall-runoff calculation module to calculate the inflow into the SWH storage unit. In *MUSIC,* runoff from pervious and impervious areas is calculated independently. Runoff from impervious areas occurs when the impervious depression store capacity is exceeded. Runoff from pervious areas occurs when the pervious depression store capacity and/or infiltration rate is exceeded. *Aquacycle* (Mitchell, 2004) separates the runoff into two components for modelling purposes (surface runoff and base flow). Both models consider impervious and pervious areas independently as well as accounting for flow from impervious areas onto pervious areas.

One of the best known and most widely used stormwater models is the USEPA's Storm Water Management Model (*SWMM*) (Fletcher *et al.*, 2013). Ashbolt *et al.* (2013) made use of *SWMM* to analyse the impact of urbanisation on an undeveloped catchment and the potential for SWH to contribute to maintaining predevelopment flows. From a stormwater modelling perspective, as with most models, calibration is essential (James, 2005). Software models such as *MUSIC* and *SWMM* require significant calibration to provide reasonable results (Fletcher *et al.*, 2013). One disadvantage of using *SWMM* compared with *MUSIC*, is that the software as it stands does not allow for costing / economic simulation. It is widely used in the RSA, though.

2.5.5.2 Regional scale harvesting

A recent study by Neumann & Maheepala (2013) into the effect of spatially lumping (where multiple SWH systems are modelled as a single system) SWH systems found that, generally, spatially lumping overestimates the yield and underestimates the overflow of SWH – as was the case with RWH (Section 2.4.5.3). They do, however, state the '*overall conclusion of the study is that the input variables of many stormwater harvesting systems spread across a catchment can be linearly combined (or summed) into a single system without introducing significant errors provided that the individual harvesting systems are well designed (i.e. storage volume just adequate to acquire the required yield)*'. This study was based on areas in Australia, and there is, therefore, a need to test these findings in a different context.

2.6 Modelling considerations and specifics

RWH and SWH share a number of similarities, and with regard to modelling there are areas of significant overlap; these aspects are thus discussed together in this section. This includes: what models are available; what level of complexity is required when modelling; the modelling of a storage unit; modelling of rainfall; modelling of water demand; economic modelling and assessment methods; performance assessment parameters; and the modelling of RWH and SWH in conjunction.

2.6.1 RWH and SWH management frameworks, models, tools and software

RWH and SWH impact on more than a single stream (e.g. water supply and drainage) of the urban water cycle (discussed in Appendix B), and thus should be modelled in an integrated manner. Unsurprisingly, therefore, there is a drive to develop software models capable of modelling the urban water cycle in a holistic manner. This section outlines what frameworks, tools, and software packages are available for modelling the urban water cycle, with a focus on which of these are most appropriate for modelling RWH and SWH.

Models simplify reality into a form that can be understood and worked with, and have become essential tools in the management of water systems (Van Waveren *et al.*, 1999; James, 2005; Wainwright & Mulligan, 2013). Van Waveren *et al.* (1999) and Wainwright & Mulligan (2013) provide a comprehensive review of: types of model, the purpose of modelling, uncertainty when modelling, calibration, sensitivity analysis and an overview of responsible / best practice when modelling water systems. Zoppou (2001), Elliott & Trowsdale (2007), Mitchell *et al.* (2007b), Last (2010) and Bach *et al.* (2014) provide a comprehensive review of the available models for modelling urban water systems both independently and in an integrated manner. Fletcher & Deletic (2008) detail the data requirements for integrated water management and consider how it affects modelling.

Software, essentially computer programs (Merriam Webster, 2012a), are essential for modelling a modern water system. Equally important is the framework, defined as '*a basic* *conceptual structure*' (Merriam Webster, 2012b), that aids in guiding research and setting up models. Selecting the most appropriate model is crucial. This section outlines what tools and frameworks are available. Chiu *et al.* (2008), Mwenge Kahinda *et al.* (2008), Goonrey *et al.* (2009), and Darshdeep & Litoria (2009) have all presented frameworks for identifying suitable sites for implementing stormwater harvesting and reuse. Some of these frameworks make use of GIS to identify applicable sites (Chiu *et al.,* 2008; Mwenge Kahinda *et al.,* 2008; Darshdeep & Litoria, 2009). Goonrey *et al.* (2009) on the other hand presented a comprehensive decision-making framework (DMF) for implementing stormwater harvesting and reuse – Appendix C. There are also many frameworks dealing with specific aspects of stormwater harvesting and reuse. For example, NRMMC *et al.* (2008) offer a framework specifically for managing the health and environmental risks of stormwater harvesting. Ilemobade *et al.* (2009) provided a framework for assessing the viability of dual reticulation – a means through which regionally harvested stormwater could be supplied to households.

An extensive review of urban water models by Breen *et al.* (2006) found that the available models failed to balance between the scope and detail. More recently, Bach *et al.* (2014) found that much of the literature focussed on modelling one component of the urban water cycle – urban drainage (collection and disposal of sewage and stormwater, but not SWH which would require modelling of supply to meet demand) – instead of the urban water cycle in an integrated manner. As a consequence, Akram *et al.* (2014) found that '*there are still noticeable lackings in the field of stormwater modeling that covers all the major elements of a stormwater harvesting system in an integrated manner'.* Fagan *et al.* (2010) highlight the fact that currently-available integrated urban water models do not generally provide adequate results to fully inform decisions. For example, there are models such as *Infoworks* (Innovyze, 2011), that have the ability to undertake detailed design of water supply, sanitation and drainage systems, but fail to integrate the three streams of the urban water cycle. Models such as *UVQ* (Mitchell & Diaper, 2005), *Aquacycle* (Mitchell, 2004) and *Watercress* (Clark *et al.*, 2002) represent the entire water cycle, but do so in a simplistic manner using a system-wide water balance (Breen *et al.*, 2006). A relatively new model *Urban Developer*, was developed in an attempt to address these shortcoming and aims to model the urban water cycle and components of it in an integrated manner (Snowdon *et al.*, 2011).

Zoppou (2001), Elliott & Trowsdale (2007), Mitchell *et al.* (2007b), Last (2010) and Bach *et al.* (2014) provide a comprehensive review of the available (over 100) models for modelling urban water systems – both independently and in an integrated manner. A review by Armitage *et al.* (2014) identified seven modelling packages, presented in Table 2-14, that seemed to offer the most potential for investigations of stormwater and urban water systems in the RSA. A detailed discussion of these models and their applicability within the RSA is provided in Armitage *et al.* (2014). The selection of the most appropriate models for this thesis is discussed in Section 4.

Table 2-14: Potential software options for this study

Software (Developer)	Intended use
SWMM (USEPA)	*SWMM* is a software package that enables dynamic rainfall-runoff modelling, and can be used to model long term or single rainfall events. The model simulates the quantity and quality of runoff that emanates from an urban environment. It is considered a '*detailed model for planning and preliminary design'* (Elliott & Trowsdale, 2007).
Source Urban (eWater)	Is a node-link model with nodes representing storages, inflows, demand etc. The nodes and links are used to build a schematic representation of a water system. Water can then be dynamically allocated to demand nodes using defined rules or network linear programming (eWater, 2015b).
MUSIC (eWater)	Is a Decision Support System (DSS) for developing conceptual designs (Elliott & Trowsdale, 2007; Wong *et al.*, 2002). *MUSIC* is used to analyse the conceptual designs of stormwater infrastructure and places particular emphasis on water quality objectives (Elliott & Trowsdale, 2007).
Urban Developer (eWater)	'*Urban Developer simulates the water supply, stormwater, and wastewater systems at a range of spatial and temporal scales within a single framework to improve the understanding of the potential of integrated urban water management* (Snowdon *et al.*, 2011).'
Aquacycle	'*Aquacycle is a daily urban water balance model which has been developed to simulate the total urban water cycle as an integrated whole and provide a tool for investigating the use of locally generated stormwater and wastewater as a substitute for imported water alongside water use efficiency. The model is intended as a gaming tool rather than a design tool* (eWater, 2015a).'
SLAMM (V10) (PV & Assoc)	*SLAMM* is a planning level tool aimed at predicting flow and pollutant discharges from a broad range of development scenarios with many different combinations of stormwater controls (PV & Associates, 2012).
ArcGIS (ESRI)	*ArcGIS* is a complete system for managing and processing geographic information(ESRI, 2012).
SUSTAIN (USEPA)	'*SUSTAIN is a decision support system to facilitate selection and placement of Best Management Practices (BMPs) and Low Impact Development (LID) techniques at strategic locations in urban watersheds*' (USEPA, 2012d). It is a tool capable of performing a comprehensive analysis of stormwater management strategies at multiple scales including basic hydraulic functioning and economic analysis. It may be used to evaluate, select and place structural BMPs within a catchment on the basis of cost and effectiveness criteria.
WEAP	The Water Evaluation and Planning System (*WEAP*) is a water balance model. It can be used to model both urban and agricultural systems. *WEAP* can be used to model a range of scenarios including, *inter alia,* '*sectoral demand analyses, water conservation, water rights and allocation priorities, groundwater and streamflow simulations, reservoir operations, hydropower generation and energy demands, pollution tracking, ecosystem requirements, and project benefit-cost analyses* (Sieber & Prukey, 2011)*'*.

2.6.2 Model complexity

'*Something is complex if it contains a great deal of information that has a high utility, while something that contains a lot of useless or meaningless information is simply complicated'* (Grand, 2000).

Wainwright & Mulligan (2013) state that an '*optimal model is one that contains sufficient complexity to explain phenomena, but no more'.* James (2005) suggests that it is sometimes assumed that the reliability of a model will increase with its complexity to a certain point, and beyond that, the reliability will decrease (Figure 2-12). James (2005) notes that this has never been proven for surface water models, but that Qaisi (1985) proved it for modelling lake chemicals. Therefore, taking a parsimonious approach to modelling – developing a model with the greatest explanatory power and the fewest parameters or complexity – *is a particularly important principle in modelling since our ability to model complexity is much greater than our ability to provide the data to parameterize, calibrate and validate those same models'*(Wainwright & Mulligan, 2013).

It is difficult to determine the required level of complexity, as there is no accepted measure of this (James, 2005). However, experience and intuition will assist in the development of good models (Wainwright & Mulligan, 2013).

Data are crucial for the development and calibration of reliable models. In theory, the more data available, the more reliable the model should be (James, 2005). There is a relationship between complexity and the amount of data that is available – as shown in Figure 2-13 – which suggests that a more complex model will be more uncertain than a less-complex model with minimal data, but less uncertain than a less-complex model with a lot of data.

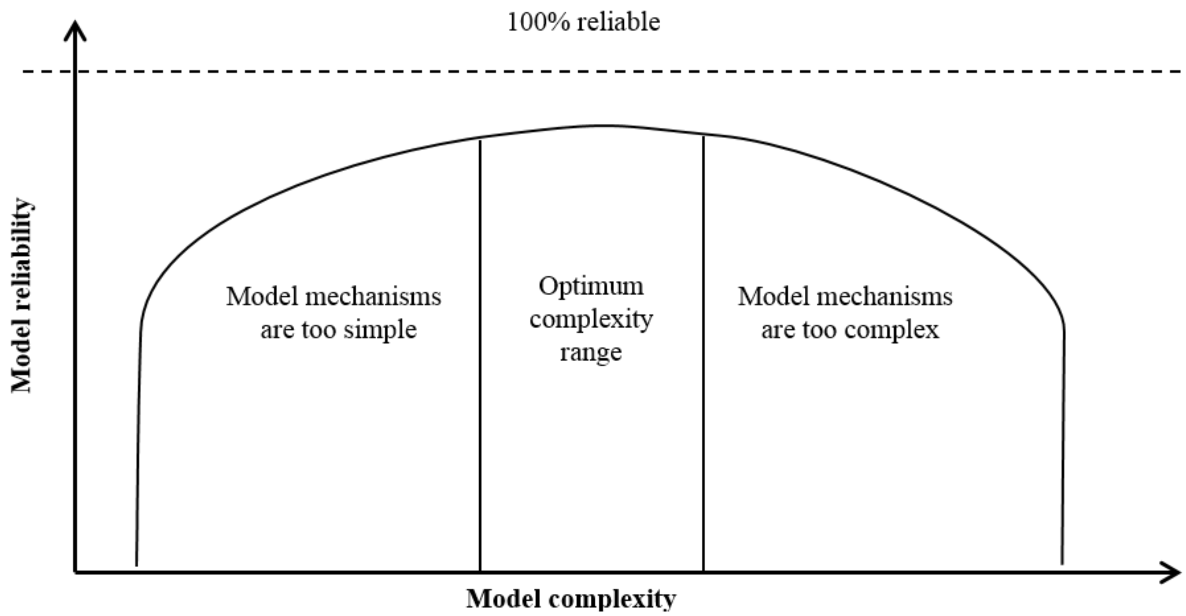

Figure 2-12: Relationship between complexity and reliability (After James, 2005)

In essence, choosing the correct level of complexity is a difficult but important part of modelling. Models should be neither overly complex nor too simple. Overly complex models will consume more time and money and potentially offer less reliable results. On the other hand, a model that is too simplistic may not offer adequate reliability (van Waveren *et al.*, 1999; James, 2005; Wainwright & Mulligan, 2013). Ideally it is '*no more complex a model or representation of reality than is absolutely necessary'* (Wainwright & Mulligan, 2013). This is particularly pertinent in the light of the discussions regarding linearly extrapolating results to the catchment scale (Sections 2.4.5.3 and 2.5.5.2).

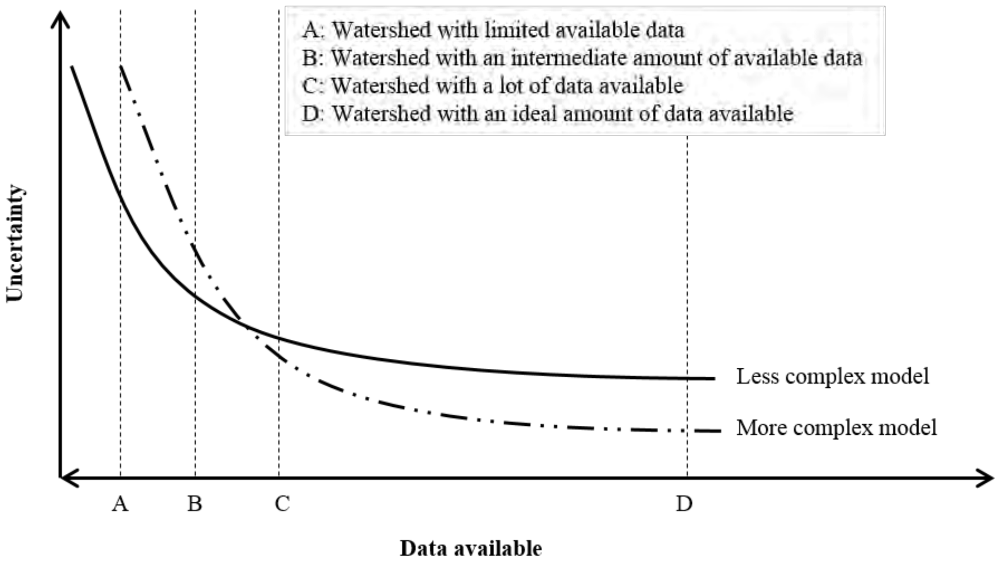

Figure 2-13: Relationship between data, uncertainty and complexity (After James, 2005)

2.6.3 Modelling storage (YAS or YBS)

Section 2.1 defined RWH and SWH as the collection, storage and use of runoff. 'Rainwater harvesting' was defined as the harvesting of roof runoff at the site scale, while SWH is the neighbourhood-to-catchment scale harvesting of runoff from all urban surfaces. While the modelling of runoff, collection (inflows) and distribution (outflows) in RWH (Section 2.4.5) and SWH (Section 2.5.5) systems is different, the modelling of the storage component of RWH and SWH (open and closed storage) systems is generally the same and makes use of behavioural simulation models.

Behavioural simulation models simulate the operation of a storage unit – be it a tank, reservoir, or pond – with respect to time by routing simulated mass flows through an algorithm that describes the operation of the storage unit (Fewkes & Butler, 2000; Fewkes & Wam, 2000; McMahon & Adeloye, 2005). The balance of the flows is calculated using Equation 2-5 and is illustrated in Figure 2-14. In the case of a sealed storage unit, such as a tank, the evaporation and incident precipitation terms in Equation 2-5 fall away (Mitchell, 2007).

$$V_t = V_{t-1} + I_t + P_t - E_t - S_t - L_t - Y_t \qquad \mathbf{2\text{-}5}$$

Where: V_t is the storage volume at the end of the current time step t, V_{t-1} is the storage volume at the end of the previous time step t and I_t is the inflow from the catchment during time t (See Sections 2.4.5.2 and 2.5.5.1 for a discussion as to how these values are calculated / modelled for RWH and SWH, respectively), P_t is incidental rainfall during time t, E_t is the evaporation from the storage unit during time t, S_t is the overflow / spillage during time t, L_t is seepage and/or leakage losses during time t, and Y_t is the yield / water demand during time t.

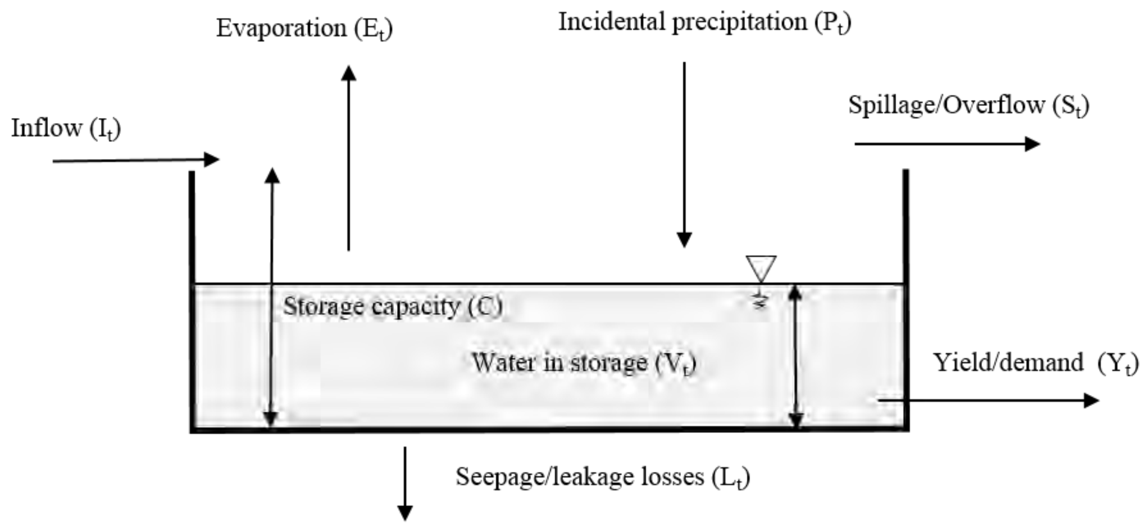

Figure 2-14: Representation of a storage unit
(After Mitchell, 2007; Roebuck, 2007; Mitchell *et al.*, 2008b)

Behavioural simulation models may be used with any time step (Liaw & Tsai, 2004), although studies have shown that the smaller the time step, the more accurate the results tend to be (Mitchell, 2007; Mitchell *et al.*, 2008b; Campisano & Modica, 2014). One limitation of behavioural simulation models, however, is that because they make use of a discrete time step, the model assumes all inflows and outflows during a specific time step occur instantaneously at the end of that time step. However, the inflows, outflows and over-flows are interrelated. For example, the overflow is affected by the quantity of inflows and outflows at any point in time. A model based on a discrete time step cannot model these events simultaneously which would require much more data and the use of differential equations to accurately represent the events in a continuous manner (Roebuck, 2007). Instead, it is assumed that the inflows and outflows occur independently of each other in a specific order. Within a time step, the estimated volume of spillage is affected by the order in which inflows and yield / water demand is realised (Fewkes & Butler, 2000; Liaw & Tsai, 2004; Mitchell, 2007; Roebuck, 2007; Mitchell *et al.*, 2008b; Islam *et al.*, 2010), as illustrated in Figure 2-15. For example, at A, B and D, it doesn't matter which order the calculations take place, but at C, because there is overflow, the volume of overflow will differ if it is calculated before or after the yield / water demand is subtracted.

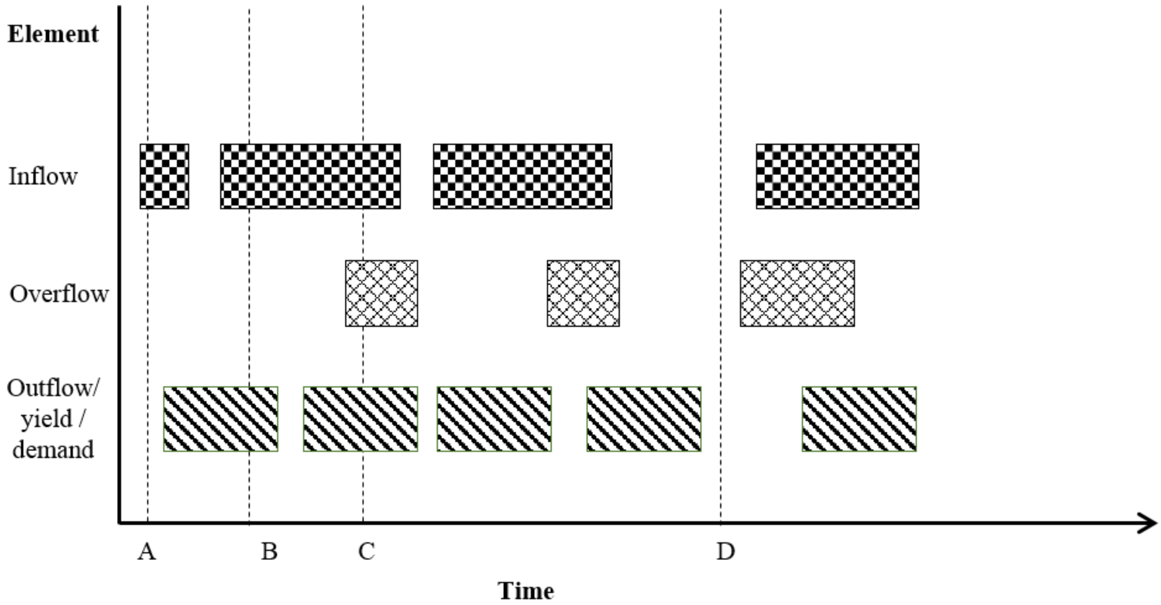

Figure 2-15: Possible water fluxes occurring simultaneously within a storage tank
(After Roebuck, 2007)

Jenkins *et al.* (1978) are credited with having developed the two fundamental algorithms to describe the operation of the storage unit (Fewkes & Butler, 2000; Roebuck, 2007). These are known as the 'yield after spillage' (YAS) and 'yield before spillage' (YBS) operating rules. The YAS operating rule is described mathematically by Equations 2-6 and 2-7.

$$Y_t = \min(D_t, V_{t-1}) \qquad \textbf{2-6}$$

$$V_t = \min(C_t - Y_t, V_{t-1} + I_t + \mathrm{P_t} - \mathrm{E_t} - Y_t) \qquad \textbf{2-7}$$

The YBS operating rule is described mathematically by Equations 2-8 and 2-9.

$$Y_t = \min(D_t, V_{t-1} + I_t) \qquad \textbf{2-8}$$

$$V_t = \min(C_t, V_{t-1} + I_t + \mathrm{P_t} - \mathrm{E_t} - Y_t) \qquad \textbf{2-9}$$

Where the terms are as previously defined for Equation 2-5. The calculation procedure for each algorithm and its potential impact on the final result is graphically illustrated in Figure 2-16.

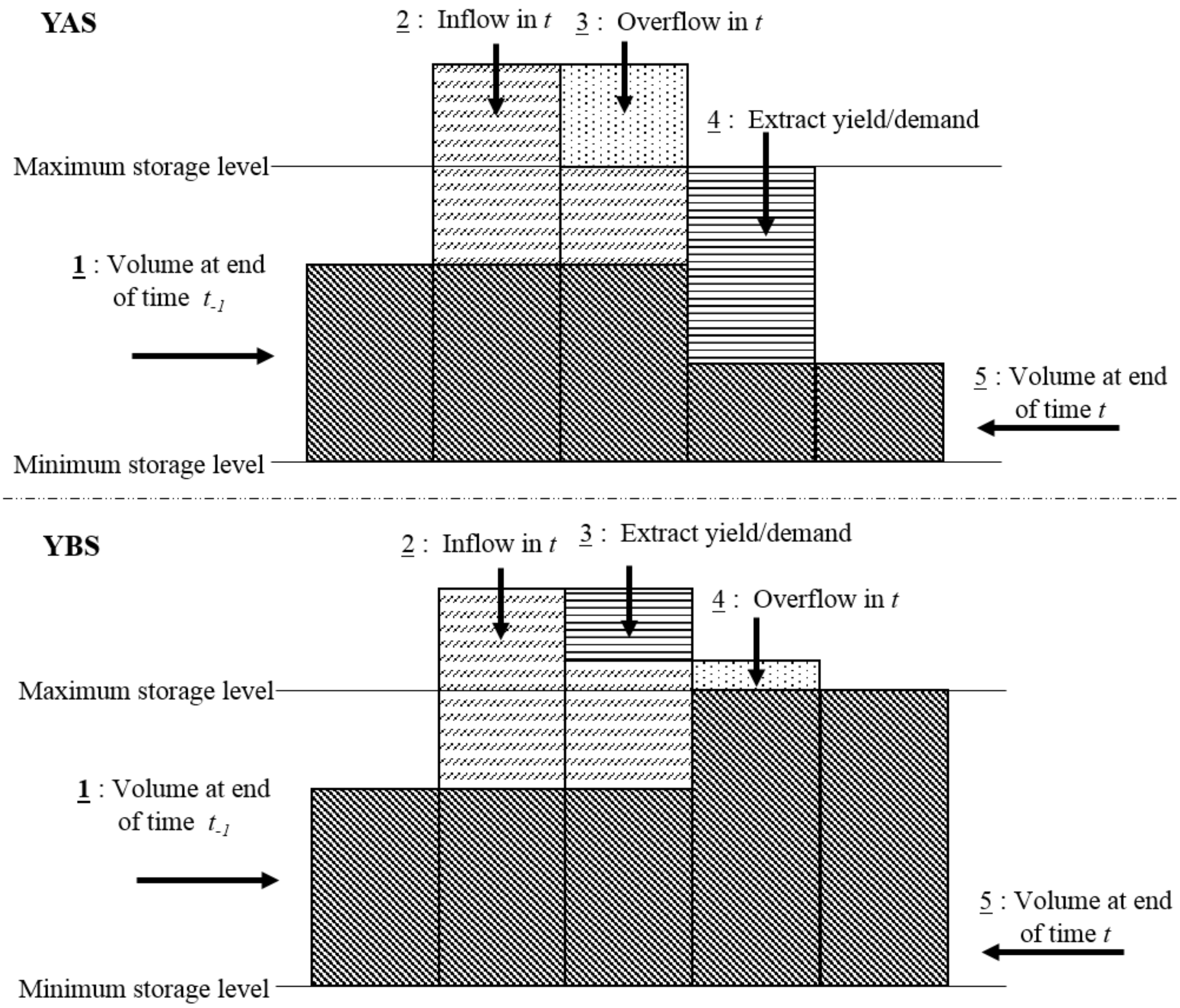

Figure 2-16: The difference between YAS and YBS operating rules
(After Roebuck, 2007)

Many authors have investigated the impact of using the YAS and YBS operating rules. A summary of their findings is presented in Table 2-15. Most of the research, with the exception of Mitchell *et al.* (2008b), has focused on RWH systems. The YAS operating rule was recommended as it was generally found to be more conservative in its results and it is therefore, widely used (e.g. Fewkes & Warm, 2000; Domènech & Saurí, 2011; Palla *et al.*, 2011; Roebuck *et al.*, 2011; Campisano *et al.*, 2013; Campisano & Modica, 2014).

2.6.4 Rainfall

Rainfall is a key factor in the performance and analysis of RWH and SWH systems (DECNSW, 2006; Roebuck, 2007). Rainfall varies depending on location, season and year (Thomas, 2002), and its characteristics (intensity and duration) can have a significant impact on the volume and rate of runoff from the catchment (Wilson *et al.*, 1979). Knoesen & Smithers (2008) note that

Table 2-15: Use of YAS/YBS operational rules in RWH and SWH studies

Reference	Systems analysed	Findings / conclusions
Fewkes & Butler (2000)	RWH systems	• Suggested that an hourly YAS model could be used as a standard against which other models could be compared and calibrated. • 'Hourly models should be used for sizing small stores with a storage fraction below or equal to 0.01. Daily models can be applied to systems with storage fractions within the range 0.01-0.125'.
Liaw & Tsai (2004)	RWH systems	Suggested that the YBS release rule is more effective than YAS under various conditions. However, Roebuck (2007) noted that, '*this appeared to be a case of the researchers choosing the modelling approach based on a predefined notion of what results would be acceptable'.*
Mitchell (2007)	RWH systems	Found that the YAS operational rule underestimated the yield and volumetric reliability, while the YBS operational rule produced an overestimation. Additionally suggested that the average of the YAS and YBS could be used to increase the accuracy of the results.
Roebuck (2007)	RWH systems	Found the YAS operating rule was appropriate in most cases, except in a study of a school. The difference in this case was not considered important.
Mitchell *et al.* (2008b)	SWH systems	'Several parameters had no significant influence on volumetric reliability: diurnal and weekly demand pattern, initial storage volume, and time-step in combination with YAS. However, the estimate of volumetric reliability is influenced by the length of climate record used, inter-annual variability of seasonal demand, open storage surface area, dead storage capacity and also time-step in combination with YBS'.
Islam *et al.* (2010)	RWH systems	YAS and YBS provided similar results. YAS was more conservative and so was used for further analysis.

the temporal distribution of rainfall – the distribution of rainfall intensities during an event – can significantly impact the timing and magnitude of the peak flow that is experienced during a storm event. As a result, the rainfall data used in this thesis were of critical importance as one of the benefits associated with RWH and SWH is the potential to attenuate peak stormwater flows (see Section 2.2.1.1). A failure to represent the magnitude of the rainfall will result in a poor representation of, and thus a poor assessment of the potential for RWH and SWH to attenuate, peak flows. Therefore, the following sub-sections focus on important considerations when obtaining and using rainfall data. This includes, *inter alia*: what rainfall data are required, how they might be obtained / generated, what limitations are commonly experienced and how they might be overcome, and what impact climate change might have on rainfall characteristics.

2.6.4.1 Hydrological year

The hydrological year can be defined as a '*continuous 12-month period selected in such a way that overall changes in storage are minimal so carry over is reduced to a minimum'* (UNESCO, 1992). WGMS (2010) suggests that in the Northern Hemisphere the hydrological year should start on 1 October and end on 30 September whilst in the Southern Hemisphere the hydrological year should start on 1 April and end on 31 March. However, in a study undertaken in South Africa, Gericke *et al.* (2004) used 1 October to 30 September, whilst the CoCT considers the hydrological year to run from 1 November to 31 October (CoCT, 2001). Van Waveren *et al.* (1999) suggest that, for calibration purposes, the use of the parameter of an average hydrological year can lead to error, and it may be best to consider a period that could represent an almost steady state. Van Waveren *et al.* (1999) thus suggest from the end of one rainy season to another.

Mitchell (2007) suggests that, when modelling RWH systems, it is preferable to model the initial conditions as having an empty storage unit / tank as it produces more accurate estimates of the yield. Mitchell (2007) notes that this becomes more significant with shorter simulation periods used to model RWH systems. How the hydrological year is defined is thus significant – especially for RWH systems – since, should the beginning of the hydrological year be in the middle of the wet season, it is likely that the storage unit would not be empty, meaning the initial conditions might not represent reality and could impact on the reliability of the results.

When modelling SWH systems, Mitchell *et al.* (2008b) found that the initial storage conditions had an insignificant impact on their results. Therefore the selection of the hydrological year is of little consequence.

2.6.4.2 Historic rainfall data versus stochastic data

Historic rainfall data are commonly used for modelling (Roebuck, 2007; DeBusk & Hunt, 2014). Historic data are data collected in the form of a time series – for example a continuous data set that records the depth of rain falling for a specific period of time and presented as depth per unit time, i.e. mm/day (Roebuck, 2007). Numerous studies considering RWH and SWH have made use of historical rainfall data; for example, Fewkes & Butler (2000), Liaw & Tsai (2004), Mitchell *et al.* (2005), Ghisi *et al.* (2006), Roebuck (2007), Burns *et al.* (2010) and Jones & Hunt (2010).

Due to limited (e.g. resolution, duration) and poor-quality data, it is not always possible to use historic rainfall data. To overcome this, many researchers – e.g. Young *et al.* (2002) and Cowden *et al.* (2008) – have made use of synthetically generated rainfall data. Stochastic rainfall models are calibrated against the statistical properties of measured data (James, 2005; Roebuck, 2007). Stochastic rainfall models have also been used in studies like Coombes *et al.* (2002a) to extend the period of analysis, for example, to 1000 years. The use of historic data results in an analysis based on one weather pattern, so stochastic models can also be used to create alternative weather patterns (Taulis & Milke, 2005). In arid areas, stochastically generated rainfall data can have significantly different statistical properties (probability of a wet day given a wet / dry day, mean rainfall etc.) to the historic data (Taulis & Milke, 2005). This may negatively affect the representivity of stochastic modelled rainfall for months of the year which typically receive little rainfall.

In the RSA the quality of historic records and availability of sub-daily rainfall data are often limited, therefore the next sections discuss the suitability of rainfall data, how it may be possible to make use of stochastic models by disaggregating daily rainfall data into sub-daily data, and what impact climate change might have on rainfall characteristics.

2.6.4.3 Suitability of rainfall data

'*Rainfall time series exhibit considerable variability over a hierarchy of timescales: within storm, between storm, seasonal, inter annual, inter decade etc.'* (Menabde & Sivapalan, 2000). Research has shown that there is significant temporal and spatial variability in rainfall data in runoff and water quality simulations (Wilson *et al.*, 1979; Debele *et al.*, 2007). Furthermore, as previously discussed in Sections 2.4, 2.5 and 2.6.3, the time step used is important for modelling peak flows of an urban catchment. The use of too big a time step has been shown to lead to an underestimation of peak flows at the catchment scale (Ormsbee, 1989; Aronica *et al.*, 2005). Ormsbee (1989) notes that the time step should be significantly less than the catchment's time of concentration, highlighting that, in small catchments, this would be less than an hour, possibly as small as five minutes. Ashbolt *et al.* (2013) used an hourly time step (for a hydrological model of the Upper Yaun Creek, Australia), as the time of concentration was greater than 1 hour and analysis indicated the hourly time step captured the peak flow fairly well. Schilling (1991) suggests a one-minute time step, while Berne *et al.* (2004) suggest a three- to five-minute time step for urban hydrological models ranging between 100-1000 ha respectively. It is, therefore, important to carefully consider the suitability of rainfall data, whether historic or stochastically generated, and what an appropriate time step would be. Roebuck (2007) suggests that, when using historic rainfall time series / data, it is necessary to consider three questions: what is a suitable time step, what is a suitable length of rainfall record and how close does the RWH system need to be to the location of rain depth measurement? These three questions are equally important when making use of a stochastically generated rainfall data set / time series.

The selection of an appropriate time step has been shown to impact the accuracy of different methods of analysis. Mitchell (2007) found that, for modelling RWH, the choice of Yield After Spillage (YAS) / Yield Before Spillage (YBS) algorithms (see Section 2.6.3) for modelling storage, became more important as the time step increased. Campisano & Modica (2014) found that increasing the time step used for analysis made a significant difference to the modelled performance of smaller rainwater tanks in particular. The difference in performance is likely as a result of a longer time step (e.g. daily) model failing to account for the continuous refilling of the storage unit. This modelling consideration is discussed further in Section 2.6.3. When modelling RWH at a cluster scale using the YAS approach, Mitchell *et al.* (2008b) found that the use of a daily time step made an insignificant difference when compared to modelling with a six-minute time step. When modelling SWH using the YAS algorithm, the time step had no significant impact on the volumetric reliability of the system (Mitchell *et al.*, 2008b). Another important consideration when selecting an appropriate time step for rainfall data to be used for modelling SWH systems is the antecedent soil conditions – the condition of the soil at the start of a storm event. As discussed in Section 2.5.5, the modelling of runoff and consequently inflows into a SWH system's storage unit is more complex than that of a RWH system due to the presence of both impervious and pervious areas in the system's catchment area. The use of too large a time step will affect the amount of infiltration realised, and therefore the volume of runoff.

The selection of the length of rainfall time series data needs to consider local climatic variations which have been shown to affect the reliability of the results of an analysis (Mitchell, 2007; Roebuck, 2007). Table 2-16 provides an overview of studies that have looked at the importance of the length of the rainfall time series / data. It is evident from Table 2-16 that while longer (as long as possible) periods of rainfall data are preferred, a number of authors have considered 10 years of rainfall data to be adequate.

Rainfall data should ideally be sourced from a rainfall monitoring station on the site under investigation. This, however, is not always possible and researchers are therefore often forced to look for data from a rainfall monitoring station that is likely to have similar rainfall characteristics to the area under consideration. Common practice among hydrologists in such situations is to use rainfall data obtained from a nearby rainfall monitoring station (Debele *et al.*, 2007). Roebuck (2007) suggests that rainfall data should be obtained from a monitoring station '*subject to a similar climate, and that is located close to, the site under investigation*'. Debele *et al.* (2007) note that, in studies by Habib *et al.* (2001) and Bradley *et al.* (2003), it has been '*reported that spatial rain gage distributions as dense as one rain gage every 100m to few 100 m away did not produce uniform readings, implying that denser rain gage distributions should be used to accurately represent the reality on the ground*'. Clearly it is unreasonable to monitor a catchment's rainfall at such a fine resolution, but it does imply that the use of rainfall data needs to be carefully considered. Berne *et al.* (2004) suggest rainfall data should be collected with a spatial resolution of 2-3 km, while Schilling (1991) suggests a 1 km^2 resolution.

In principle, it would be ideal to have rainfall data with a fine time step (e.g. 5 minute), a long record (>50 years) and with as many rain gauges as possible. Since this is unrealistic in many places, it is important to consider the impact of not using 'ideal' data which include, *inter alia*, underestimating peak flows, poor representation of long term climate trends, etc.

2.6.4.4 Disaggregation of rainfall data

For reasons already discussed, it is ideal to have rainfall data that are temporally and spatially representative. Segond *et al.* (2006) note that reasonably long rainfall records are commonly available for daily data, while data at finer time steps are often limited. Where daily rainfall data are available, disaggregation models have been used to reduce the temporal resolution to create finer time steps (James, 2005). The disaggregated rainfall data, which approximates high resolution rainfall data, may then be used as a substitute for high resolution historic data where this does not exist.

Table 2-16: Summary of advice for selecting historic rainfall data

Source	Recommendations, comments, or analysis period used for modelling
Wanielista *et al.* (1991)	• > 15 years (for modelling stormwater harvesting)
Gould & Nissen-Petersen (1999)	• A minimum of 10 years of historic data is adequate • 20-30 years of historic data is preferable
Fewkes (1999)	• 50 years*
Heggen (2000)	• Minimum of 5 years of historic data • Most water supply projects use 20–40 years of historic data
Herrmann & Schmida (2000)	10 years*
Konig (2001)	• Minimum of 10 years of historic data • Data to be obtained from the nearest station
Thomas (2002)	• Low-cost and low-security systems can be modelled with 5 or 10 years' worth of data • Large RWH systems that constitute a supply of last resort in arid areas require at least 25 years' worth of data
Yuan *et al.* (2003)	• 10 years* • Undertook an economic analysis of agricultural RWH
Liaw & Tsai (2004)	At least 50 years of historic data
Taulis & Milke (2005)	'*The longer the weather record, the better the risks associated with these projects or policies can be assessed*'
Mitchell (2007)	• 1 year of historic data is not recommended • 10 years of historic data is adequate (if representative of long-term trends) • 50 years of historic data is best
Xiao *et al.* (2007)	• At least 10 years of historic data
Basinger *et al.* (2010)	25 years*
Mwenge Kahinda *et al.* (2010)	20 years*
Neumann *et al.* (2011)	50 years*
Palla *et al.* (2011)	30 years
Roebuck *et al.* (2012)	• 37 years* • Used all the available historical data
Ghisi *et al.* (2012)	"*For the cases analysed, it was found that for all situations in which the short-term time series contained more than 10 years, the difference in the potential for potablewater savings in relation to the long-term time series was minimal.*"
Seo *et al.* (2015)	30 years*

* Analysis period used for modelling

There has been a significant amount of research in the field of rainfall disaggregation over the last few decades (Gyasi-Agyei, 2011). Amongst others, Burian *et al.* (2001), Debele *et al.* (2007), Nijssen *et al.* (2009), Gyasi-Agyei (2011), Safeeq & Fares (2011), Fletcher *et al.* (2013)

and Lu & Qin (2013) provide an overview of the numerous methods available for disaggregating rainfall data at a single rainfall station. Lu & Qin (2013) point out that the majority of models developed to date can only disaggregate rainfall data at a single site. However, with respect to the integrated downscaling and disaggregation of rainfall data – spatial and temporal – at multiple sites that is required for hydrological modelling, there is limited research (Mezghani & Hingray, 2009; Lu & Qin, 2014).

Many models, *inter alia*, Wilks (1998), Burton *et al.* (2008) and Jennings *et al.* (2010), have developed stochastic weather generators capable of multi-site rainfall generation. Lu & Qin (2013) however note that most of the models were unable to deal with disaggregation at the same time. Segond *et al.* (2006), as well as Mezghani & Hingray (2009) both presented approaches that used multiple models for multi-site disaggregation. Koutsoyiannis *et al.* (2003) presented a model, MuDRain, that is capable of multi-site disaggregation. Lu & Qin (2013) describe MuDRain (Koutsoyiannis, 2003) as a 'viable attempt' at multi-site spatial and temporal disaggregation. MuDRain '*is a methodology* [model / tool] *for spatial-temporal disaggregation of rainfall. It involves the combination of several univariate and multivariate rainfall models operating at different time scales in a disaggregation framework that can appropriately modify outputs of finer time scale models so as to become consistent with given coarser time scale series'* (Fytilas, 2002). MuDRain has been used in a number of studies and has outperformed alternative approaches and models (Debele *et al.*, 2007; Safeeq & Fares, 2011; Lu & Qin, 2014). It is worth noting that, unlike the models for single-site disaggregation, none of the models identified for multisite disaggregation were developed to disaggregate data into finer time steps than one hour. The use of MuDRain in this thesis is further discussed in Chapter 4.

2.6.4.5 Climate change

'*Human interference with the climate system is occurring, and climate change poses risks for human and natural systems'* (IPCC, 2014b). The RSA Government accepts the conclusions of the Intergovernmental Panel on Climate Change (IPCC) that warming of the climate system is indisputable (RSA, 2010). Climate change has the potential to have significant impacts on the environment, local and global economics, and human welfare (Turpie *et al.*, 2002; Schulze *et al.*, 2005; Mukheibir, 2008; RSA, 2011a, 2011b; IPCC, 2014b). Within urban areas, it is generally predicted that the increase in global temperatures will be exacerbated as a result of the urban heat island effect (IPCC, 2014a). Willems *et al.* (2012) indicate that rainfall intensities are typically expected to increase at small urban hydrology scales ranging from 10% to 60% by 2100, from historic levels recorded from 1961 to 1990. Such changes in temperature and rainfall are expected to impact urban drainage (Willems *et al.*, 2012) and have the potential to affect the viability of RWH and SWH. Therefore, where possible, climate change should be considered when the performance of RWH and SWH systems are evaluated.

Mason *et al.* (1999) have shown that there are already signs that there have been changes in extreme rainfall events in the RSA. Mason *et al.* (1999) indicate that the intensity of the 10-year high rainfall events has generally increased in the RSA by over 10%.Schulze (2005a) undertook a study entitled *Climate Change and Water Resources, Southern Africa Studies on Scenarios, Impacts, Vulnerabilities and Adaptation.* The predictions of this study that are relevant to this research include (Hewitson *et al.*, 2005; Schulze, 2005a), *inter alia*:

- A warming of surface and air temperatures. Inland regions are expected to warm more than coastal regions.
- There will be an increase in precipitation in some regions, but a shorter winter season in the southwest, a slight increase in intensity of precipitation, and drying in the west of southern Africa.
- A general decrease in winter rainfall over the typical winter rainfall regions of South Africa – such as in the Western Cape, the province in which the CoCT is situated.
- For coastal regions subject to significant orographic precipitation, the seasonal totals may be expected to remain relatively stable or increase.

The above predictions could have significant impacts on the viability of RWH and SWH including, inter alia, the following:

- An increase in temperature is likely to result in increase evaporative losses for open storage systems such as SWH ponds.
- While an increase in precipitation could increase the potential yield for both RWH and SWH. The inverse would apply for a decrease in rainfall.
- The shorter winter (in this case rainfall) season would mean that RWH/SWH systems would likely require larger storages in order to meet the same demand (or benefit from the increased rainfall, as the current climate would allow for a smaller storage to be refilled more often.

Climate change is further expected to increase the risk of fire and reduce the available water resources in the Western Cape (OneWorld Sustainable Investments, 2008). In a recent stormwater master planning report for the CoCT, Morris *et al.* (2012) increased the modelled rainfall depth for design storms by 15% to account for changes in the intensity of extreme events. This was based on an analysis of the potential impact that climate change might have on rainfall intensities in Cape Town.

2.6.5 Residential water demand

In the RSA, the provision of water services is generally guided by the so-called *Red Book* (CSIR, 2005b), which was initially published as the *Blue Book* (DCD, 1983) in 1983 (van Zyl *et al.*, 2008). The design guidance for municipal water-demand estimation provided in these versions is the same (Husselmann & van Zyl, 2006; van Zyl *et al.*, 2008). In other words, the estimation of water demand for design purposes is based on guidelines dating from prior to

1983. Van Zyl *et al.* (2008) showed that these guidelines are in need of revision, with only 53% of the suburbs they studied fitting within the current guidance. Therefore, for a study such as this it will be necessary to assess the 'actual' water demand within the suburbs and not revert to the standard guidelines used in the RSA.

There are many factors that influence how water is used in a household and why such variations are present. These include, *inter alia*, household size, household income, climate, available technology, seasons, day of the week, etc. (Roberts, 2005; Heinrich, 2006). These factors are not considered in the *Red Book* (CSIR, 2005b). The estimation and simulation of water demand is of critical importance in the modelling of RWH and SWH systems since it determines the amount of water that is needed for various end uses and feeds into the design of a RWH/SWH scheme. It is also important as it affects the volume of water in storage. The quicker the water in storage is used, the more storage volume there will be available to attenuate the runoff from the next storm. For a study of this nature where the viability of RWH and SWH is being considered under a range of scenarios, including the scenarios for a range of different end uses for harvested water, it is critically important to understand what factors affect how and when water is used. These are discussed in the following sub-sections.

2.6.5.1 Factors affecting water use

Many studies from across the world have considered how water is used within a household. A large number originate from Australia (e.g. Loh & Coghlan, 2003; Roberts, 2005; Water Corporation, 2009; Beal & Stewart, 2011) and the United States of America (USA) (e.g. Mayer *et al.*, 1999); the most comprehensive of which collected detailed end-use data from between 100 (Australia) and 1,200 (USA) households. Other studies have been conducted in Canada, New Zealand and South Africa (Heinrich, 2006; Jacobs, 2007; Willis *et al.*, 2011).

As already noted, the amount of water used and what it is used for varies from house to house. Table 2-17 highlights some of the most pertinent findings from the studies mentioned above. It is also evident that the accurate modelling of end-use water demand requires an extensive amount of background data and knowledge about how water is being used in the area that is being modelled.

Table 2-17: Factors affecting water demand (After Heinrich, 2006)

Variable	Description
Household size	'There is a very strong relationship between the volume used for clothes washing and the household size with economies of scale occurring for this end use as households get larger' (Roberts, 2005)
	'*Household size is shown to be an important indicator of water use for toilet flushing'* (Mayer *et al.*, 1999)
	'*The volume of in-house usage is heavily dependent on household size'* (Loh & Coghlan, 2003)
	Larger households are typically more water efficient on a per-capita basis, but still have a higher total water demand (Beal & Stewart, 2011)

Table 2-17 (continued)**: Factors affecting water demand**

Variable	Description
Age of people in each household	The presence of children under twelve is a highly significant factor in the frequency with which showers are taken (Roberts, 2005)
	Children and teens used incrementally more water than adults (Mayer *et al.*, 1999)
	Water demand for clothes washing increases with the number of teenagers in a household (Mayer *et al.*, 1999)
Job / Type of work	Tap and toilet usage decrease in line with the number of people who work full time outside of the home (Mayer *et al.*, 1999)
Garden size	Garden size typically increases outdoor water demand; however, water restrictions can be successful in reducing the relative demand (Beal & Stewart, 2011)
Watering behaviour	'*If your garden gets watered twice a week, you use more water than someone who just waters once a week'* (Heinrich, 2006)
	'*Manual or automatic sprinkler systems were the main methodology for 29% of households but accounted for 52% of irrigation volume. Conversely, those homes for which the handheld hose is the main method make up 57% of homes but only 43% of the total irrigation volume*' (Roberts, 2005)
	'*Sprinkling demands exhibit much greater within-day variation than domestic demand'* (Howe & Linaweaver, 1967)
Climate and data	'*The higher the amount of rainfall, and the lower the temperatures, the less water is being used for irrigation, which is a major end use*' (Heinrich, 2006)
	Roberts (2005) reported summer water demand (784 l/day) was roughly 50% higher than the average winter water demand (511 l/day)
	Loh and Coghlan (2003) reported seasonal increases in outdoor demand
	Irrigation is concentrated in the summer months (Howe & Linaweaver, 1967)
Type of property	'*On average, people who lived in single residential houses used 8 kilolitres per person [per year] more than those living in multi-residential properties*' (Water Corporation, 2009)
Household income	Household income had a significant effect on ex-house water usage (Loh & Coghlan, 2003)
	'*For clothes washer use, some trends were found between increasing household income and higher clothes washing consumption, which would be expected given that many of the higher income families were also larger and had higher numbers of children'* (Beal & Stewart, 2011)
	Most end uses show a slight increase with income, although unknown water use decreases (Mayer *et al.*, 1999)

2.6.5.2 Diurnal and seasonal water use

The factors in Table 2-13 not only impact on total water demand, but also on when the demand is realised during the day. The result of these factors is that the instantaneous water demand within households varies continuously from day to day, and between seasons (Mayer *et al.*, 1999; Loh & Coghlan, 2003; Roberts, 2005).

Figure 2-17 shows the average hourly use pattern from 1200 households spread across twelve sites in the USA (Mayer *et al.*, 1999). Both the indoor and outdoor demands peaks in the morning and late afternoon / evening. In this particular study, the outdoor demand proved to be a significant component of the total water demand.

Figure 2-18 shows the average hourly demand as a percentage of the total water demand from 100 households in Australia (Roberts, 2005). The summer demand is roughly 1.5 times the winter demand. It is interesting to note the significant afternoon peak in summer. Roberts (2005) notes that, in winter, the morning and afternoon peaks are driven by shower use, but in summer, the afternoon peak is driven by outdoor use – irrigation. An aspect that has seemingly not received attention in any of the studies is the impact of holidays and weekends on total water demand and the diurnal pattern. This is important because, as an example, university residences will likely have significantly reduced water demand due to lower occupancy levels during these times. This will not only impact on the volume of harvested water used to meet demand, but will affect the volume available to attenuate storage. Where rainfall seasons and holidays coincide, it could result in reduced stormwater management benefits. Where possible it is important to consider, or at least recognise, the effects that holidays or weekends could have on water demand.

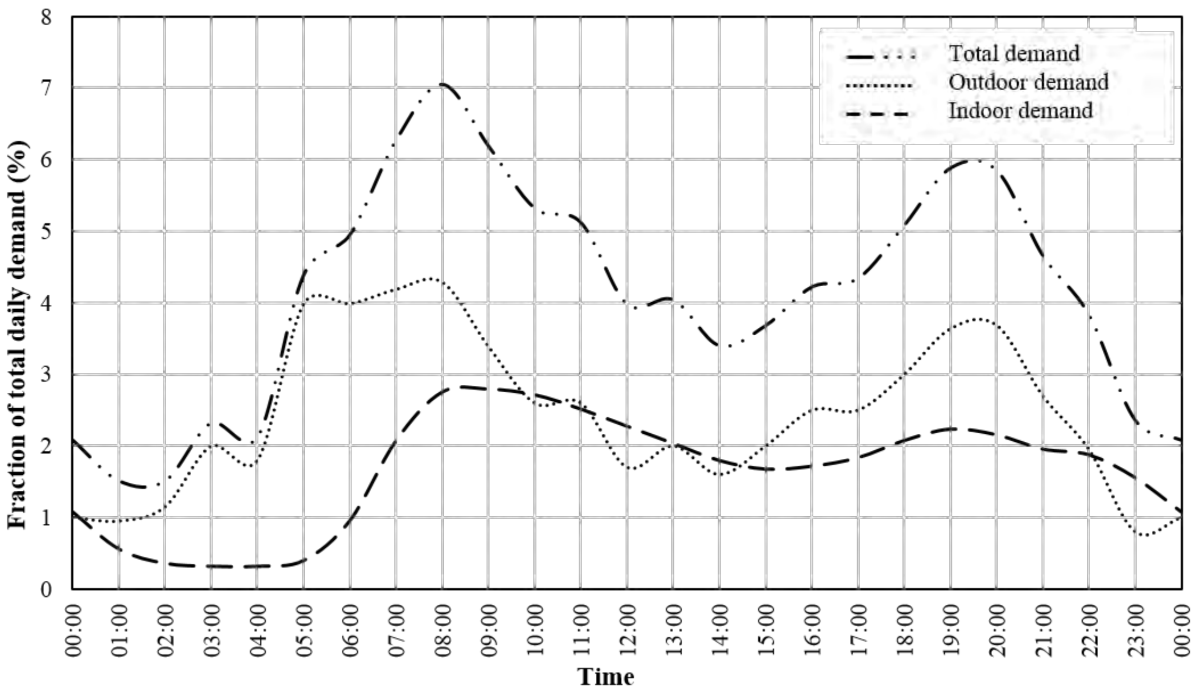

Figure 2-17: Residential diurnal – total, indoor and outdoor – water demand patterns (Mayer *et al.*, 1999)

As with rainfall data, the spatial and temporal detail of the water demand data is important. Section 2.6.3 discussed the challenges of modelling RWH and SWH storage units, and the two approaches (YAS or YBS) that can be used. The accuracy of whichever approach is selected will be affected by a number of factors including, *inter alia*, the resolution (i.e. high resolution where each property's demand is either recorded or stochastically modelled based on unique characteristics, or low resolution where an average water demand is used), and the spatial and temporal representivity of the water demand data. For example, Coombes & Barry (2012) showed that the use of averaged water demand in the analysis of Sydney's water resources led to errors in the results of analyses, and therefore they suggested that studies should make use of spatially and temporally explicit methods of systems analysis to avoid failing to realise the full potential (positive or negative) of alternative water management approaches such as RWH and SWH. These concerns are not new; for example Buchberger & Wu's (1995) models of municipal distribution systems made use of spatial and temporal averages. They found that, while at a distribution system scale it may not make a large impact on the results, at the local scale this approach may mask the actual behaviour of the local flow regime. Thus it is evident that the use of averaged data, even if spatially and temporally representative, may impact on the results of an analysis; and the impact will be more notable at a smaller scale. Duncan & Mitchell (2008), however, note that many models have made use of average water demand at a daily time step along with daily climate data.

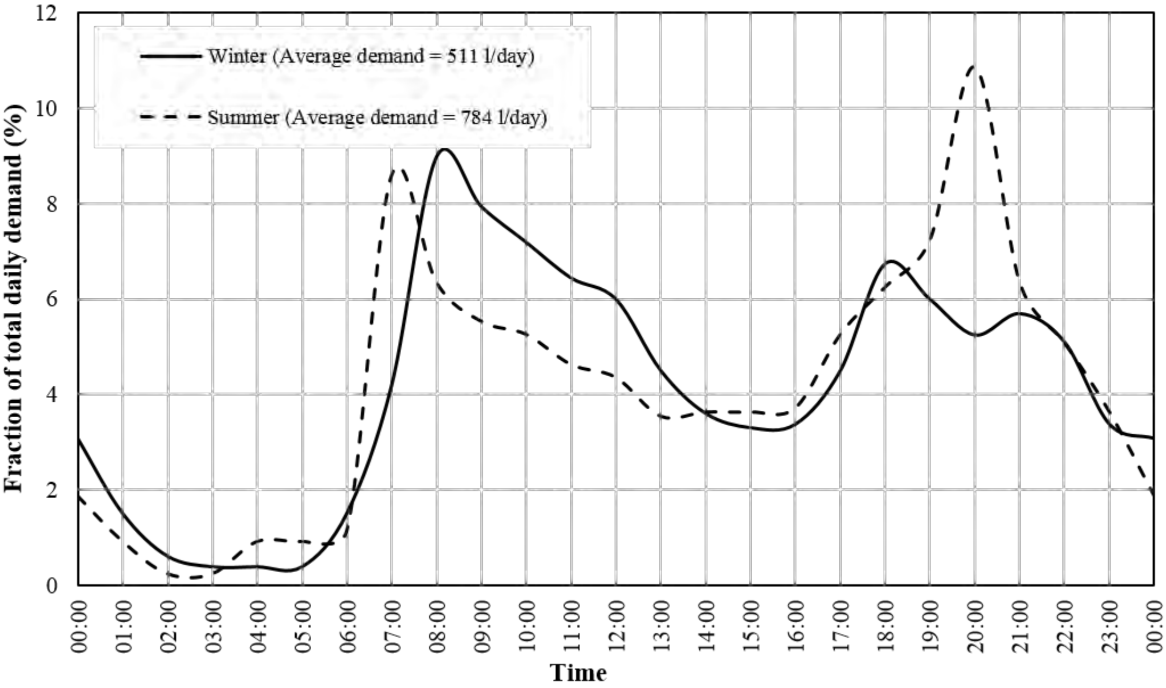

Figure 2-18: Seasonal diurnal water demand pattern (After Roberts, 2005)

Duncan & Mitchell (2008) note that in some cases, a diurnal water demand pattern has been superimposed upon the daily demand. However, for the analysis of alternative water use and reuse scenarios at the household scale, they felt that the representation of a diurnal water demand pattern may require more detailed demand data. Such data is not readily available as a result of monitoring. To overcome the lack of monitoring and shortage of data relating to diurnal water use, Buchberger & Wu (1995), Duncan & Mitchell (2008), and Blokker *et al.* (2010) used a stochastic approach to simulate water demand. These models have been applied in subsequent studies across the USA and Australia. They typically represent water demand as a ‘pulse’ (period of flow) based on: the probability of an event (e.g. flushing a toilet) happening at a specific point in time, the duration of use, and the volume of water typically used during such an event. These models can then be used to stochastically generate water demand patterns for properties with different descriptive characteristics (e.g. household size, garden size, pool, etc.). The stochastically generated patterns will also result in houses with similar characteristics having different water demand patterns that are more representative of a real-case environment.

Water demand simulation models typically require high resolution end-use demand data as an input (Buchberger & Wu, 1995). Blokker *et al.* (2010) note that obtaining the required data for input (or calibration) is expensive and results in a descriptive model that is not easily transferable outside of the place where the data is collected. As an alternative, they proposed a predictive rather than descriptive stochastic end-use model that made use of information relating to different appliances and residential end users. Blokker *et al.* (2010) claim that because their stochastic *‘end-use model is based on statistical information rather than flow measurements, the model is transferable to diverse residential areas in different countries’*. This claim has not been proven, however, and while their modelled results show a good correspondence to measured water demands, the statistical input data required for modelling is detailed and the model was only tested against measured data collected in the same region in which the model was developed. Therefore, the transferability of the model has not been established. Duncan & Mitchell (2008) use a similar approach that has been modified in subsequent studies; for example, Maheepala *et al.* (2013). It seems unreasonable not to calibrate any model, at least to check, when using it in a new location. This poses a problem in the RSA where data for calibration is limited.

Jacobs & Haarhoff (2004) developed the Residential End Use Model (REUM), an end use model that considered a large number of input parameters such as: indoor toilet flushing, bathing and showering, garden watering, pool water use, leaks, etc. Additionally the REUM model makes use of estimated evapotranspiration to infer outdoor irrigation and pool demand. Du Plessis & Jacobs (2014), further developed and the model to include an irrigation efficiency factor that accounts for the difference between theoretical and actual water demand. One potential problem with the use of end-use models, such as Jacobs & Haarhoff (2004), for estimating outdoor demand based on evapotranspiration as a measure of outdoor water demand for irrigation is highlighted in Mayer *et al.* (1999) who found that 22 percent of properties used less than 10% of the theoretical requirement while 17 percent used more than 100%.

2.6.6 Economic assessment of alternatives

Many methods are available for economic appraisal in the water sector (Roebuck, 2007). Section 2.2 highlighted that it is necessary where possible, to consider more than the direct costs of a system – such as the potential environmental benefits – in order to understand the true cost or benefit associated with a system. Philp *et al.* (2008) identify five economic evaluation frameworks to economically assess RWH and SWH options. These are: (DECNSW, 2006; Roebuck, 2007; Philp *et al.*, 2008; Dobes & Bennett, 2009):

- **Benefit Cost Analysis (BCA):** quantifies all the major costs and benefits of each option in monetary terms. BCA determines whether an individual / society would be better or worse off overall, as a result of implementing one or more alternatives.
- **Cost Effectiveness Analysis (CEA):** assumes that all benefits can be aggregated into a single attribute (e.g. kilolitres of water). Cost is then considered in relation to this attribute. CEA is only useful where the benefits are largely the same in nature.
- **Cost Utility Analysis (CUA):** the costs of an option are considered in relation to the benefits, which are expressed as a utility (benefit).
- **Multiple Criteria Analysis (MCA):** evaluates proposals against a set of predetermined criteria. While cost can be a criterion, it is not necessarily required.
- **Triple Bottom Line Analysis (TBL):** essentially an MCA, with a focus on the equal consideration of environmental, social and economic elements.

Hajkowicz (2006) presented a useful framework, summarised in Figure 2-19, for selecting an appropriate economic evaluation method. The BCA is considered a more comprehensive technique than CEA and '*is normally the preferred technique wherever feasible'* (DECNSW, 2006). A difficulty with the BCA analysis for RWH/SWH though, is that the benefits may be difficult to quantify – if possible at all. Where neither BCA nor CEA is appropriate, a CUA or MCA may be considered. The CUA can be used on its own or as an extension of the MCA (Philp *et al.*, 2008). Philp *et al.* (2008) note that the use of MCA has had a positive reception in Australia and has been 'repeatedly' suggested for the evaluation of stormwater harvesting options. Dobes & Bennett (2009) agree that it has become popular in Australia, but argue that the method is 'fundamentally flawed' and is used as an easy 'short cut' around the fundamental complexities of benefit-cost analysis. Dobes & Bennett (2009) consider the TBL to be a simplistic MCA, suggesting that it has no underlying principles or methodology and '*is superfluous in a cost-benefit analysis undertaken from a national social perspective by a government agency*'. Each analysis method requires different amounts and types of data; varies in its complexity; and has its own strengths and weaknesses. As with modelling RWH and SWH systems (Section 2.6.2), the selection of the method will need to consider the availability of data and the complexity of the method in relation to the accuracy of the results required. The economic analysis methods used in this thesis are discussed in Chapter 4.

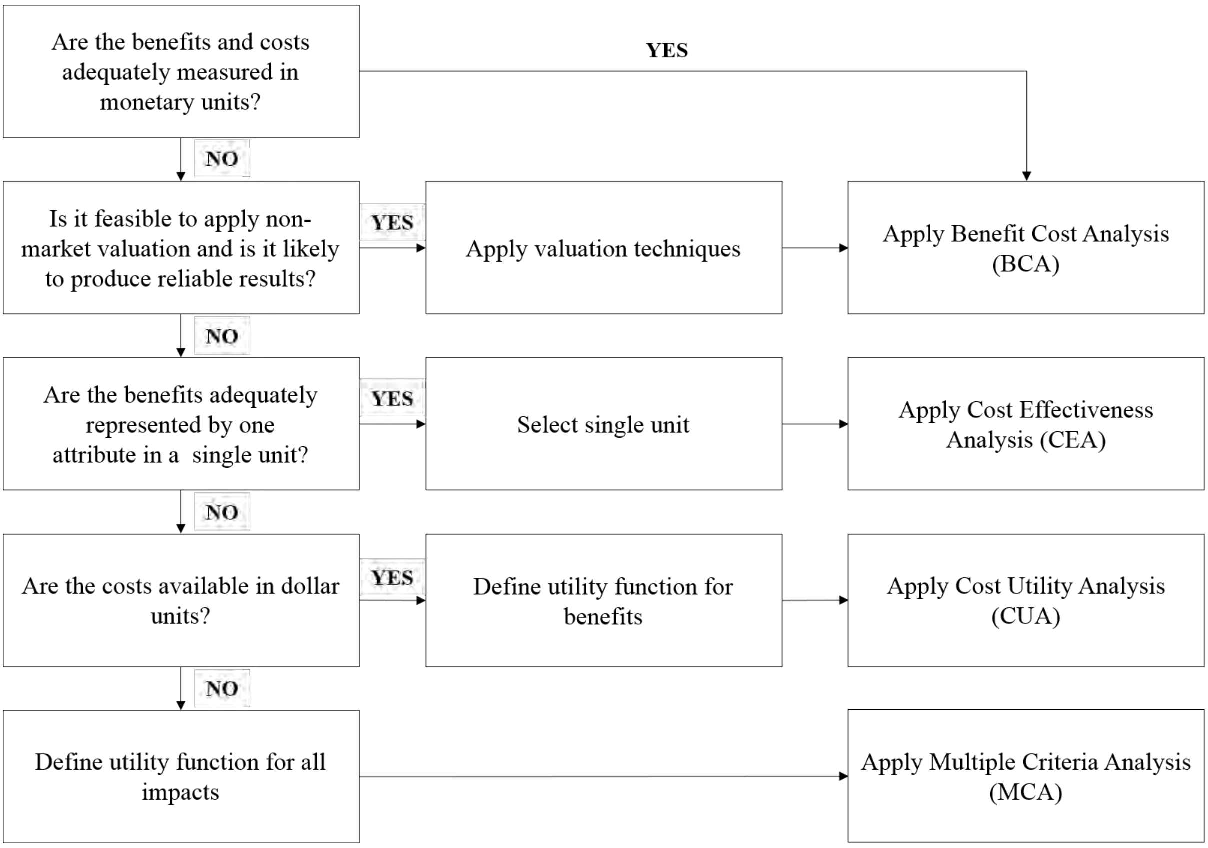

Figure 2-19: Selecting an economic evaluation method (Hajkowicz, 2006)

2.6.6.1 Life-cycle Cost Analysis

Life-cycle Costing (LCC) is '*the systematic consideration of all relevant costs and revenues associated with the acquisition and ownership of an asset'* (Clift & Bourke, 1999). LCC may be used as part of any of the above approaches in order to better understand the costs, and in some cases, the benefits of a system over its entire life cycle. LCC essentially considers all the costs associated with an asset. This would include design, construction, establishment of vegetation (where relevant), maintenance (inspections, regular, irregular and corrective), disposal and land costs (Lampe *et al.*, 2005; DECNSW, 2006; Roebuck, 2007; Philp *et al.*, 2008). While DECNSW (2006) and Philp *et al.* (2008) consider LCC an approach for obtaining the 'cost' input for an economic analysis (BCA / CEA / CUA), Lampe *et al.* (2005) suggest, as shown in Figure 2-20, that LCC can also be used as an economic tool that considers benefits as well. The costs and any benefits are discounted to their present value, which is then used to assess the viability / cost-effectiveness of an option.

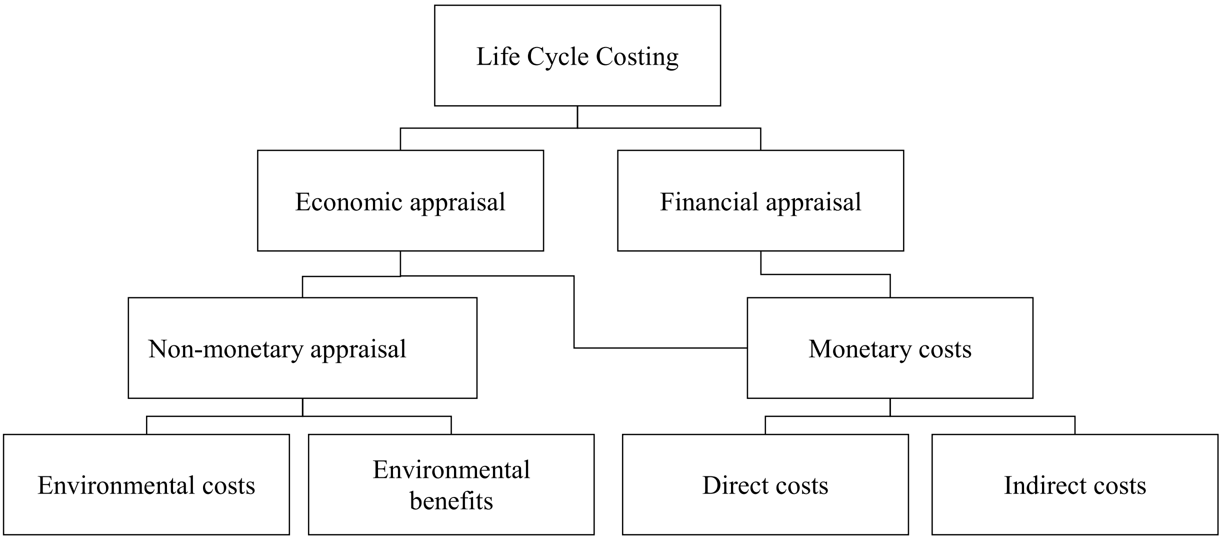

Figure 2-20: Approaches to Life-cycle Costing (After Lampe *et al.*, 2005)

2.6.6.2 Valuation of benefits and impacts

While direct costs are typically already monetised, a significant challenge for economic analyses such as BCA or economic LCC, is the monetisation of benefits. One important category of benefits referred to as 'environmental goods and services' (EGS) can be defined as the flows of benefits derived from the environment (de Wit *et al.*, 2009), including the water quality treatment offered by wetlands, or the amenity value offered by a reservoir. Other benefits could include, amongst others, the reduction in potable water demand; while impacts could include, for example, the breeding of mosquitoes. The valuation of EGS, other benefits and impacts is a specialised field. TEEB (2010) provides an up-to-date overview of the latest frameworks for evaluating the Total Economic Value (TEV) of ecosystems by accounting for the full range of disparit benefits that the environment may offer. This section, however, provides a brief overview of how individual benefits may be valued to allow for inclusion in a BCA or LCC of an RWH/SWH system. While it is possible to identify the benefits (externalities), attaching an economic value to them is not simple (Lampe *et al.*, 2005). Many techniques are available for valuing benefits and impacts (Table 2-18).

Lampe *et al.* (2005) suggest that the most popular method of estimating values for environmental characteristics is the Contingent Valuation Method (CVM). The CVM considers the public's willingness to pay for a change in the quality or quantity of an environmental good or service. This will be area specific (Lampe *et al.*, 2005) and requires an investment in social surveys.

Cost of Replacement (CoR) is a method that determines the cost of relocating / developing an equivalent site elsewhere. In the case of environmental cost, 'compensation' is not always made, nor is it given to compensating the environment. If it were, there would be no environmental damage or costs (Bowers, 1998). With reference to stormwater management, Smit *et al.* (2002) state: '*Open space and catchment managers could use this methodology to compare the annual management and operation costs with the replacement cost and value of* *these systems. Furthermore, this type of data could be used to motivate for resources to ensure that future engineering solutions will not be required to replace degraded natural systems no longer able to provide the required services to the CCT'.*

It is possible to use multiple techniques, either to compare outcomes or to complement each other. The technique or techniques selected need to consider the data constraints (historical and current), experience, time and personnel.

Table 2-18: Economic valuation techniques (Pagiola *et al.*, 2004)

	Method	**Approach**	**Use**
Revealed preference method	Production function (also known as 'change in productivity')	Traces impact of change in ecosystem services on produced goods	Any impact that affects produced goods
	Cost of illness, human capital	Traces impact of change in ecosystem services on morbidity and mortality	Any impact that affects health (e.g. air pollution)
	Substitute / replacement cost	Uses cost of replacing the lost goods or services	Any loss of goods or services
	Travel cost (TCM)	Derives demand curve from data on actual travel costs	Valuing recreational activities
	Hedonic pricing	Extracts effect of environmental factors on price of goods that include those factors	Air quality, scenic beauty, cultural benefits
Stated preference methods	Contingent valuation (CV)	Asks respondents directly about their willingness to pay for a specified service	Any service
	Choice modelling	Asks respondents to choose their preferred option from a set of alternatives with particular attributes	Any service
Other	Benefits transfer	Uses results obtained in one context in a different context	Any for which suitable comparison are available
	Expert judgment	Based on the experience of the expert	Any service

2.6.7 Assessing performance

Table 2-19 highlights many of the variables that have been used to evaluate the performance of systems. The selection of which variables are used depends on what the focus of the design or analysis is. The variables of interest in this study relate to assessing the reduction in runoff and the reliability of the system.

Table 2-19: Criteria used to assess the performance of RWH systems
(After DeBusk & Hunt, 2014)

Reference	Criteria used to assess performance
Basinger *et al.* (2010); Imteaz *et al.* (2011); Imteaz *et al.* (2012)	$\text{Reliability} = \dfrac{\text{number of days tank volume is sufficient to meet demand}}{\text{number of days in evaluation period}}$
Briggs & Reidy (2010)	Water savings: • percentage of total demand met • volume of potable water saved Runoff reduction: • percentage of total precipitation captured by the RWH system • volume of runoff captured and used (versus leaving the system via overflow) Reliability: percentage of individual demands fully met by the RWH system
Wanielista *et al.* (1991) Farreny *et al.* (2011)	$\text{Efficiency} = \dfrac{\text{Amount of rainwater / stormwater harvested}}{\text{Total volume of rainwater / stormwater that could've been harvested}}$
Fewkes & Butler (2000); Fewkes & Warm (2000); Roebuck (2007); Roebuck *et al.* (2012); Palla *et al.* (2011); Palla *et al.* (2012)	$\text{Water-saving efficiency (WSE)} = \dfrac{\text{rainwater yield from RWH system}}{\text{total demand}}$
Thomas (2002); Liaw & Tsai (2004); Mitchell *et al.* (2008b); Zhang *et al.* (2009); Neumann *et al.* (2011)	$\text{Volumetric reliability(VR)} = \dfrac{\text{volume of rainwater supplied}}{\text{total demand during evaluation period}}$
Jones & Hunt (2010)	• Usage replaced: See Water Savings Efficiency (WSE) • Annual water savings: average of monetary savings from using rainwater to replace public water supply • Overflow frequency: percentage of precipitation events that created overflow from the system • Dry cistern frequency: percentage of days when demand could not be met with rainwater • Payback period: number of years of system use required for monetary savings to equal the cost of the system
Palla *et al.* (2012)	Median Detention Time

Wanielista *et al.* (1991) and Farreny *et al.* (2011) considered 'efficiency' to be a measure of the amount of runoff harvested as a ratio to the total runoff. This 'efficiency' is essentially a measure of the reduction of runoff. Jones & Hunt (2010), on the other hand used the frequency at which the RWH system overflowed as a measure of the system's performance.

Two definitions of reliability exist. The first focuses on the number of days that the system is able to supply water for use as a ratio of the total number of days in the analysis period – see Basinger *et al.* (2010), Imteaz *et al.* (2011) and Imteaz *et al.* (2012). The second, known as the volumetric reliability, is a measure of the water supplied as a ratio of the total water demand – see Thomas (2002), Liaw & Tsai (2004), Mitchell *et al.* (2008b), Zhang *et al.* (2009) and Neumann *et al.* (2011).

2.6.8 Modelling RWH and SWH together

An extensive body of research exists regarding the design, implementation, operation, risks, costs and potential benefits of rainwater and stormwater harvesting systems. However, Akram *et al.* (2014) note that while there is a significant amount of research regarding stormwater harvesting, most of it focuses on a single sub-system (e.g. collection, storage, treatment, or distribution). Fewkes (2006), Roebuck (2007) and DeBusk & Hunt (2014) provide an overview of the research focused on RWH systems, while Goonrey (2005), Philp *et al.* (2008) and Akram *et al.* (2014) provide an overview of the research focused on SWH systems. There are, surprisingly, few studies that consider RWH and SWH in combination or as alternatives. The study of Mitchell *et al.* (2005) is one exception. Mitchell *et al.* (2005) made use of the modelling software *Aquacycle* (Mitchell, 2004) and found that centralised SWH outperformed RWH – from a cost per kilolitre perspective. However, Aquacycle does not account for the spatial and temporal impacts of linearly upscaling the effects of RWH/SWH, which can lead to significant overestimations of the volumetric reliability of RWH systems (Mitchell *et al.*, 2008a; Xu *et al.*, 2010; Coultas *et al.*, 2011; Maheepala *et al.*, 2011, 2013; Mashford *et al.*, 2011; Neumann *et al.*, 2011) – see Section 2.4.5. Neumann & Maheepala (2013) note that, '*based on the studies of rainwater tanks reported in the literature, it can be expected that the use of average values of input variables to represent the combined system could also introduce errors*'. There is, however, a lack of research considering the catchment scale impacts of RWH and SWH – individually and/or together – on peak event flows that did not make use of linear extrapolation to upscale the effects of RWH and SWH. Akram *et al.* (2014) therefore suggest that there is still a need for integrated models and modelling.

2.6.9 Stormwater modelling: continuous vs. event models

An important aspect of this research is to assess the potential benefits of RWH and SWH. An often-cited benefit of RWH/SWH is the attenuation of peak flows and the mitigation of flooding. To assess this, it is necessary to model the stormwater system as a whole. Traditionally, stormwater modelling is undertaken using a single storm event or design storm, but according to James (2005), this is no longer appropriate. Newton & Walton (2000) showed that the use of continuous simulation identified that the design discharges for smaller events were being underestimated by event-based methods. Tan *et al.* (2008) showed that using an event-based calibration was better for reproducing the overall shape of a hydrograph, peak flow and time to peak, but continuous-event calibration was better for providing runoff volume. Therefore, Tan *et al.* (2008) concluded that, where runoff volume is the main concern (as it is in stormwater harvesting), continuous data should be used for calibration. Boughton & Droop (2003) suggest that the choice of event-based or continuous modelling remains a matter of 'personal preference', but this may be due to event based modelling typically requiring less data and having shorter run times which is often preferred. Clearly, there is still a level of dispute about which approach to use in modelling urban runoff, but one of the shortcomings of event modelling is that the antecedent conditions are not considered (Wanielista *et al.*, 1991). In the case of SWH, this would include the level of water in the pond, which could have an impact on the volume detained. In light of Section 2.6.2, whether or not to use a continuous model depends mainly on the purpose of the model. This decision must consider the advantages and disadvantages of event-based and continuous modelling.

2.7 Summary and need for further research

The RSA faces a range of challenges with regard to the management of water, not the least of which is water scarcity. The potential exists for climate change to further complicate these challenges by decreasing the availability of water while simultaneously increasing demand. The RSA is not alone in facing these challenges and, in response to these and other challenges, there has been a paradigm shift internationally to manage water more holistically. While the terminology might vary from decade to decade, country to country, or even at different management scales, it is apparent that there is a move towards a new paradigm in water management, one that recognises the value of water in all its competing uses. As a result of this paradigm shift, there has been a growing interest in rainwater harvesting (RWH) and stormwater harvesting (SWH).

The literature has also demonstrated that, while there has been a significant amount of research internationally focused on RWH at a site scale, there has been limited consideration of the regional scale impacts (positive or negative) of RWH in urban areas for residential use. Where the regional scale impacts have been considered, it has been done in a simplistic manner, which has subsequently been shown to be unreliable. Within the RSA, there has been relatively little notable research into the impacts, whether positive or negative, of urban domestic RWH in South Africa. Jacobs *et al.* (2011), for example, showed that RWH for garden irrigation in the Western Cape is not viable due to the climate (winter rainfall), but did not consider alternative uses such as toilet flushing. While Mwenge Kahinda (2010) did consider the regional impacts of RWH, the methods employed were not only simplistic, but also not based on data representative of the urban development or demand. There are no studies in RSA that have considered the costs, stormwater management impacts, or water demand benefits of RWH. Internationally, there is little research on the stormwater management benefits of RWH, as water conservation is often the primary goal of implementation (DeBusk & Hunt, 2014).

SWH is part of a rapidly developing field internationally. While the RSA has experience with SWH in the form of the Atlantis Water Resource Management Scheme, it is an isolated example that started off as an interim solution while a more conventional pipeline was developed (DWAF, 2010). SWH is not considered or included in water management planning in the RSA.

One of the major barriers to the widespread implementation of SWH is the paucity of reliable and affordable treatment technologies (Hatt *et al.*, 2004b; Philp *et al.*, 2008). This is a challenge to the long-term success of SWH as individuals may be unaware of the risks associated with SWH, or how to mitigate them.

Internationally, there is a lack of studies considering the impacts of RWH and SWH in combination. While RWH and SWH have broadly similar benefits, there are distinct differences as well. If roof runoff is managed at site scale, it might result in a reduction of stormwater runoff, consequently compromising the viability of SWH. This has not to date been assessed.

The literature has shown that access to data is important for studies of this nature. It is also clear that many of the most advanced studies are from 'developed' countries such as Australia, where data is available – but may be inappropriate in a 'developing' world situation. The RSA is a developing country where useful data are often not available, so this potentially poses a problem. Data availability allows for more complex models, which in principle should provide more accurate results of the benefits and/or impacts of RWH and SWH. The literature has shown that uncertainty as to the costs and benefits of alternative approaches to water management, such as RWH and SWH, are potentially barriers to their wider acceptance and adoption – both institutionally and socially – due to a suspicion that these approaches are more expensive than the conventional alternatives. There is, therefore, a need for a study that considers the benefits and costs of RWH and SWH, which could be used to motivate for or against the adoption of RWH and/or SWH in the RSA.

3. Case study

The only way to quantify the benefits and potential viability of rainwater and stormwater harvesting – particularily the stormwater management benefits – was to select and model a representative catchment. As a result of data constraints, especially data availability, the method needed to be customised to the selected catchment. The selection of the catchment therefore was critical to this research and preceeded the development of the method of analysis described in Chapter 4. Consequently, this chapter provides an overview of the selected catchment and why it was selected. Section 4.1 provides an overview of the scope and limitations of this study within the context of the selected case study catchment.

A number of catchments in the CoCT were considered including, amongst others, the Salt River (the catchment was considered to be too large), Disa River (the catchment was considered to be too small with insufficient development diversity and poor data availability), Sand River (too many informal settlements and poor data availability), etc. The Liesbeek River Catchment was selected for this study as it incorporates a diversity of land uses, represents a range of wealth levels, has significant historical importance for the CoCT and the RSA, and had the necessary data (see Chapter 4) available for the effective development of the detailed models required for simulating catchment-wide RWH and SWH. While the catchment represents a range of wealth levels, it does not contain any informal settlements / slums typical of many urbanised catchments in the RSA. This is fortunate for the following reasons:

i) The data required for the proposed analysis were not available for informal settlements.

ii) Due to high population densities, poor provision of services and high levels of pollution, the complexities and challenges with regard to the management of risks associated with the use of alternative water sources in a fit for purpose manner within informal settlements are magnified in comparison to formal settlements.

iii) Informal settlements in the RSA are typically associated with extremely poor runoff water quality which would negatively impact on the viability of SWH systems.

One of the biggest challenges in this thesis was dealing with the complexity of an urbanised catchment that has a significant amount of irrelevant 'data', but very limited relevant data to explain the significant variations in social, economic and climatic variations seen in the catchment. These variations have the potential to significantly affect the relative uncertainty of the results. These challenges and limitations are further discussed in Chapter 4.

3.1 History of the Liesbeek River Catchment

The Liesbeek River Catchment, sometimes spelt Liesbeeck, is situated on the eastern slopes of Table Mountain in the City of Cape Town (CoCT) (Figure 3-1) (Evans, 2007; Robinson, 2011). The Liesbeek River was 'discovered' by European settlers on the 28 April 1652. Jan Van Riebeeck – commander of the Dutch settlement – described it as '*the loveliest of fresh rivers*'

(Murray, 2003). Initially, it was named '*Varsche*' and subsequently the '*Soete'* and then the '*Amstel*'. Finally, by 1657, Van Riebeeck had settled on the name '*Liesbeek*'. The Liesbeek River Catchment is approximately 2,600 hectares in extent and is the oldest urbanised river valley in the RSA (Evans, 2007). The river itself is approximately 9 km long and is fed by numerous streams running down the eastern slopes of Table Mountain (Evans, 2007; Brown & Magoba, 2009; Robinson, 2011).

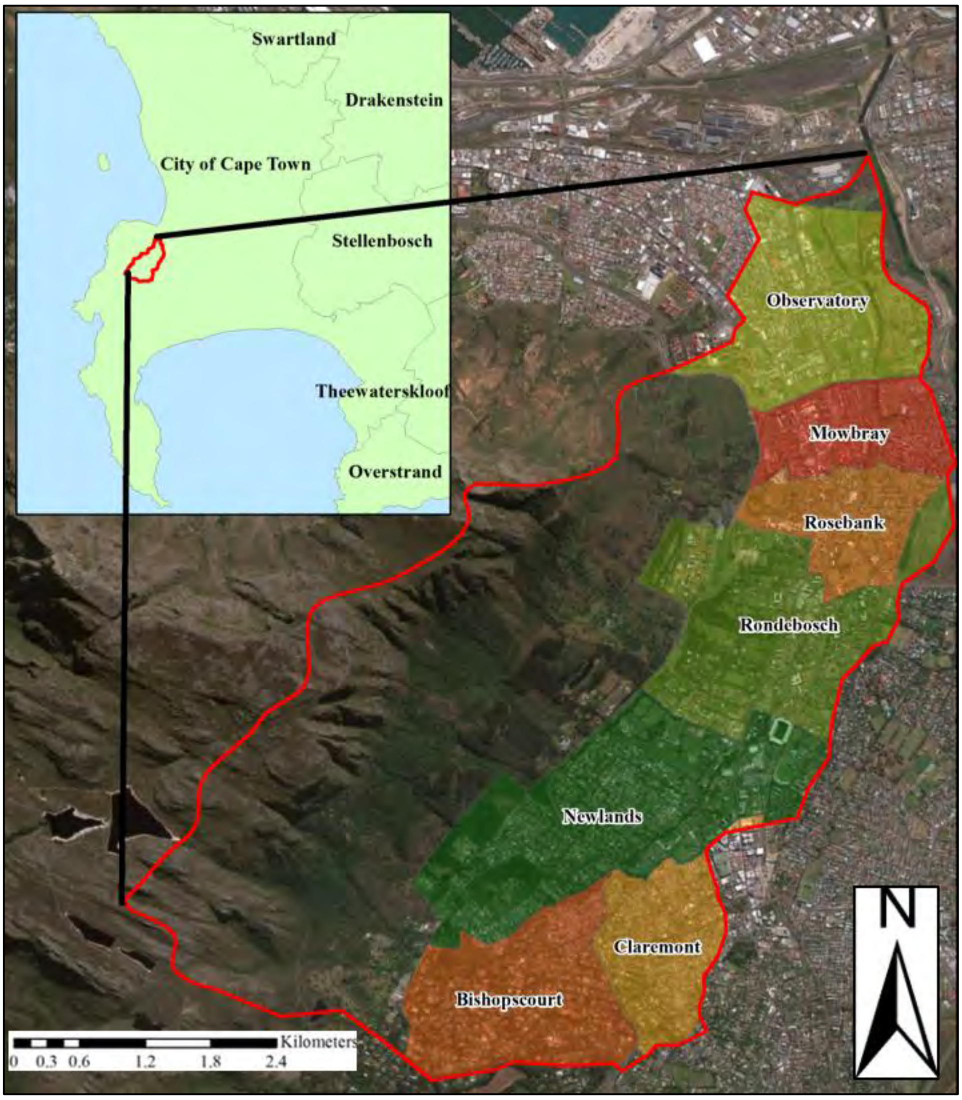

Figure 3-1: Liesbeek River Catchment

When Van Riebeeck began setting up the first European settlement in the Cape (RSA), there was no intention to develop a colony. Instead, he was tasked with setting up a defensible fort, acquiring fresh water, planting fresh produce and bartering with the local inhabitants – Khoi-Khoi – for sheep and cattle. However, as a result of tensions with the Khoi-Khoi and the growing population of settlers, it became difficult to meet demand. A decision was made to expand the settlement (Robinson, 2011). In February 1657, the first real colony was established along the Liesbeek River at Rondebosch by nine of the Company's servants who were discharged from Company service and were allotted parcels of land, approximately 8.6 hectares each (Brown & Magoba, 2009; Robinson, 2011). In this manner, the colonisation of South Africa was begun (Badlam, 2011).

As the settlement grew into a colony and the colony expanded, infrastructure such as railways was put in place. This ultimately led to the draining of the marshland, which disturbed the natural conditions of the surrounding watercourses and led to the creation of an artificial canal on a new route (Murray, 2003). In the first half of the twentieth century, flooding started to become a serious problem in the Liesbeek River Catchment as a result of increasing urbanisation. Consequently, between 1942 and 1962, large portions of the Liesbeek River were canalised.

Currently, the river is highly impacted by urbanisation. In total, approximately 50% of the catchment is urbanised – with the balance taken up by the Kirstenbosch Botanical Gardens, forestry plantations and the Table Mountain National Park. Six of the CoCT's suburbs are either partially or entirely located within the Liesbeek River Catchment (see Figure 3-1) (CoCT, 2009d). The lower reaches of the river have the highest levels of urbanisation within the catchment. Since 1990, there have been many initiatives to re-establish aquatic life and improve the aesthetics of the river (Evans, 2007; Brown & Magoba, 2009). These attempts have largely been localised around the banks of the river and have not targeted the catchment as a whole. While there is evidence of gradual densification in the catchment, in the form of new blocks of flats being constructed in place of former free-standing houses, the catchment as a whole has shown no signs of significant change in the last 14 years, as can be seen in the time series of aerial photographs of the catchment from 2000 to 2014 – the period for which climate, social and technical data are available (Appendix D).

3.2 Rainfall and evaporation

The CoCT has a Mediterranean climate characterised by mild, wet winters and dry, warm summers (Rohli & Vega, 2011). The average rainfall in the CoCT is 515 mm/yr. (WMO, 2014); however, rainfall and evaporation are highly variable across the CoCT owing to the presence of mountainous topography within the City's boundaries. The Liesbeek River Catchment specifically, is affected by the presence of the Peninsula Mountain chain to the west. Within the Liesbeek River Catchment, the maximum annual rainfall (1500 mm/yr.) is more than double the minimum (600 mm/yr.) – see Figure 3-2a. While less significant, evaporation also varies – in this instance, between 1300 mm/yr. and 1550 mm/yr. across the catchment – see Figure 3-2b. This large variation in rainfall and evaporation has the potential to significantly affect the viability of RWH and SWH within the catchment.

3.3 Land use, property value and income in the catchment

The diversity of land uses found in the Liesbeek River Catchment is illustrated in Figure 3-3 (whole catchment) and Figure 3-4 (urbanised part of the catchment) and is expressed as a percentage of area occupied by each. Only 50% of the catchment is effectively urbanised; the other 50% is made up of 'conservation and nature areas' (43%) and 'urban open space' (7%) (CoCT, 2009a). Within the urbanised part of the catchment, the southern end (Bishopscourt) consists almost entirely of general residential suburban households. Throughout the rest of the urbanised part of the catchment, general residential properties are interspersed with blocks of flats, educational institutions and community facilities. Commercial activities are largely focused around Main Road which runs the length of the catchment – see Figure 3-5.

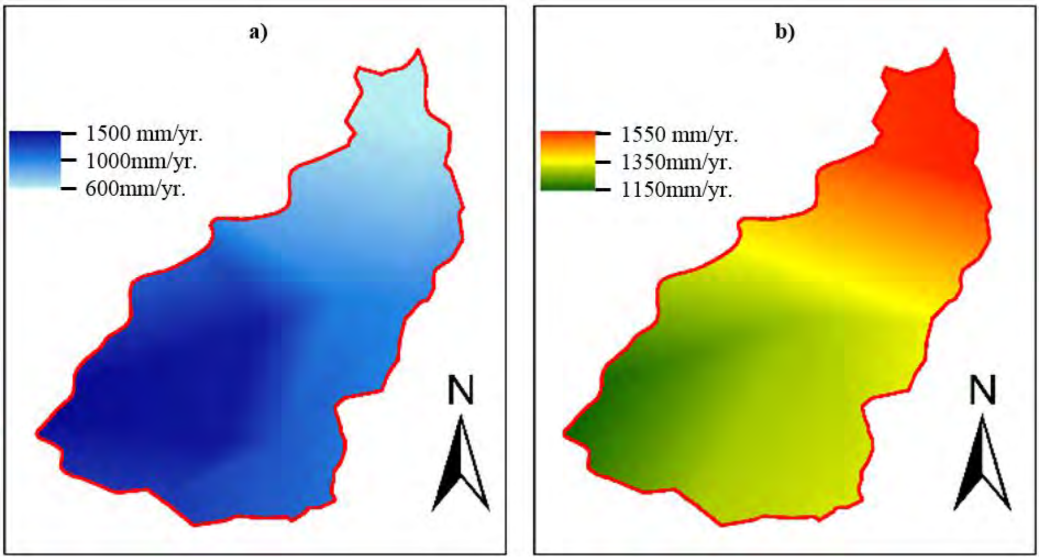

Figure 3-2: a) Annual average precipitation and b) annual average evaporation across the Liesbeek River Catchment

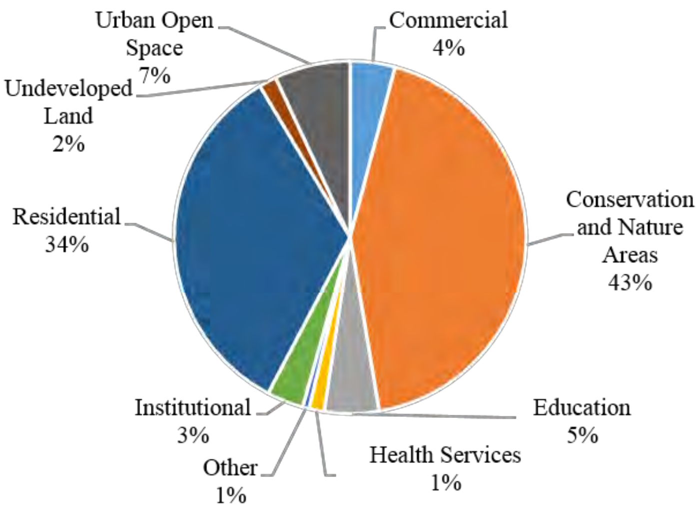

Figure 3-3: Breakdown of land use in the Liesbeek River Catchment overall

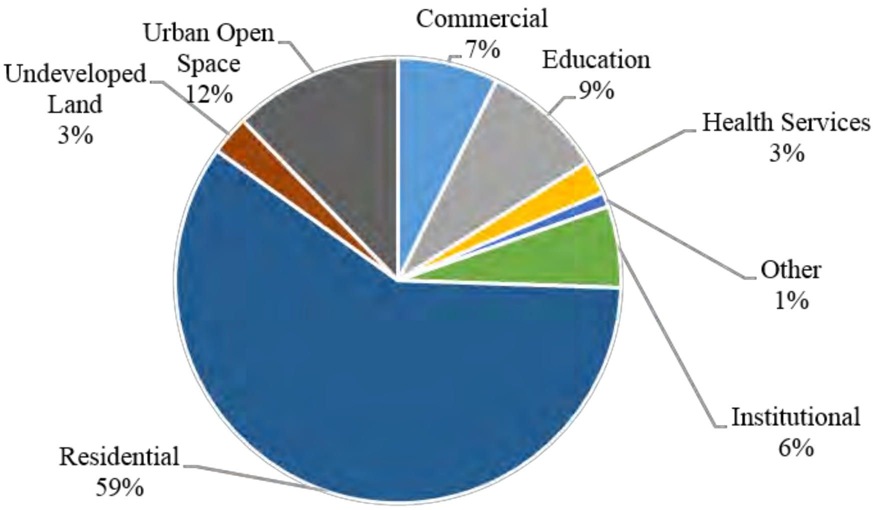

Figure 3-4: Breakdown of land use in the urbanised area of the Liesbeek River Catchment alone

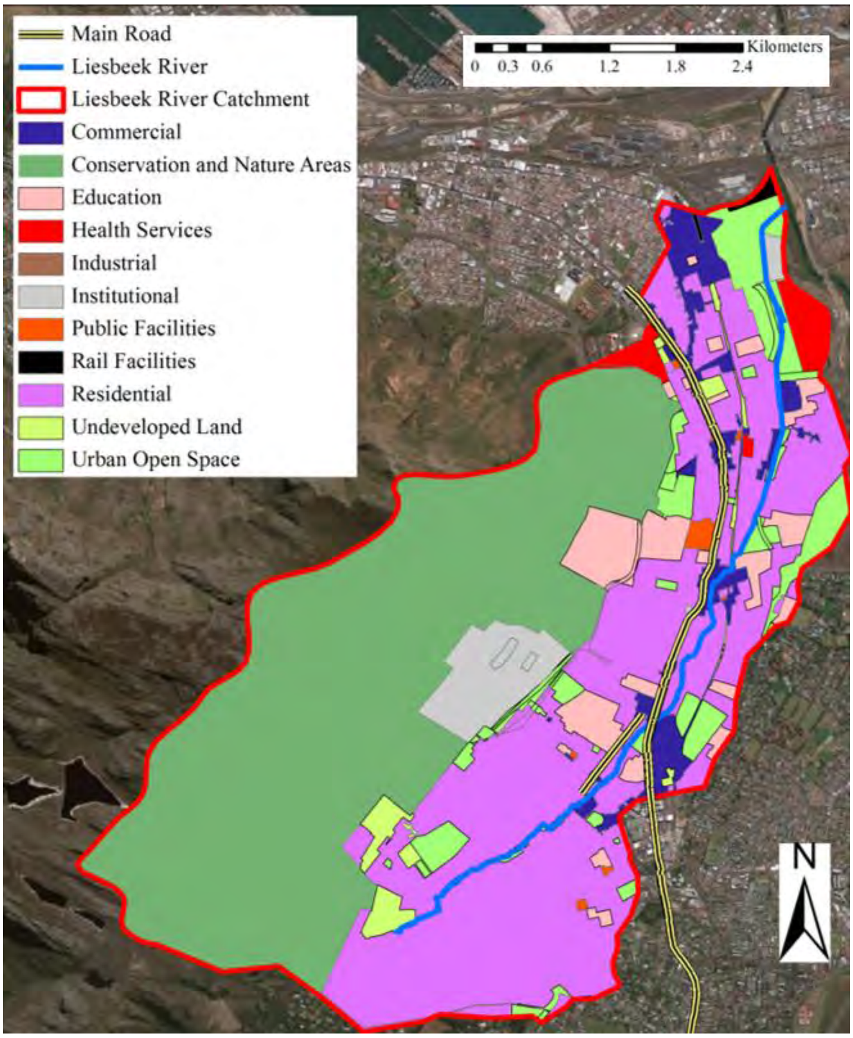

Figure 3-5: Land use in the Liesbeek catchment

According to StatsSA (2013a), the Liesbeek River Catchment had a population of approximately 31,000 people in 2011. The population was, and continues to be, unevenly distributed across the catchment. Bishopscourt, the most affluent suburb, has a density of around four people per hectare, whereas most of Rosebank, Mowbray and Observatory has a density of between 40 and 60 people per hectare. In areas where there are university residents and/or blocks of flats, the density reaches a maximum of 300 people per hectare.

Table 3-1 presents typical property value and household income from the property valuation data available from the CoCT (CoCT, 2012) and household data from Census 2011 (StatsSA, 2013a), respectively.

Owing in part to the presence of the University of Cape Town (UCT) within the catchment, as much as 80% of the residential accommodation in the middle to lower parts of the catchment is rented – mostly by students – as shown in Figure 3-6. This is important as it raises questions as to the social acceptability of RWH and SWH within the catchment – there is likely to be much less incentive for people renting properties to make savings compared with property owners. As noted by Fletcher *et al.* (2008), the success and the mitigation of the risks associated with RWH/SWH is dependent on the knowledge and commitment of the user, which in this case would often be the person renting the property, not the owner. Whether people who pay rentals that often include the supply of potable water would accept and use harvested rainwater / stormwater needs to be carefully considered through a social study (see Section 2.6.9) were RWH ever to be seriously considered for the Liesbeek River Catchment.

Table 3-1: Overview of land use data within the Liesbeek River Catchment

Suburb	Average erf size (m^2)	Median Household income (2011ZAR/yr.)	Median property value (2012ZAR)	Median property value (2012ZAR/m^2)
Bishopscourt	3,200	697,000	8,000,000	3,800
Claremont	870	493,000	3,730,000	5,700
Mowbray	470	243,000	1,490,000	3,600
Newlands	920	535,000	3,600,000	5,300
Observatory	280	188,000	1,190,000	5,100
Rondebosch	590	268,000	2,510,000	4,400
Rosebank	650	179,000	2,510,000	4,400

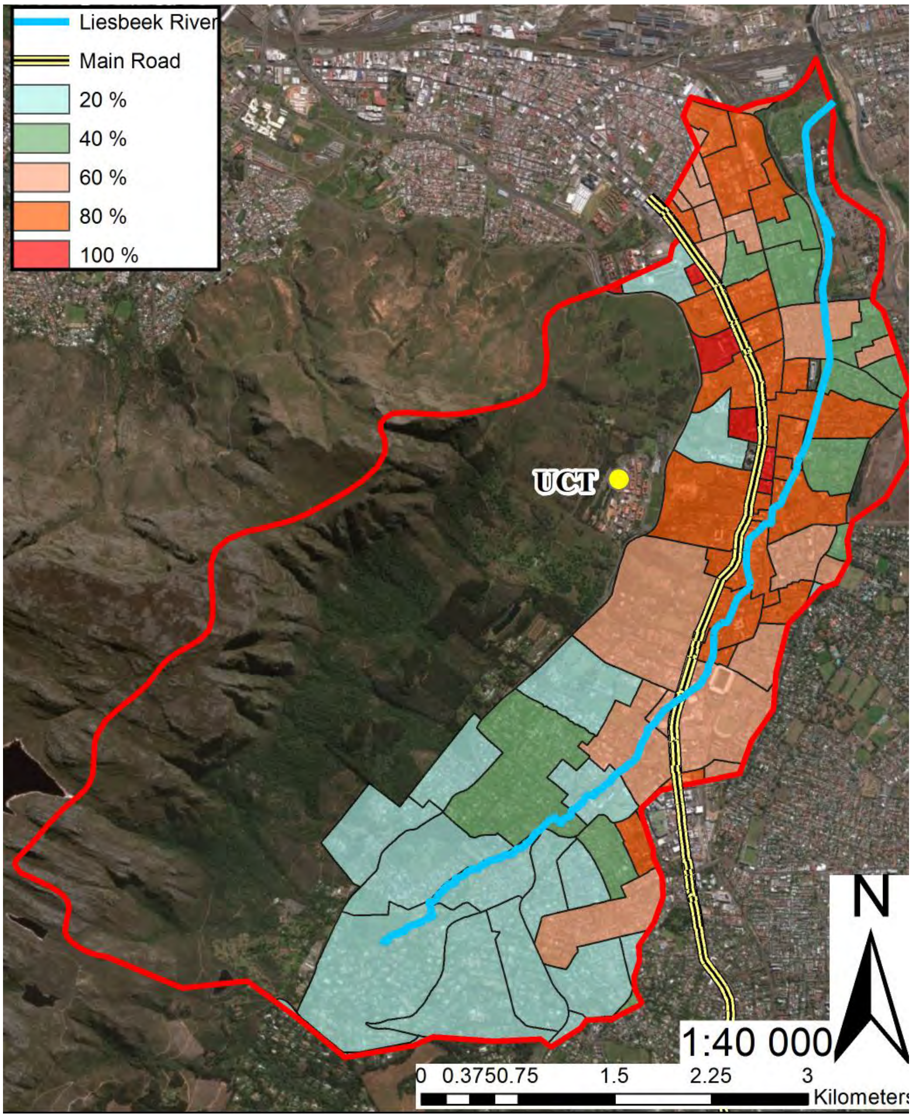

Figure 3-6: Percentage of households renting properties in the Liesbeek River Catchment

3.4 Summary of case study

The Liesbeek River Catchment incorporates a diversity of land uses, represents a range of wealth levels, and has significant historical importance for the CoCT and the RSA. Additionally, unlike many other catchments in the RSA, the necessary data for the effective development of the detailed models required for simulating catchment-wide RWH and SWH were available for the Liesbeek River Catchment.

4. Method

4.1 Scope and limitations

This thesis aims to investigate the viability of RWH and SWH in in the residential areas of the Liesbeek River Catchment, Cape Town, South Africa. The scope of this research is:

- In this research, RWH is primarily focused on single residential properties (i.e. houses, not flats) because RWH systems for blocks of flats would need to be designed for each block based on site-specific characteristics such as, height of the building, number of units, etc. These factors could significantly influence the cost, and unlike houses which are typically limited to two storeys, the blocks of flats in the catchment range from 3-10 storeys. It was also found that the level of occupation (number of units occupied in a block of flats) varied significantly and was inconsistent both between blocks of flats, and over time. As such it was not possible to obtain reliable modelling information for flats and this would likely lead to misleading results (further discussion is provided in Section 4.2.4)
- In this research, SWH considers end-uses in all residential properties (including houses, flats and university residents). This was possible due to the Census data providing an estimate of the total number of people in an area. Since SWH is not considered an on-site system in this study, it was not necessary to know exactly which building or block of flats people were living in, but rather how many people were serviced by a particular system, and the relevant suburb's per-capita indoor water demand in conjunction with the average end-use split (Section 4.2.4) used to estimate the toilet demand. It was assumed people living in flats and houses had similar toilet utilisation.
- This research primarily focused on the management of RWH and SWH from a quantitative perspective. The benefits of RWH and SWH were primarily assessed from a flood mitigation, water demand reduction and economic perspective
- This study investigated whether, and under what conditions, RWH and SWH may be considered as viable alternative water resources in urban catchments in the RSA. Therefore:
    - As far as was possible, RSA data were collected and utilised. Where no local data were available, international sources of data were used (see Sections 4.2.3.5 and 4.4);
    - Wherever possible, the software packages used in this thesis were those widely used in the RSA; and
    - An additional outcome of this study will be the development of a method that could easily and cost effectively be replicated across the RSA.

The limitations of this research are:

- The availability of reliable / reasonable life cycle costing and economic data is a common problem for researchers across the world (Wong & Ashley, 2006; Fletcher *et al.*, 2008, 2013). Wherever possible, cost data were collected from a number of sources. Life cycle costs were estimated using data provided with the Simple Economic Model as part of the South African Guidelines for Sustainable Drainage Systems and developed by this author (Armitage *et al.*, 2013) – see Section 4.4.5.
- A lack of readily available historic water quality data limited the potential to develop a calibrated water quality model that would provide any additional insights. However, the literature suggests that, provided rainwater / stormwater use is restricted to appropriate (typically non-potable) end-uses, *inter alia*, garden irrigation, flushing toilets etc., water quality in formalised residential areas can be managed, and associated risks mitigated through standard treatment methods, e.g. Ultra-violet filtration. Thus, assuming adequate treatment was provided (see Section 4.4), it was deemed unnecessary to model water quality – see Section 4.2.5 for further discussion of water quality and Section 4.4 for a discussion of how potential risks associated with water quality were accounted for.
- While this research assesses the potential impact of climate change on RWH and SWH, the level of the analysis is dictated by the availability of downscaled climate change models – see Section 4.2.3.3.

The following were not considered as part of this research:

- RWH was not considered for commercial properties in the catchment as it was found that most buildings housed a number of small separate shops, or were large shopping centres. The small shops typically have minimal water demand, such that RWH would not be viable. The large shopping centres would need to be assessed individually, based on the type of tenant and the volumes of water being used for different end-uses. This would require monitoring of each centre and shop, which was not possible for the whole catchment. The literature also indicates that where there is a high water demand and adequate collection area RWH would be viable – as is often the case for large commercial properties – but that these would need to be assessed individually.
- Educational, sporting and institutional properties in the catchment were found to have been relatively proactive in reducing their dependence on municipal water by making use of groundwater to flush toilets and irrigate fields. To assume these properties would instead use harvested rainwater or stormwater could lead to overly positive results for the viability of RWH and SWH in the catchment. Additionally, records for this use were not available, and therefore these properties were not included in this study.
- Groote Schuur hospital, one of the major users of water, was not considered in this study due to the risks associated with cross connections in a hospital and the impact this might have on patients. This would be further problematic in the RSA where maintenance and

accountability structures are often sub-standard. Additionally, from a practical point of view, the building is old and the potential to retrofit it is economically limited.

After eliminating other water users, residential properties account for 6348 of the 7286 properties in the catchment (approximately 87% of all properties). Specific limitations and related assumptions are discussed in Sections 4.2 to 4.5.3.

4.2 Data collection and processing

This thesis required the collection of a significant amount of data – over 500 gigabytes. Table 4-1 provides an overview of the data requirements for each aspect. There is a lot of overlap between the different aspects but, for example, the economic data required at the household (RWH) level is different from the economic data required at a regional (SWH) level. In the RSA, there have been no major studies that provide detailed end-use, household, or catchment-scale land use data comparable to that found in international studies such as Mayer *et al.* (1999); Loh & Coghlan (2003); Beal *et al.* (2011); and Biermann *et al.* (2012). Data was collected from the RSA, and preferably from the Liesbeek River Catchment. The following sections provide detail as to how the data were collected, processed and interpreted.

Table 4-1: Data requirements for modelling RWH and SWH

Data	Rainwater harvesting	Stormwater harvesting and Flood attenuation (suggested benefit of RWH and SWH)
Climate data	Rainfall data	
	Evaporation data	
Catchment	• Roof area	• Catchment area • Catchment imperviousness
	• Runoff coefficient	• Soil infiltration parameters • Depression storage
	• Depression storage	• Flow length • Overland flow roughness coefficients
Collection system	• First flush filter volume • Filter coefficient	• Collection network (e.g. manholes, pipe sizes etc.) • Conduit (pipe / channel) roughness coefficients
Storage	Storage size	
Distribution	Water demand data	
Economic data	Life cycle costs, benefits etc.	

4.2.1 Land-use data

The availability of data for modelling and calibration was identified as a potentially significant limitation to this research. Data for modelling and calibration were identified as a potentially significant limitation in this research. From a RWH perspective, it was evident that studies such as Biermann *et al.* (2012), which investigated the physical characteristics – including the total roof area, roof area connected to the RWH system, RWH storage tank capacity, and what end-uses the RWH is used for, e.g. toilets – have not been undertaken in the RSA. Therefore, the ability to estimate a 'typical property's roof area' (either as an average area or through the use of a statistical distribution) in order to apply '*the established basic method of analyzing the behavior of multiple rainwater tanks is by continuous simulation of a single rainwater tank system'* (Neumann *et al.*, 2011) was limited. Further, as discussed in Section 2.4.5.3, research by Mitchell *et al.* (2008a), Xu *et al.* (2010), Coultas *et al.* (2011), Maheepala *et al.* (2011), Maheepala *et al*. (2013), Mashford *et al.* (2011) and Neumann *et al.* (2011), has shown that the spatial and temporal variation in, amongst other factors, roof area may result in an error if the results of a 'typical' property with averaged inputs for roof area, demand, storage size, etc. are linearly scaled to represent the whole catchment. Consequently, these authors made use of stochastic analysis to represent the spatial and temporal variation. Mitchell *et al.* (2008a) and Neumann *et al.* (2011), amongst others, made use of values reported in literature and personal communication with members of the local water industry to estimate the descriptive statistics (mean, minimum, maximum, standard deviation) of roof areas, depression storage, tank size and effective roof area factors. Mitchell *et al.* (2008a) and Neumann *et al.* (2011) further assumed that, within a catchment, these values would be normally distributed. For this study, it was felt that – while the depression storage and effective roof area factors could be assumed to be normally distributed, as these are the products of an '*infinite number of independent random events'* (StatSoft Inc., 2013) – it would not be reasonable to assume a normal distribution for tank sizes and roof areas. A preliminary analysis indicated that the roof area distributions varied between suburbs, and that, even if they could be assumed to be normally distributed within a suburb, no local data existed to suggest using the appropriate statistics (mean, median, standard deviation).

From a SWH and flood modelling perspective, it was evident that there was limited flow data and that previous catchment stormwater models (see Section 4.3) were inadequate for this research and could not be relied upon to accurately model runoff using a continuous simulation. The collection of detailed land-use data was thus required for the development of the stormwater model, especially for the impervious fraction of the catchment. In so doing, it was possible to reduce the number of calibration variables. It was, therefore, decided to collect as much land-use data as possible. Two options existed for capturing this data, namely: manually capturing the data from orthophotographs or using automated capture methods which made use of the available LiDAR data (discussed further in Section 4.2.2). It was decided to capture the data manually from orthophotographs (CoCT, 2009b), as the resolution of the available LiDAR was not adequate for automated capture. The entire catchment was classified into one of the following categories:

- Roof areas
    - Flat roofs
    - Sloped roofs
- Impervious surfaces
    - Roads
    - Parking lots
    - Driveways
    - Sidewalks
    - Recreational (e.g. tennis courts)
- Pervious surfaces
    - Garden
    - Sidewalks
    - Undeveloped land
    - Recreational (e.g. soccer field)
- Swimming pools

In order to minimise error, the data capturers were provided with a set of standards to be used. These included: data capture to be undertaken at a minimum scale of 1:500 or greater (e.g. 1:400) and using geometric shapes to capture areas (where more than one shape was required, these could be merged). A verification process – including automated checking using the topology function in ArcGIS (ESRI, 2010) and a systematic manual check followed by spot checks by two separate researchers – was then undertaken to ensure accuracy of the captured data. Figure 4-1 indicates the typical level of detail for roofs, swimming pools and roads.

It was possible to obtain typical property value and household income from the property valuations data available from the CoCT (CoCT, 2012) and household data from Census 2011 (StatsSA, 2013a), respectively. These are summarised by suburb in Table 3-1 (and illustrated in Figure 3-3, Figure 3-5 and Figure 3-6). Of incidental interest is that some of the least expensive land measured on a square metre basis is to be found in Bishopscourt, which has the highest household income but, significantly, the largest property areas.

4.2.2 Topography

Topographical data were obtained from the CoCT in the form of Light Detection and Ranging (LiDAR). LiDAR is a '*remote sensing method that uses light in the form of a pulsed laser to measure ranges (variable distances) to the Earth. These light pulses – combined with other data recorded by the airborne system – generate precise, three-dimensional information about the shape of the Earth and its surface characteristics*' (NOAA, 2014).

Figure 4-1: Typical level of detail for roofs, swimming pools and roads

The available LiDAR data had a resolution of approximately 1m (spacing between measurement points). The data were provided in two sets. The first contained only ground points, and the second included all measured points (e.g. points on roofs). Using the first set of data, it was possible to create an accurate Digital Elevation Model (DEM) / surface model. This provided an accurate model for evaluating 2D flooding (See Section 4.3).

Unfortunately, due to the significant tree cover in some areas and the resolution of the LiDAR data, the results of automated roof area captures (based on analysis of the height of points above the ground surface) were, on visual inspection, found not to be accurate or reasonable. Therefore, it was necessary to capture them manually using orthophotographs – as discussed in Section 4.2.1.

4.2.3 Rainfall and evaporation

Rainfall and evaporation are important inputs when modelling RWH and SWH, as they can be limiting factors (DECNSW, 2006; Roebuck, 2007). This section discusses the collection and processing of rainfall and evaporation data for this study.

4.2.3.1 Hydrological year

Section 2.6.4.1 provides a discussion of the definition and importance of the hydrological year. For this research, the hydrological year was made to coincide with the calendar year (i.e. 1st January to 31st December) for the following reasons:

- Mitchell (2007) recommends that the simulation of RWH systems should regard the initial store as empty, and also Mitchell (2007) notes that this becomes more significant the shorter the simulation period. Owing to the fact that the start and end dates of the selected period fall in the middle of summer in Cape Town, it is very likely that RWH storage tanks will be empty throughout the catchment. It is therefore possible to model the initial conditions accurately.
- It would be easier to simulate the initial conditions when modelling the catchment (Section 4.2.5).
- Had the hydrological year started at another time (e.g. October; as is the norm for the RSA's DWS), it would not be possible to obtain rainfall data (Section 4.2.3.2) at all the selected stations for at least 10 years.

4.2.3.2 Rainfall data

As highlighted in Section 3.2, there is significant variability in rainfall within the Liesbeek River Catchment (from 600 to 1,500 mm/yr.). The South African Weather Service (SAWS) has three rainfall monitoring stations within the catchment, shown in Figure 4-2 (Kirstenbosch, Groote Schuur and Observatory). On further analysis, the data at Groote Schuur were found to have significant gaps in their records and the station was therefore deemed unsuitable. The Department of Water Affairs (DWA) also has a rainfall monitoring station in the catchment, shown in Figure 4-2 (Newlands), which was found to be reliable. However, it was felt that three stations were inadequate to represent the variability in rainfall within the catchment. In order to obtain additional rainfall records, an advertisement was placed in a local newspaper requesting anyone with rainfall data to make them available. A total of eight responses were received from private citizens who had been keeping rainfall records. These records ranged from 1 year to over 35 years. Each one of these records were assessed based on: the number of data gaps, what time the data providers said they took their reading, and the correlation of the record with the nearest SAWS rainfall station. The result was that three (Boschoff Street, Newlands; Rouwkoop Road, Rondebosch; and Spin Street, Observatory) of the initial eight stations were found to be acceptable. The selected stations were maintained by retired (at the time of the study) individuals who had been keeping records for an extended period of time, with recordings taken in the morning, typically around eight o'clock. This was valuable as DWS and SAWS daily readings are also reported at eight o'clock each day.

Each record was analysed and, if necessary, gaps filled in by reference to the nearest SAWS station. Two typical approaches were used for patching:

- Where the record indicated that a reading represented more than one day's worth of rainfall and was less than the volume of the rain gauge, the total was linearly scaled according to the nearest SAWS station's record.
- In cases where the record indicated a data provider had taken a holiday, the nearest SAWS station's record was used to linearly scale (based on the mean annual rainfall of both stations) the data based on the ratio of the mean annual rainfall.

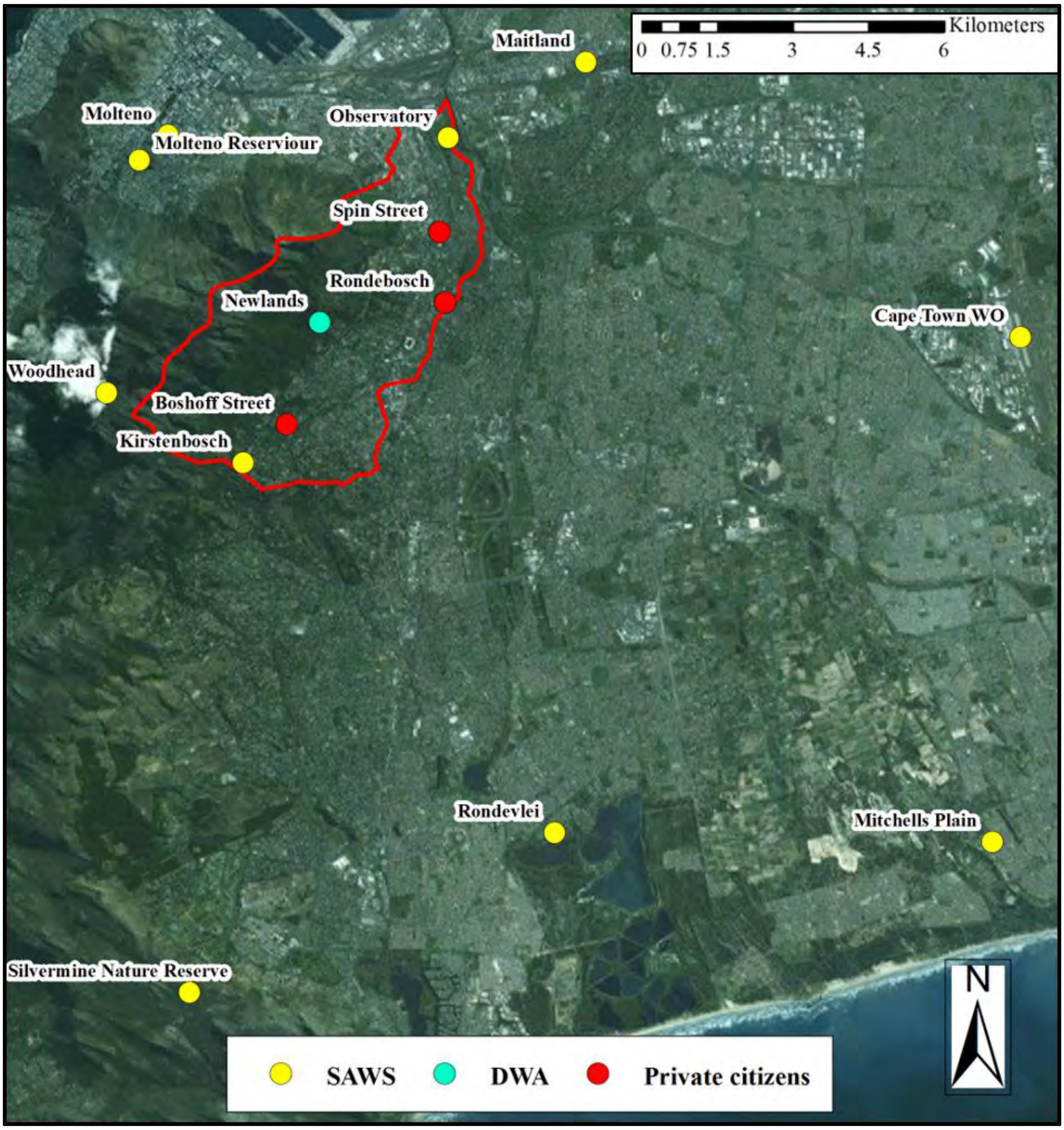

Figure 4-2: Rainfall gauging stations in and near the Liesbeek River Catchment

The result was that six (three private citizens, two SAWS and one DWS) ten-year daily rainfall records were available for the model. They were (fortuitously) distributed across the catchment. The records were limited to ten years as data at the Boschoff Street and Rondebosch stations were limited to this period. The mean annual precipitation (MAP) was calculated at each station, and using ArcMap©, interpolated across the catchment, as was shown previously in Figure 3-2.

It was evident that the temporal and spatial variability was significant in the catchment. In order to undertake an investigation using a simulation period of more than ten years would have required data to be stochastically generated for this catchment. It would, however, have been extremely difficult to replicate the temporal and spatial variability of rainfall within the catchment using the available models. Consequently, it was decided that generating longer rainfall records without adequately capturing the spatial and temporal variability of the rainfall would not have been particularly useful due to the intention to assess the impact of SWH and RWH on attenuating peak stormwater flows and mitigating flooding. Therefore it was decided to make use of the available 10 years of data, which, as highlighted in Section 2.6.4 and Table 2-16, is deemed adequate for the analysis of RWH and SWH.

While daily data are acceptable for RWH models that do not consider peak flow rates, they are not very useful for modelling the impact that RWH and SWH might have on peak flows and flooding. The mitigation of peak flows and flooding were important considerations in this study as they are typically put forward as benefits of RWH and SWH, which this research sought to investigate. It was, therefore, necessary to disaggregate the rainfall data. Only two rainfall stations had sub-daily (five minute) rainfall data (Kirstenbosch and Observatory), but this was limited to between 2 to 4 years' worth.

In order to generate 10 years of sub-daily rainfall data (i.e. extend the available dataset by 6 years), the method used in Fytilas (2002) was adopted. The method is outlined in Figure 4-3. This approach made use of the software *Hyetos* (Koutsoyiannis & Onof, 2001) to temporally disaggregate rainfall data from a station that had some sub-daily rainfall data (in this case, Kirstenbosch) to provide a complete 10 year sub-daily rainfall data set. Care was taken to preserve the basic descriptive statistics (mean, standard deviation, proportion of dry periods, lag 1 autocorrelation and cross correlation). *MuDRain*, a model capable of multi-site rainfall disaggregation, was then used to spatially and temporally disaggregate the available daily data at the remaining five rainfall stations. Key input parameters are presented in Appendix E.

The result of the above process was an hourly rainfall time series at each station that was as spatially and temporally representative as possible given the circumstances. This does not equate to being the same as what happened, but rather, statistically representative of how the rainfall might have been. The advantage of this approach is that it preserves the daily rainfall as measured across the catchment whilst providing an indication of the typical temporal and spatial nature of rainfall at a time step of one hour, which is less than the catchment's calculated time of concentration (one hour and 20 minutes).

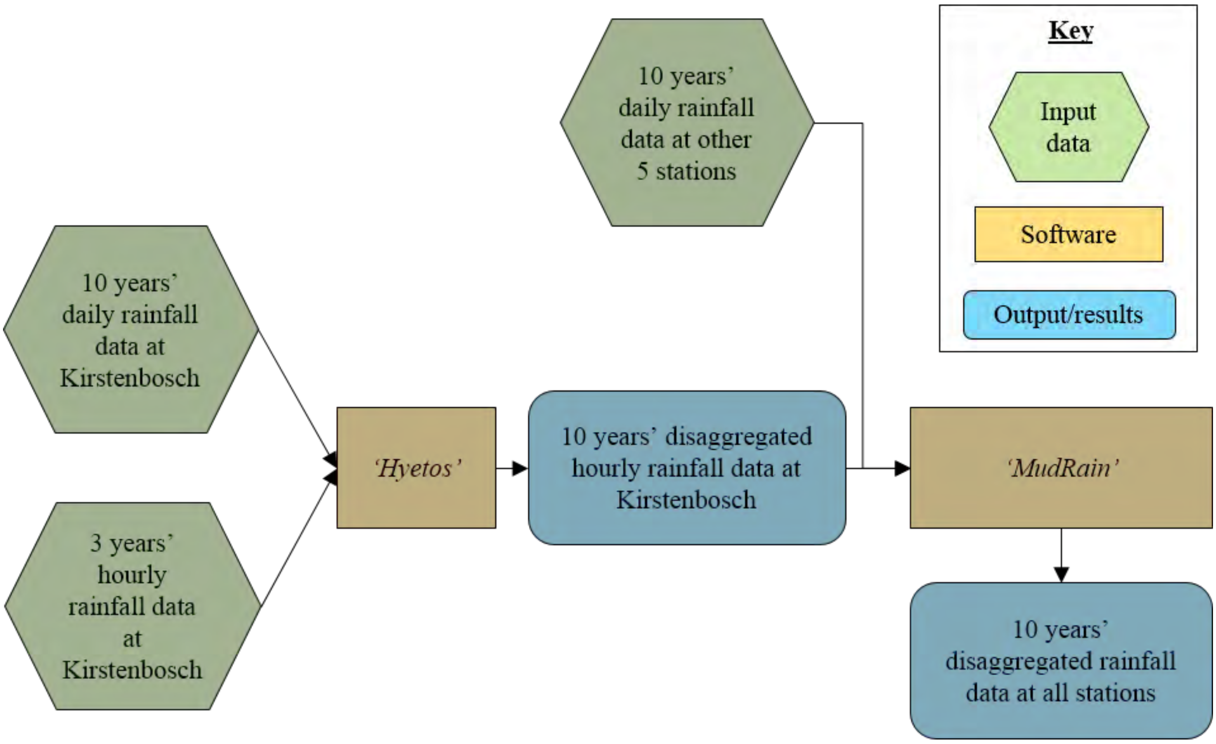

Figure 4-3: Rainfall disaggregation process

Sub-daily rainfall data was available for the entire calibration period (at both Kirstenbosch and Observatory). It was therefore not necessary to make use of *Hyetos* – which was used to disaggregate the available historic daily data at Kirstenbosch and Observatory – and consequently it was only necessary to make use of *MuDRain* to spatially and temporally disaggregate the daily rainfall data to hourly rainfall data at the remaining stations for the purposes of calibration.

4.2.3.3 Evaporation data

Evaporation gauging stations were scarcer that rainfall gauging stations, with only one station within the Liesbeek River Catchment in operation, as shown in Figure 4-4. Another five stations with historic data (see Figure 4-4) were available in neighbouring catchments. However two of these stations (Constantia and Seekoe Vlei) were no longer operating. The evaporation data were available as readings from 'Symons pans'; and at some stations, 'Class A Pan' readings were also available. As the Symons tank readings were available at all stations, these readings were used for uniformity. The readings were adjusted to reservoir evaporation based on the recommendations of the DWA (Myburgh & Kriel, 2012), using the coefficients presented in Table 4-2 and Equation 4-1.

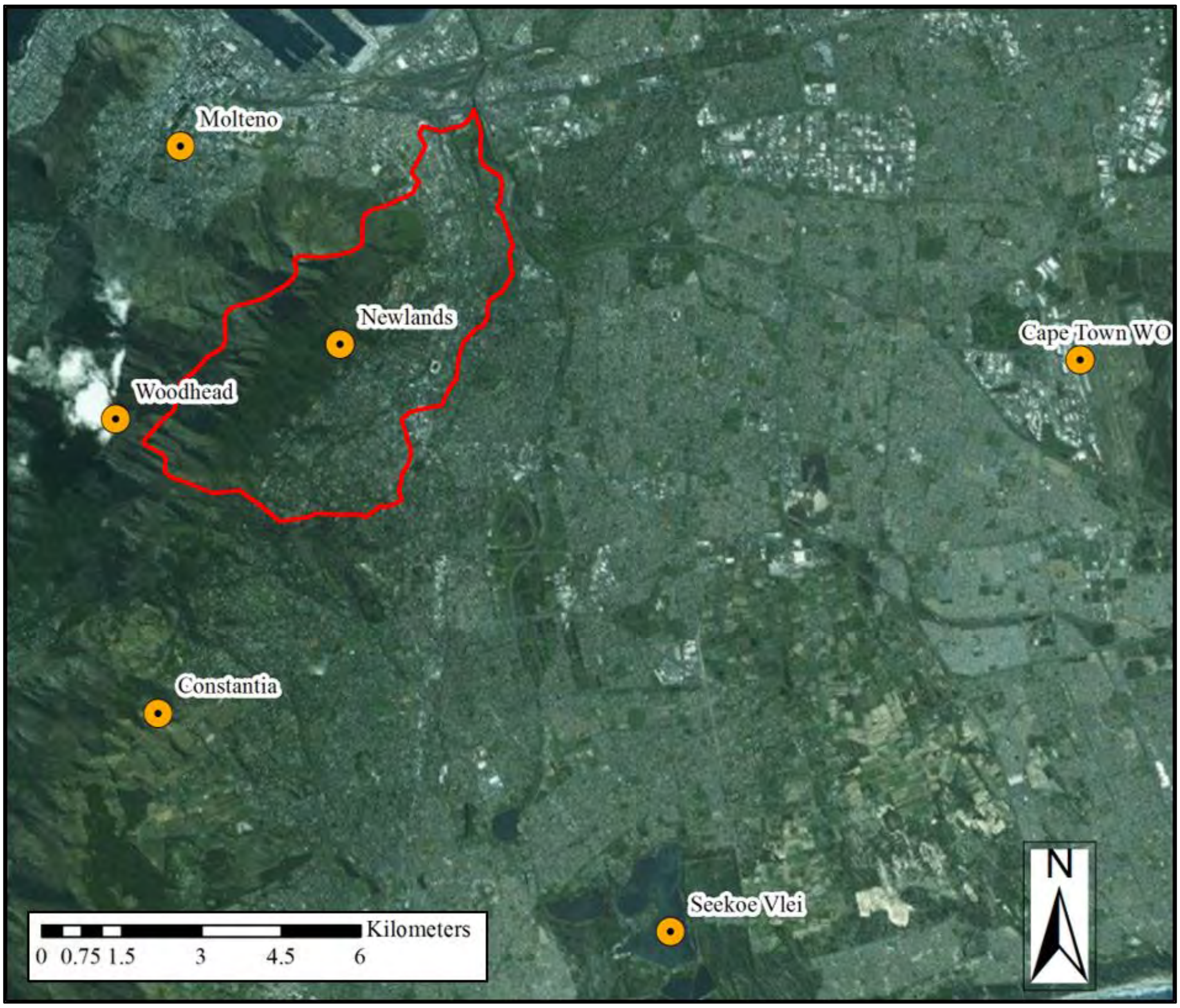

Figure 4-4: Evaporation gauging stations in and near the Liesbeek River Catchment

Table 4-2: Symons tank to reservoir evaporation factors (Midgley *et al.*, 1990; Myburgh & Kriel, 2012)

Month	**Pan coefficient**	**Month**	**Pan coefficient**	**Month**	**Pan coefficient**	**Month**	**Pan coefficient**
January	0.84	April	0.88	July	0.83	October	0.81
February	0.88	May	0.87	August	0.81	November	0.82
March	0.88	June	0.85	September	0.81	December	0.83

$$E_R = k_m \times p_{m/d} \qquad \textbf{4-1}$$

Where E_R = Evaporation from an open water surface / reservoir (mm); k_m = crop factor, pan coefficient or pool / reservoir / open water body monthly factor and $p_{m/d}$ = pan evaporation for month or day (mm)

The mean evaporation was then calculated at each evaporation gauging station and interpolated across the catchment on a month by month basis (refer to Figure 3-2).

Daily evaporation data were derived using the process laid out in Figure 4-5. Using the historic temperature (daily minimum and maximum) data at Kirstenbosch, evapotranspiration was computed using *SWMM*, which makes use of Hargreaves' method (Rossman, 2008). The Hargreaves method '*has shown reasonable results with a global validity*' (Allen *et al.*, 1998). The resulting time series of daily evapotranspiration values were calibrated using Equation 4-2 (Allen *et al.*, 1998), against catchment evapotranspiration as estimated using historical data multiplied by the pan-coefficients, presented in Midgley *et al.* (1990, p. 34). The results were then scaled using a monthly calibration factor according to the monthly historic evaporation records throughout the catchment.

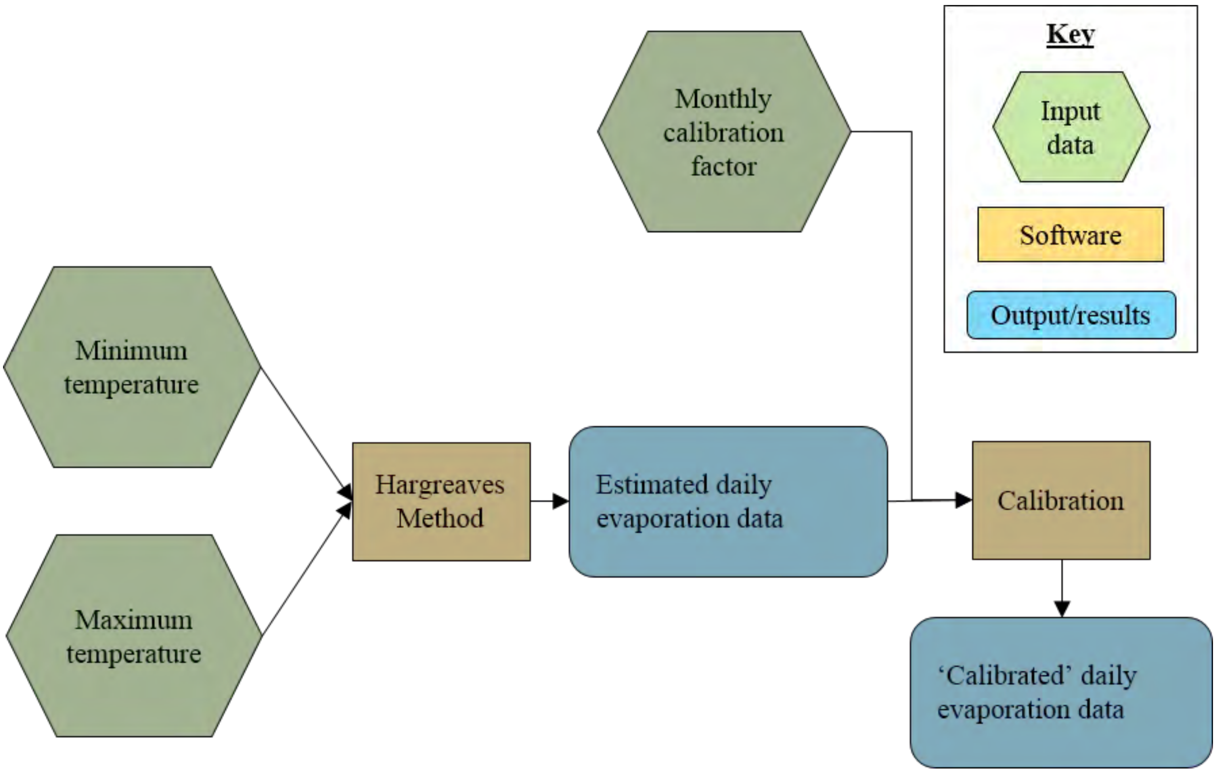

Figure 4-5: Daily evaporation data calculation process

$$ET_o = a + b \times ET_h \quad \textbf{4-2}$$

$$S_P = \frac{ET_o}{k_{CET}} \quad \textbf{4-3}$$

Where: ET_o = evapotranspiration (mm); a,b = coefficients determined by regression analyses or by visual fitting; ET_h = evapotranspiration calculated using Hargreaves Method (mm); S_p = Symons pan evaporation (mm) and k_{CET} = coefficient for catchment evapotranspiration in Midgley *et al.* (1990, p. 34)

4.2.3.4 Modelling the effects of climate change

The reality and potential consequences of climate change mean that studies such as this should attempt to consider its potential impacts. Downscaled rainfall data based on the CMIP5 models (Coupled Model Inter-comparison Project Phase 5) – available from the SAWS through the University of Cape Town's Climate Systems Analysis Group – were used to analyse the impacts of climate change. The data were based on two (RCP4.5 and RCP8.5) of the Representative Concentration Pathways (RCPs) discussed in Vuuren *et al.* (2011). The RCP4.5 may be considered as an intermediate mitigation scenario, while the RCP8.5 should be seen as a high-emission scenario (van Vuuren *et al.*, 2011). Data were only available at two SAWS rain gauge stations (Kirstenbosch and Observatory). Sixteen models, as listed in Appendix L, were used. This represented a total of 31 climate change scenarios at each station. The Kirstenbosch station represented two urbanised catchments that contained 100 properties, while the Observatory station represented 14 catchments and approximately 1,300 properties – as shown in Figure 4-6.

Unfortunately, it was not possible to represent the whole Liesbeek River Catchment with downscaled climate change data, but fortuitously, the suburbs that did have data (Bishopscourt and Observatory) represent opposite ends of the spectrum when it comes to property size, personal wealth, rainfall, evaporation and other factors that influence water demand (Section 2.6.5). As such, it allowed for inferences to be drawn from modelling the changes in the viability of RWH across the catchment. Due to the scale of SWH, and the time step of the available downscaled data, it was not possible to model the SWH in the same manner as RWH owing to the fact that the available climate change adjusted rainfall data only covered seven SWH catchments, which was considered too few to provide a reasonable level of insight into the impact of climate change on the SWH systems. Instead the 10 year hourly rainfall and evaporation data was adjusted on a monthly basis according to the average predicted impact of climate change.

No climate-change adjusted data were available for evaporation. Therefore, the same method applied in Section 4.2.3.3 was used. Evaporation was computed from each climate change model's simulated temperature records using *SWMM*. The results were then scaled according to the variations between the calculated evaporation and historic evaporation measurements.

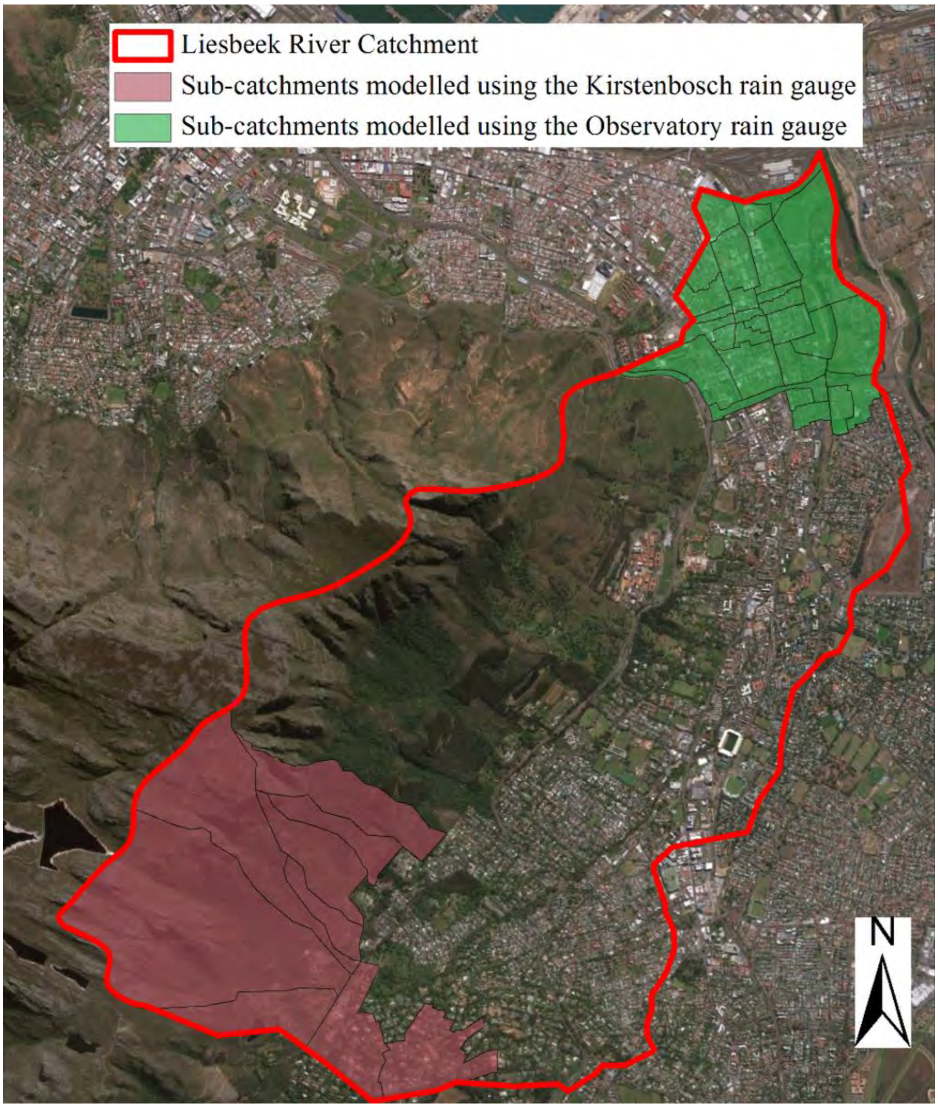

Figure 4-6: Areas represented by the Kirstenbosch and Observatory rain gauges

4.2.3.5 Overview of climate change data

The impact of climate change was assessed considering the predicted changes in rainfall and evaporation between the period 1979–2012 and the period 2050–2099 suggested by a selection of downscaled climate change models. As discussed in Section 4.2.3.4, a total of 31 climate change scenarios that represented intermediate and high-emission scenarios were considered. The rainfall data is as per the output from each respective model, while the evaporation data is derived from the temperature data – as discussed in Section 4.2.3.3. A number of trends were evident and determined to be relevant to this research; they are highlighted in the following figures. The summary of the rainfall and evaporation data for each scenario are presented in Appendix N.

Figure 4-7 illustrates the current average monthly rainfall for the Observatory rainfall station and the potential changes as a result of climate change, expressed as a percentage of current rainfall. It is evident that there are more significant changes for some months than others. Generally though, it is reasonable to expect: a decrease in rainfall for January to May;

the rainfall for November, June and August to remain roughly unchanged; and July, October and December to experience slightly increased rainfall. On the other hand, the climate change modelling for the Kirstenbosch rainfall station (Figure 4-8) indicates that, in general, it is reasonable to expect a decrease in rainfall for January, March, April and May; February to remain roughly unchanged; and June to December to experience slightly increased rainfall.

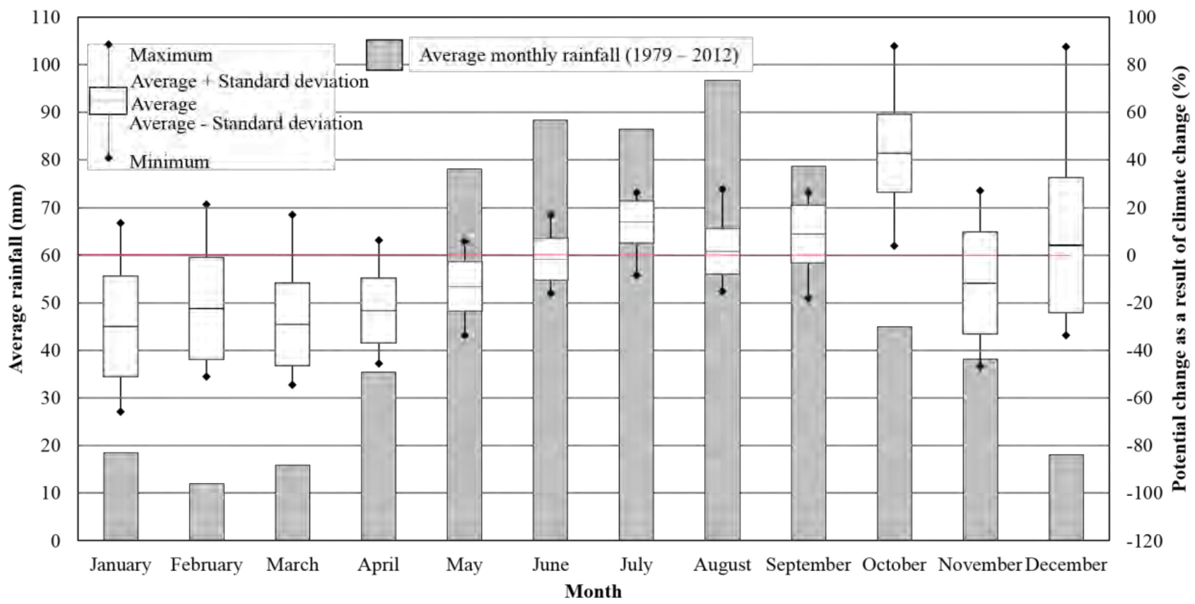

Figure 4-7: Potential impact of climate change on rainfall (Observatory)

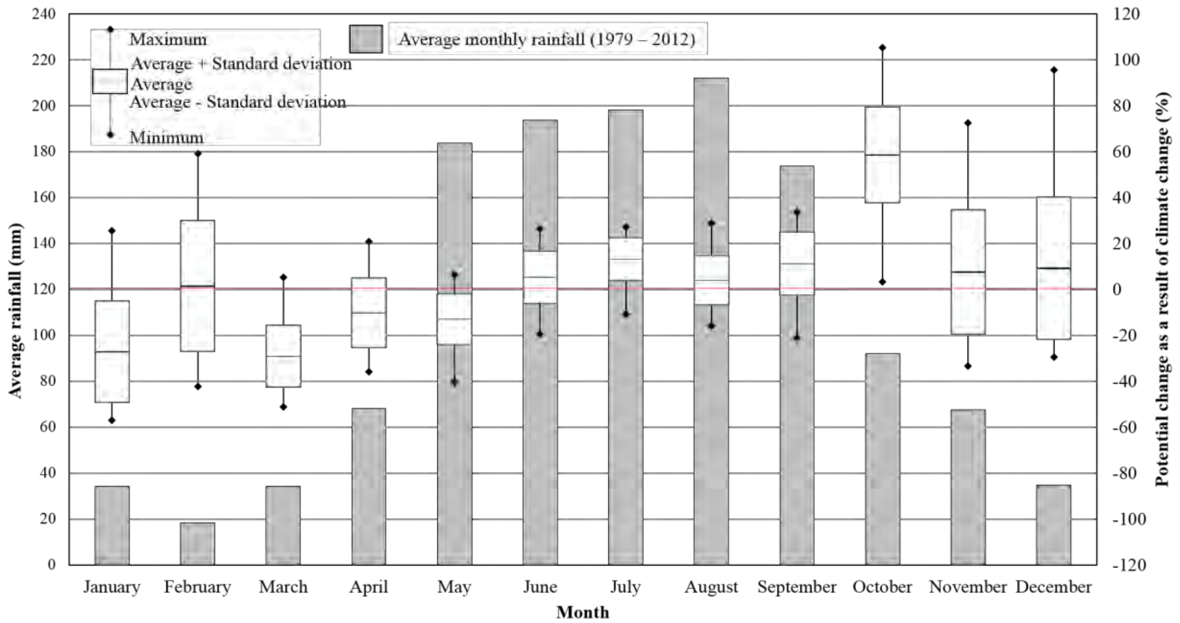

Figure 4-8: Potential impact of climate change on rainfall (Kirstenbosch)

Figure 4-9 illustrates the current average monthly evaporation at Observatory and the potential changes as a result of climate change (as a percentage of current evaporation). Unlike rainfall, there is a clear trend that evaporation will increase over the whole year. A similar trend is to be seen for the Kirstenbosch station (Appendix N).

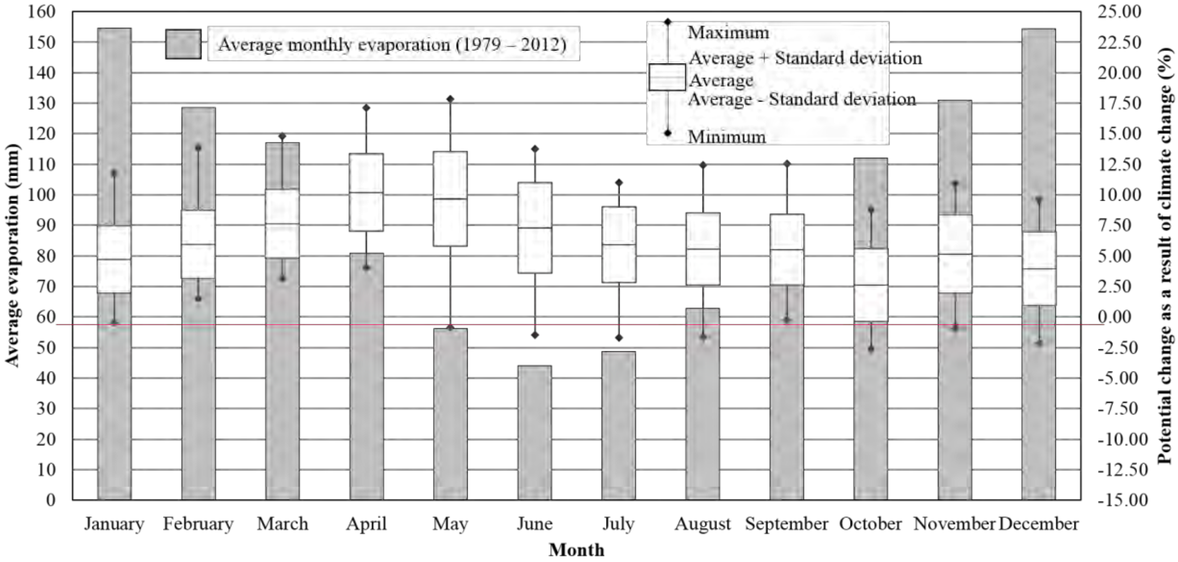

Figure 4-9: Potential impact of climate change on evaporation (Observatory)

The potential impact of climate change is significant as decreased rainfall and increased evaporation have the potential to reduce runoff volumes, which could significantly impact the performance of both RWH and SWH systems. This is discussed further in Section 5.1.2.

4.2.4 Water demand disaggregation

In order to assess the viability of using rainwater and stormwater for different end uses, it is necessary to have an estimate of the water demand for each end use. The end uses can be broadly characterised into indoor end uses and outdoor end uses. It is also important that, as far as possible, the spatial and temporal nature of water demand be represented as realistically as possible to minimise error. The variation and factors affecting the temporal and spatial variation of water demand are discussed in Section 2.6.5. Considering the relative importance of water demand in modelling RWH and SWH systems, it was necessary to disaggregate the available water demand data. The process undertaken in order to disaggregate the available water demand data is broadly divided into four stages, as outlined in Figure 4-10.

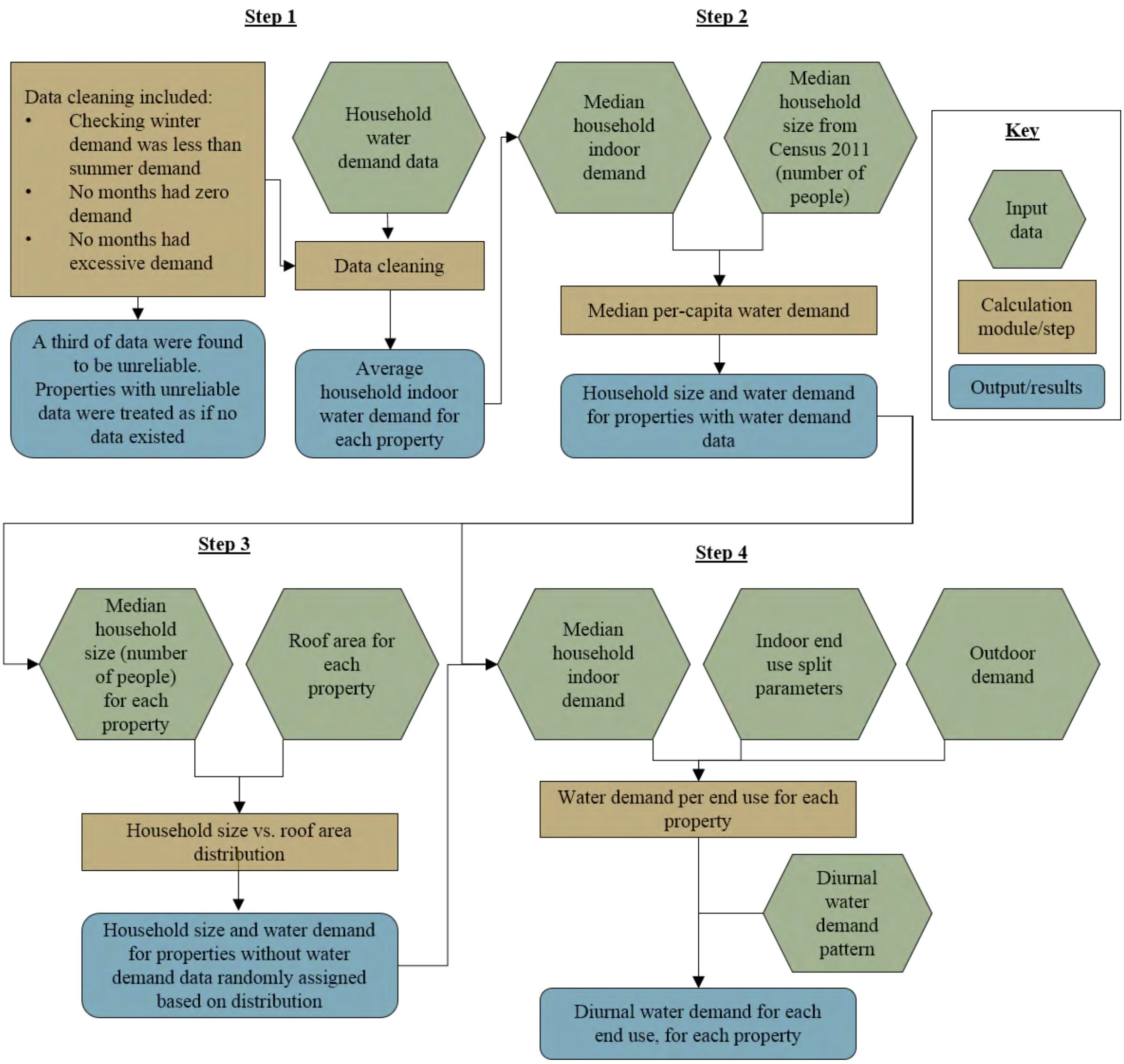

Figure 4-10: Process for disaggregating water demand

4.2.4.1 Step 1

Step 1 is focused on determining the average indoor water demand for each household.

Monthly water billing data were available from the CoCT; however, the data needed significant ‘cleaning’, as the water meters are not always read on the same day every month. This task was undertaken by GLS Consulting (Fair, 2013) who interpolated the readings to the 25th of each month for the period June 2010 to May 2011. Fortuitously, the period of the data ends in the same year as the latest Census for which population data are available. The available data for each household were then filtered in *Microsoft Excel* using the following assumptions:

- The average demand was greater than 6 kℓ per month (this equates to the minimum free-basic service amount used as a social provision of water to the very poor, who would not be expected to use additional water during summer);
- Since the local climate is characterised by mild, wet winters and dry, warm summers (Section 3.2) there is typically no need for irrigation during winter, but a significant need for irrigation during the summer months. Therefore, the winter demand is assumed to be less than summer demand (to eliminate anomalies that cannot be readily reconciled with end-use modelling and therefore imply some other unknown behaviour, e.g. leakage); and
- The demand during winter is reasonably constant (within 10% of the average winter demand) in order to eliminate unknown behaviour (e.g. pipe bursts).

A third of the households were removed through this filtering process – most commonly as a result of missing monthly data.

In order to estimate indoor water demand, an assumption was made that water demand during winter, when there is generally abundant rainfall (Cape Town is a winter rainfall area), would largely equate to indoor demand – as there would be no need to irrigate gardens or fill pools etc. During the summer months when rainfall is typically scarce, there is generally a demand for outdoor irrigation and filling of pools. Figure 4-11 illustrates that as rainfall decreases, irrigation demand increases – the exact profile of this relationship is dependent on a number of factors, *inter alia*, the levels of evapotranspiration, amount of rainfall, type of plants, and the presence or absence of automated irrigation systems. This approach has been used elsewhere (e.g. Howe & Linaweaver, 1967; Mayer *et al.*, 1999; DeOreo, 2011). End-use modelling, using the REUM model developed by Jacobs & Haarhoff (2004), was adopted to identify which months most realistically represented indoor demand (i.e. no need for outdoor demand). Figure 4-12 indicates the results for outdoor water demand for the different suburbs. For the purposes of uniformity, it was assumed that the average water demand for each property during May to July – months for which there is no requirement for irrigation anywhere in the catchment – represented a reasonable estimate of indoor water demand, as outdoor demands (e.g. car washing) during these months are likely to be relatively small compared with indoor demands and, for the sake of simplicity, are ignored. The output from Step 1 is an estimate of indoor water demand for each property that had reliable monthly water demand data.

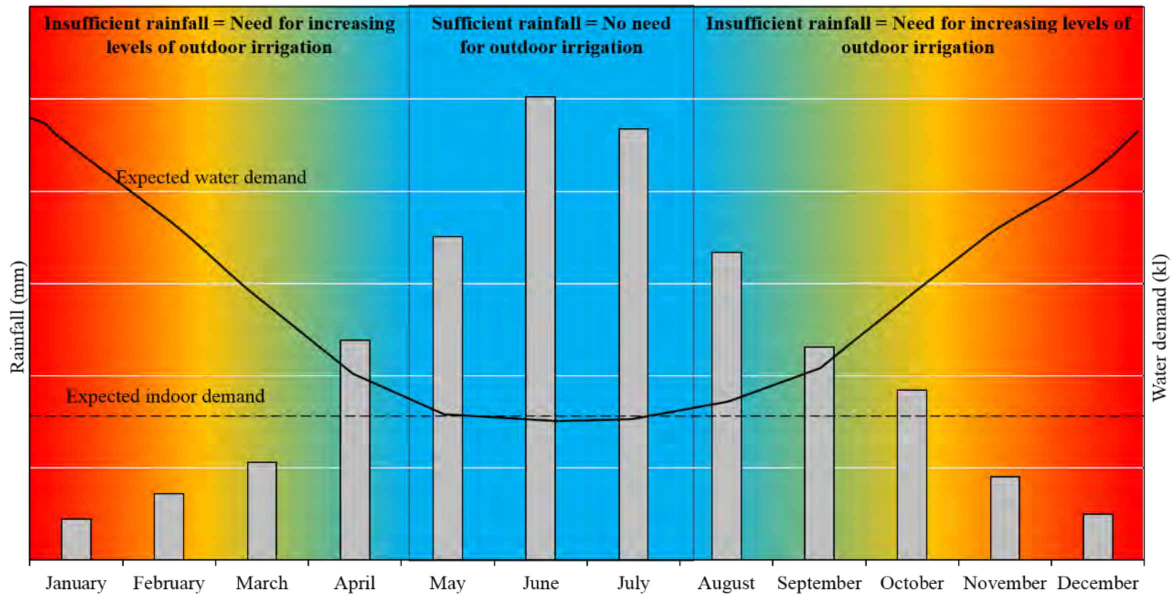

Figure 4-11: Theoretical basis for estimating indoor demand

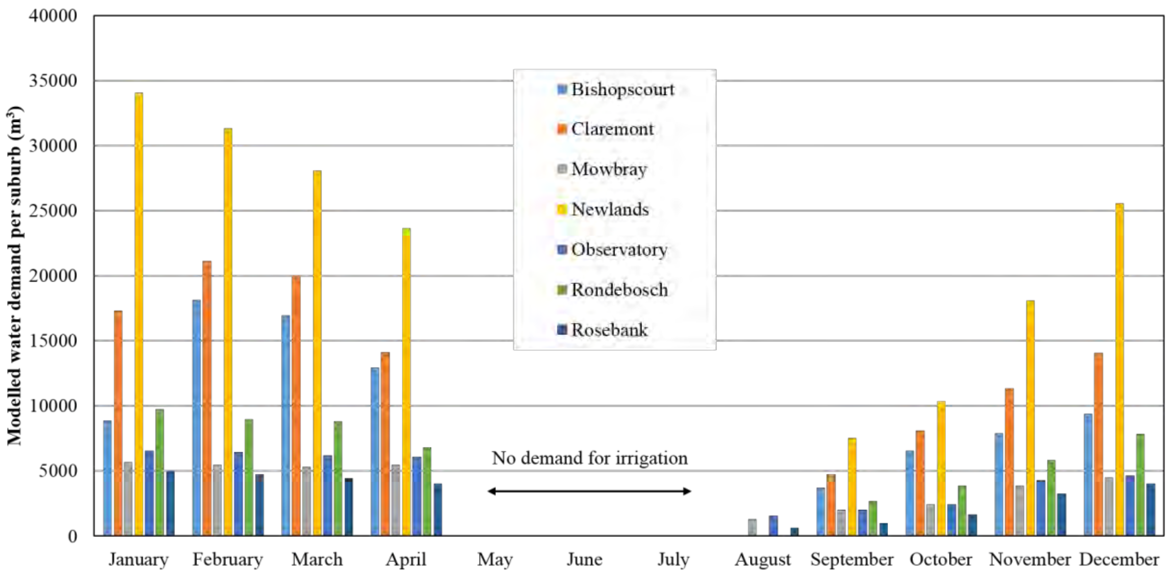

Figure 4-12: Modelled irrigation demand used to identify the most appropriate months to assume no outdoor demand

4.2.4.2 Step 2

Step 2 is focused on determining the household size for each property with reliable water demand data.

In order to ensure confidentiality, StatsSA (2013a) does not provide Census 2011 data at the household level, but rather at the 'small area' level. At this level, the data represent at least 500 people, ensuring anonymity (Grobbelaar, 2005). StatsSA (2013a) does, however, provide a count of the households' sizes (number of people) in ten different categories (households between 1-9 people and greater than 10 people). Using this data, an estimate of the median household size was calculated for each suburb (which incorporates a number of 'small areas') – owing to the fact that the sample sizes (after filtering) at the small-area level were too small to use for the calculation of a median household water demand, which is required for the next calculation. Furthermore, in line with other studies (e.g. van Zyl *et al.*, 2008), it was assumed that socio-economic conditions within each suburb were homogeneous and thus the median household size and median water demand would coincide. It was, however, recognised that there was a considerable range between, for example, Bishopscourt, which houses some of the wealthiest people in the CoCT on the one hand, and Observatory, which is occupied by much poorer people, on the other.

The median indoor demand was divided by the median household size to determine an assessment of the 'average' indoor per capita demand for each suburb – Equation 4-4.

$$\textit{Indoor Water Demand Per Capita} = \frac{\textit{Median household indoor water demand}}{\textit{Median household size}} \qquad \textbf{4-4}$$

Each property's estimated indoor water demand (for those properties with water demand data after the filtering process) was then divided by their respective suburb's estimated indoor per-capita water demand to estimate the property's household size (number of people rounded to the nearest integer).The output from Step 2 is: an estimate of the per-capita indoor water demand for each suburb; and the number of people per household for each property that had reliable monthly water demand data. The per-capita water demand per suburb is presented in Table 4-3.

Table 4-3: Per-capita water demand by suburb

Suburb	Estimated indoor AADD (*l*/cap.day)	Suburb	Estimated indoor AADD (*l*/cap.day)
Bishopscourt	260	Observatory	180
Claremont	280	Rondebosch	210
Mowbray	220	Rosebank	240
Newlands	260		

4.2.4.3 Step 3

Step 3 is focused on determining the household size and water demand for each property without reliable water demand data.

Since a third of properties did not have reliable water demand data it was necessary to estimate water demand figures for these properties. Section 2.6.5.1 highlighted that there is a strong correlation between household size and indoor water demand. Therefore it was decided to estimate the indoor water demand for properties without water meter data using an estimate for household size. The estimated household size could then be multiplied by the average (for the appropriate suburb) per capita water demand value for each property. While there are inaccuracies at an individual household level, this approach should give reasonable results at the subcatchment and catchment level.

In order to estimate the indoor household water demand (assumed to be equivalent to household size × average per capita water demand for relevant suburbs) for properties without water demand data, it was necessary to find a proxy for household water demand. Table 4-4 presents a correlation matrix, derived from all properties in the Liesbeek River Catchment with water demand data, linking four factors that could potentially be linked to household water demand in the Liesbeek River catchment. Van Zyl *et al.* (2008) showed that property size, property value and geographical location are the dominant parameters influencing municipal water use. A number of authors have shown that outdoor demand is the result of the local climate, size of outdoor area / garden (that is maintained through irrigation) and the presence of a swimming pool (e.g. Mayer *et al.*, 1999). A variable that has not typically been considered is roof area, most likely because of the difficulty in obtaining this data. Roof area could, however, be considered an indicator of both household size and income (factors that drive water demand – see Table 2-17). It is evident that the correlation between water demand and each of the factors is relatively weak, especially for indoor water demand. The correlation further varies from catchment to catchment. Analysis of the various correlation coefficients indicated that the link between water demand (as a function of household size) and roof area – whilst poor – was as good as any other, and relatively easy to determine from the collected data (Section 4.2.1). Furthermore roof area can notionally be linked to household size (number of people living in a household is likely linked to the size of house they choose to live in).

Table 4-4: Correlation of variables that may be related to water demand

	Total water demand	**Indoor water demand**
Property area (m^2)	0.54	0.46
Total property value (2012ZAR)	0.55	0.44
Roof area (m^2)	0.55	0.46
Outdoor area (m^2)	0.52	0.39

The estimation of realistic indoor water demands from properties without reliable water use data was then determined as follows:

i) Properties in each suburb were categorised by roof area (grouped in bands of 200 m^2). Roof area was selected as a proxy for indoor water demand – which is assumed to be equivalent to household size × average per capita water demand for relevant suburb.

ii) For properties with indoor water demand data, the number of people in each household within each roof area band (as determined in Step 2) was used to determine the probability for household size (from 1 to 10 people in a household) for each band. Figure 4-13 illustrates, as an example, the probabilities for each household size within each band, for the suburb of Bishopscourt. These probabilities were converted into cumulative probability functions, as illustrated in Figure 4-14.

iii) Household sizes (number of people) were assigned to properties without water demand data, based on the probabilities developed above. This was done making use of a random number generator (in Microsoft Excel), that assigned properties without water demand data a random number between 0 and 1. This random number in conjunction with the appropriate cumulative probability function (based on the suburb within which the property is located, and the property's roof size) was used to assign the number of people in the household. This number was rounded up to the nearest integer – as people can only be represented in integer form. For example, a property in Bishopscourt with a roof area of less than 200m^2 which was assigned a random number less than 0.35 would be assigned a household size of 1; if the property was assigned a random number between 0.35 and 0.76 the property would be assigned a household size of 2; if the property was assigned a random number between 0.76 and 0.87 the property would be assigned a household size of 3 etc.

iv) The indoor water demand for each property was then calculated by multiplying the relevant suburb's calculated per-capita water demand (calculated in Step 2) by the property's assigned household size. This was done for all properties without water demand data in the catchment.

The output of this analysis was tested by comparing the predicted population for Bishopscourt with that reported in Census 2011. While Census 2011 provides useful insights into the population dynamics within the Liesbeek River Catchment, it will inevitably contain errors which may occur for a number of reasons, *inter alia*, erroneous omissions and inclusions, and reporting errors, as discussed in StatsSA (2012). Bishopscourt has only single residential properties and therefore the total population reflected in the Census should be equivalent to the total population calculated using the above approach. The results using the above approach indicated a population of 1,808 people in Bishopscourt, which is approximately 98% of that reported in the 2011 Census (StatsSA, 2013a). It was felt that this was a reasonable approximation of the suburb's population and that, due to potential errors in the 2011 Census and the poor quality of the water demand records, it was unlikely that a better correlation could be expected. Unfortunately, all the other suburbs contained blocks of flats (for which it was not possible to estimate the number of people present, as discussed in Section 4.1), and could therefore not be checked.

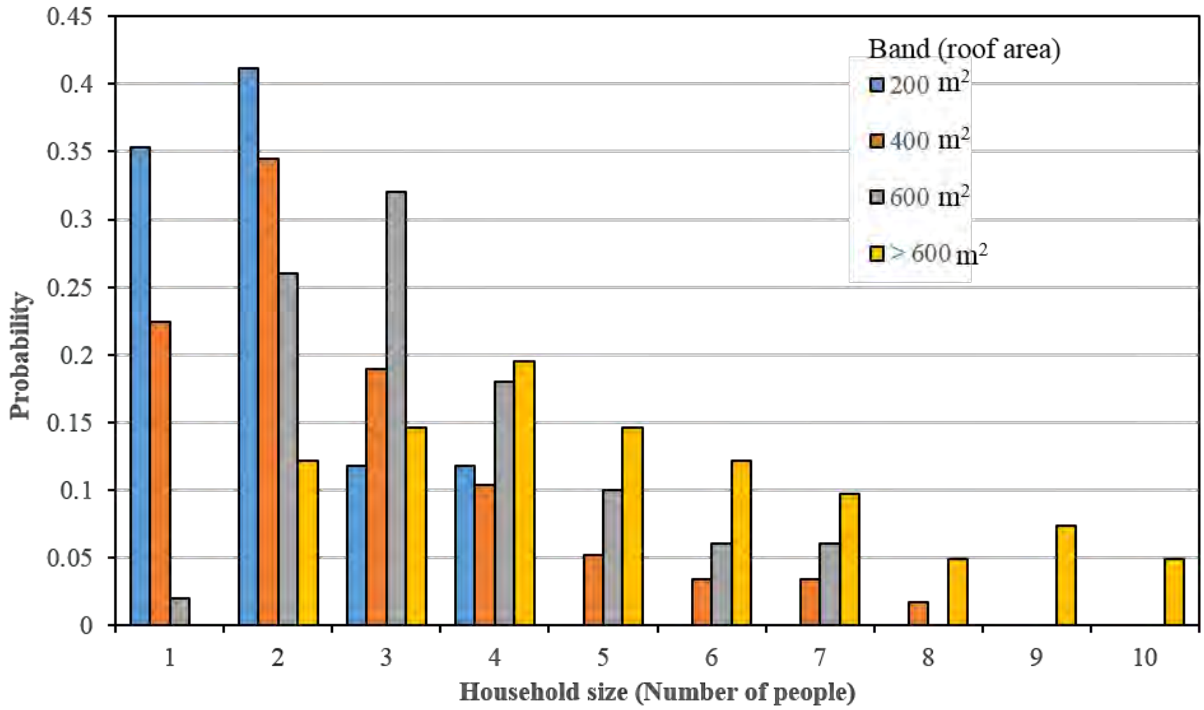

Figure 4-13: Probability of household sizes based on roof area for the suburb of Bishopscourt

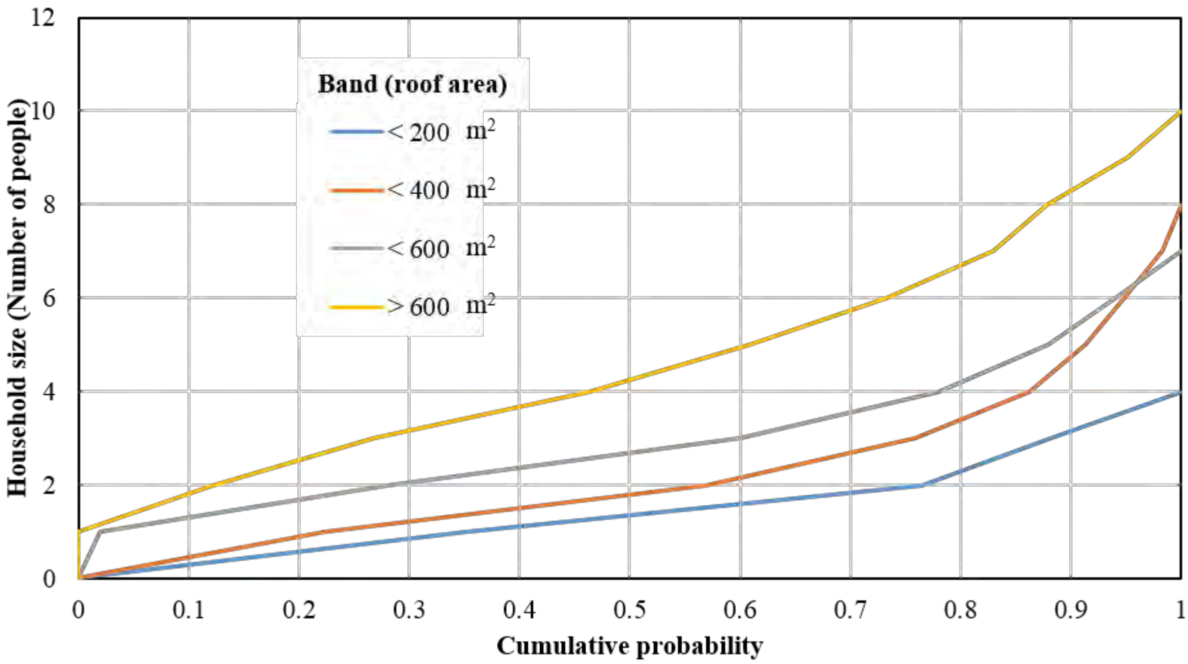

Figure 4-14: Cumulative probability of household sizes based on roof area

4.2.4.4 Step 4

Step 4 is focused on determining the water demand for each end use, for each property.

The average indoor water demand split (Figure 4-15) was estimated using indicative figures given in the literature (e.g. Mayer *et al.* (1999); Jacobs & Haarhoff (2004); Roberts (2005); Willis *et al.* (2009); Beal *et al.* (2011)). Adjustments using Couvelis (2012) (a study of on-site leakage in Cape Town) were made to account for on-site leakage as a percentage of indoor water demand.

Outdoor water demand was estimated using the end-use models proposed by Jacobs & Haarhoff (2004), with adjustments for irrigation efficiency as presented by du Plessis & Jacobs (2014). Outdoor demand was split into two main categories: garden irrigation demand and swimming pool demand. It was assumed that other demands such as car washing were negligible. Monthly water demand for irrigation was calculated using Equation 4-5, and monthly water demand for swimming pools was calculated using Equation 4-6. One change was made to the models to account for the fact that whilst Jacobs & Haarhoff (2004) estimated average monthly daily water demand (AMDD) by dividing the monthly water demand (MWD) by the number of days in the month, in this research the number of dry days in the month was used instead, as it was assumed individuals would not irrigate on days when it rained. This assumption is not entirely correct in that, especially in wealthier areas, it was observed that, in some cases, properties with automated systems would still irrigate on days when it rained.

$$MWD_{garden} = \frac{(f_{m,e} \times s_e) \times \left((k_{m,e} \times p_m) - r_m\right)}{IE} \qquad \textbf{4-5}$$

$$MWD_{pools} = (f_{m,e} \times s_e) \times \left((k_{m,e} \times p_m) - r_m\right) + \left(a_e \times b_e \times c_e\right) \qquad \textbf{4-6}$$

$$r_m = \begin{bmatrix} R & (R < 25\text{mm}) \\ (0.504 \cdot \mathrm{R} + 12.4 & (25\text{mm} \leq R < 152) \\ 89 & (R \geq 152) \end{bmatrix} \qquad \textbf{4-7}$$

Where: MWD = monthly water demand (kℓ/mnth); m = monthly; e = end-use; $f_{m,e}$ = pool cover factor/garden irrigation factor; s_e = surface area of vegetation type or pool (m^2); $k_{m,e}$ = crop factor or pool/reservoir/open water body factor; $p_{A\text{-}pan\,/\,Symons\ Pan}$ = pan evaporation (mm); r_m = effective rainfall (mm); IE = irrigation efficiency factor; d = days in a month; a_e = presence of pool filter; b_e = event filtering volume (m^3); c_e = frequency of use per month and R = monthly rainfall in mm/month.

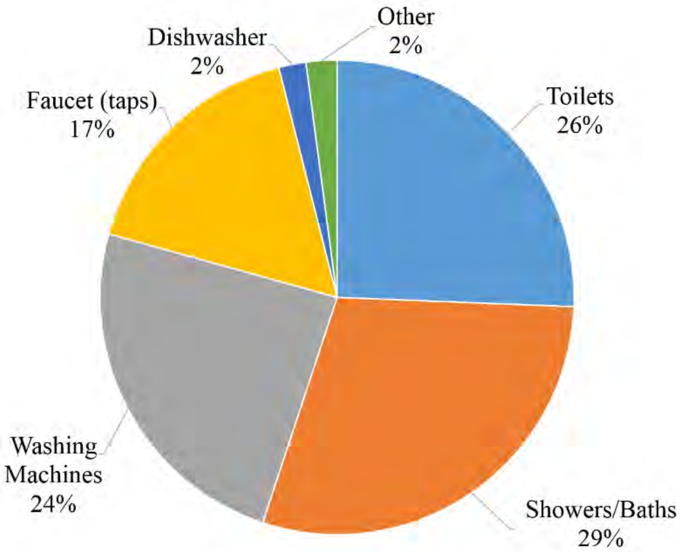

Figure 4-15: Typical end uses derived from international studies (Mayer *et al.*, 1999; Jacobs & Haarhoff, 2004; Roberts, 2005; Willis *et al.*, 2010; Beal *et al.*, 2011)

The outdoor irrigation factors used in the end-use modelling of outdoor demand required calibration. Two approaches to calibration were possible. The first approach would be to take each property's monthly water demand and subtract the indoor water demand and then calibrate the irrigation factors for the modelled outdoor demand for each property, for each month. The second approach would be to assume, in line with Section 4.2.4.2, that socio-economic conditions within each suburb were homogeneous and thus the water demand and how water is used outdoors would be largely similar in a suburb. The first approach could not be applied to all the properties as one third of the properties did not have reliable water demand data. For consistency it was decided to make use of the second approach which offered a simple and reasonable alternative applicable to all properties in the catchment. The second approach was undertaken using the following steps:

- The outdoor demand for each property with reliable water demand data was estimated by subtracting the household's estimated indoor demand from total water demand.
- The sum of all the individual estimated outdoor water demands (for properties with water demand data) was calculated for each suburb – and considered the 'actual' outdoor water demand for all properties with reliable water demand data in the suburb.
- Equations 4-5 and 4-6 were used to calculate and estimate outdoor water demand – that required calibration. Calibration was achieved using an iterative process which assumed that the irrigation efficiency factor (the degree to which a garden is over or under irrigated) remains constant throughout the year, the irrigation efficiency and monthly crop factors were adjusted until the 'actual' demand equalled the calculated demand for each month.

This process could have been simplified by considering the quotient of the crop factor and irrigation efficiency factor as a single factor for calibration – however, the iterative approach was taken to determine the degree to which properties in a particular suburb were typically over or under irrigating their gardens. The result was a set of irrigation efficiency and monthly crop factors for each suburb. These were then used to estimate the typical outdoor water demand for properties which did not have any water demand data.

Section 4.2.3.3 noted that Class A – Pan Evaporation data were generally not available. This proved problematic since crop factors ($k_{m,e}$) used to adjust measured evaporation to reflect evapotranspiration are based on Class A – Pan Evaporation data. As a result, the evaporation data used to estimate garden irrigation demand were adjusted on a monthly basis to reflect Class A – Pan Evaporation data, using Equation 4-8, in line with Midgley *et al.* (1990).

$$\textit{Class A-Pan} = 26.3622 + 1.0786 \times \frac{\textit{Evaporation (Reservoir)}}{\textit{Monthly conversion coefficient}} \tag{4-8}$$

In order to be able to simulate the impact of RWH and SWH on peak flows and flooding, it is important to simulate the amount of storage available to attenuate the runoff. This necessarily requires that the inflows and outflows (water demand) from the RWH and SWH storage are modelled at as fine a time step as possible, within reason. Where high resolution end-use data is not available (most situations) this can be done in two ways. The first is through the use of stochastic models, and the second is by superimposing a diurnal water demand pattern. Due to the significant variation in per-capita water demand between the different suburbs (Table 4-3) and the paucity of South African data on which to develop (or adjust the inputs of) a sub-daily stochastic model, the use of a stochastic water demand model would have had the effect of increasing the complexity of the analysis with little or no additional confidence in the quality of the results – as discussed in Section 2.6.2. Therefore it was decided to superimpose a typical diurnal water demand pattern for each end use. Duncan & Mitchell (2008) note that this has been done elsewhere. The diurnal patterns presented in Mayer *et al.* (1999) (summarised in Figure 2-17) were selected for the following reasons:

- They presented a diurnal pattern for each end-use.
- They presented results from a diversity of conditions.
- The diurnal patterns presented were based on a large sample in comparison to other available studies.

4.2.5 Water quality

The CoCT has a number of water quality monitoring points in rivers across Cape Town (Figure 4-16). At each point, a monthly grab sample is taken and tested for the following parameters: Total Suspended Solids, Total Phosphorous, Nitrates, Nitrites, Ammonia, Conductivity, Dissolved Oxygen, *E.Coli*, Faecal Coliforms, Orthophosphates, PH and Temperature.

While the records at the different monitoring points date as far back as 1975, only two of the sampling points (NR22 and NR08) are still used and overlap with the analysis period (Table 4-5). Furthermore, the monthly time step between grab samples means that the variation in water quality data is not sufficiently captured for the calibration of high resolution water quality models, especially during storm events which are of major interest for SWH. The consideration of water quality is further discussed in Section 4.3.7.

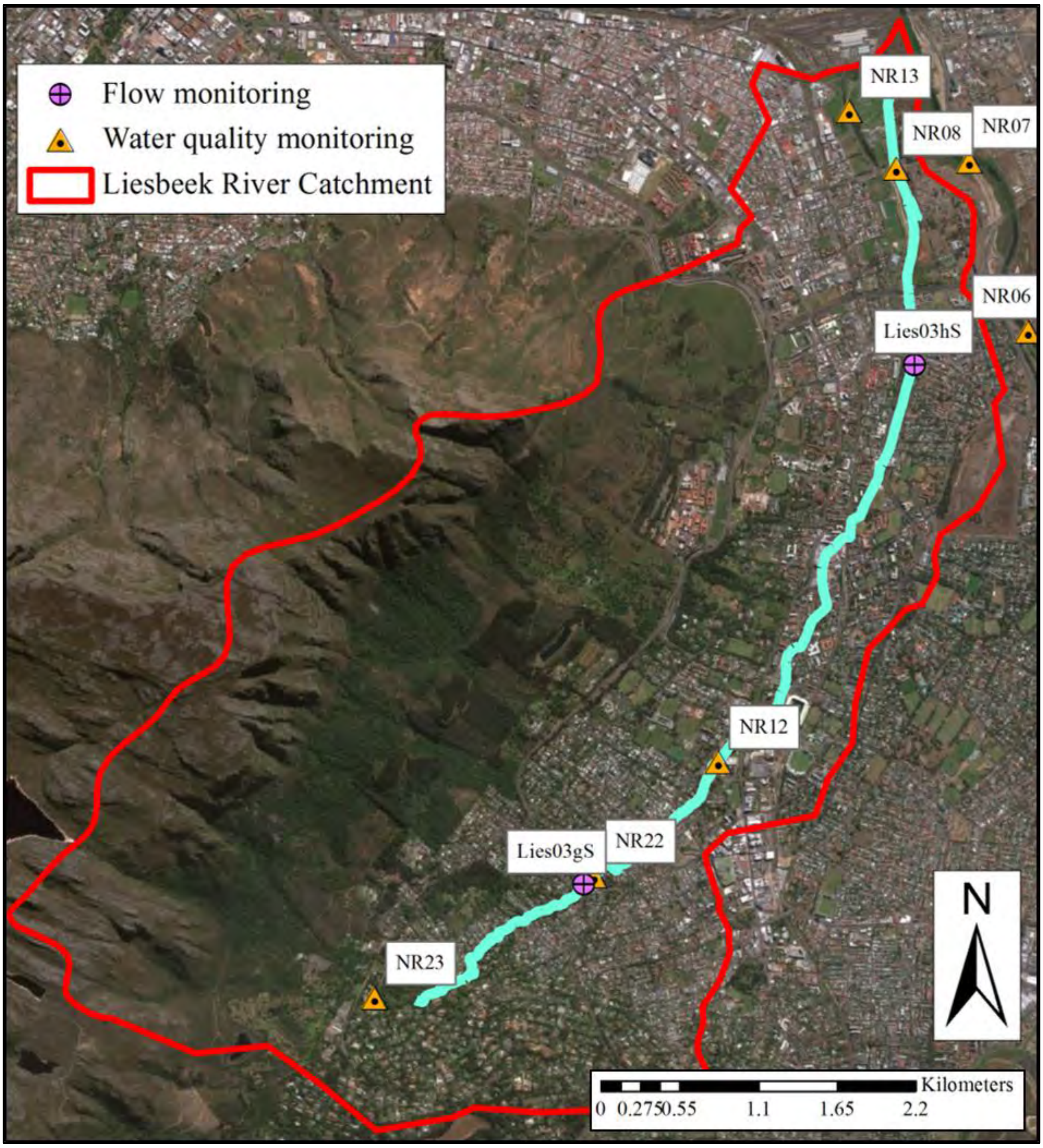

Figure 4-16: Water quality and flow monitoring stations

Table 4-5: Record length at selected water quality monitoring points

Target Guideline	NR23	NR22	NR12	NR08	NR13
Length of record	1988-2003	1988-current	1975-2011	1975-current	1975-2003

4.2.6 Flow data

The CoCT owns and maintains a number of continuous flow monitoring stations in rivers across Cape Town. These have been in operation for many years. Two of these flow monitoring stations (Lies03gS and Lies03hS) are located in the Liesbeek River. Unfortunately, although the stations were installed many years ago, they have not been maintained on a continuous basis and so data were only available for a period of 13 months (September 2012 to October 2013) at one of the stations (Lies03hS) whilst another station (Lies03gS) was clearly giving false data (constant readings). As a result, only Lies03hS was available for the calibration of the flow model. Fortunately, this gauge was located in the lower reaches of the catchment – see Figure 4-16. The use of the flow data for calibration is discussed in Section 4.3.

4.2.7 Cost data for economic analysis

In order to assess the financial and economic implications of RWH and SWH, it was necessary to collect life-cycle cost data. The following sections provide an overview of the source of the collected data. The use of the data is described in Sections 4.4, 4.5 and 4.5.2.

4.2.7.1 Capital cost data: General

Capital cost data were collected from many different sources:

- A number of recent (post 2010) successful tenders for bulk stormwater infrastructure projects in the RSA.
- A number of recent (post 2010) quotes from companies who specialise in the installation of RWH systems.
- The RSA's Department of Cooperative Governance and Traditional Affairs (DoCGTA) 'Industry guide to infrastructure service delivery levels and unit costs' (DoCGTA, 2010). This manual provides a significant amount of capital cost and maintenance cost data and was an important source of data for this research as it is based on extensive local research and stakeholder input.
- Bester *et al.* (2010) presented a set of simple-to-apply formulae that considered the cost of all standard components involved in the construction of gravity and pressure pipelines. The cost of manholes, for example, is averaged over the length of the pipeline. The formulae reflect the average values of a large number of successful tenders in the RSA. Special 'components', e.g. pressure reducing valves, however need to be added as an additional cost. These formulae are useful as they give an indication of the typical cost

for constructing water services in the RSA and are used by consultants to provide planning estimates as to the cost of projects (Bester *et al.*, 2010; Jacobs, 2014). The algorithms have been updated on a number of occasions.

- Where public data were not readily available – or sufficiently informative – private companies offering relevant services (e.g. a company that installs RWH systems) were approached to provide basic capital cost data for a range of alternatives. With respect to RWH systems, more than 20 companies were approached for data, but only two provided detailed information on the installation of a complete RWH system. On the other hand, many quotes, preliminary quotes and advertising material for components (e.g. storage tanks) of RWH systems were available from a number of additional companies. With respect to SWH, most of the data were obtained from recent tenders and recent literature. Specialist input into the cost of UV treatment was obtained from a company who specialises in supplying and installing large UV treatment systems.

In order to protect the commercial interests of companies that provided data, as with Roebuck (2007) companies have been referred to using pseudonyms, e.g. 'Company A'.

4.2.7.2 Capital cost data: Dual reticulation

Dual reticulation is where an additional water supply pipe is installed and typically used for non-potable water supply. The non-potable water supply is sometimes referred to as the 'third pipe' where there is one pipe for potable water, one pipe for sewage, and one pipe for non-potable water supply. The costs of a dual reticulation system are crucial to assessing the cost of SWH in the Liesbeek River Catchment as it is envisaged that harvested stormwater will be treated to non-potable water standards and used for non-potable end-uses. Since SWH can be undertaken at a range of scales (e.g. subcatchment to catchment), it was necessary to consider the costs at the different scales.

17 urbanised subcatchments (representing 14% of all the urbanised catchments) in the Liesbeek River Catchment were selected for dual reticulation design and costing. The selected subcatchments represented a range in size (3-30ha) and were distributed across the catchment, as illustrated in Figure 4-17. For each subcatchment, a water reticulation network (including pumps) was designed (using WADISO, a locally developed software package that essentially integrates EPANET with a GIS engine), that was capable of meeting peak water demand. The historical AADD (Section 4.2.3.5) was used for modelling the system. Peak flows were calculated according to the 'Red Book' (CSIR, 2005b), the accepted standard design guideline in the RSA. The design did not include additional capacity for fire flows, as this was assumed to be accounted for in the primary (potable) water supply network. The capital costs for each network were calculated using an updated version of the algorithms presented by Bester *et al.* (2010).

The analysis indicated that, as expected, the cost per connection of a dual reticulation network at the subcatchment scale decreased as the density of the catchment increased, as illustrated in Figure 4-18.

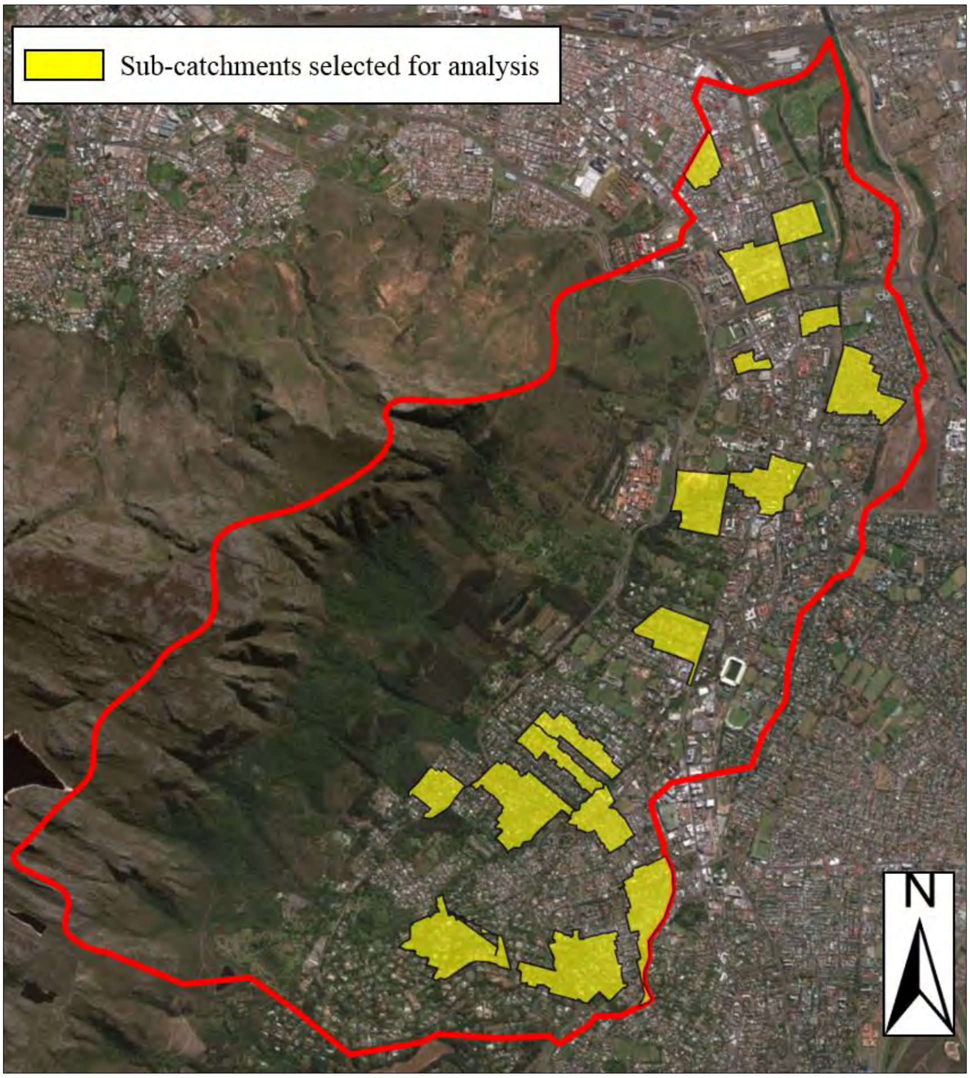

Figure 4-17: Subcatchments selected for analysis of dual reticulation costs

The same approach was repeated at the whole catchment scale, which had a mean density of 5.23 connections per hectare. The results indicated a mean cost of ZAR 21,000.00 per connection. This is 4.2% more than the ZAR 20,150.00 that would have been estimated using Figure 4-18.

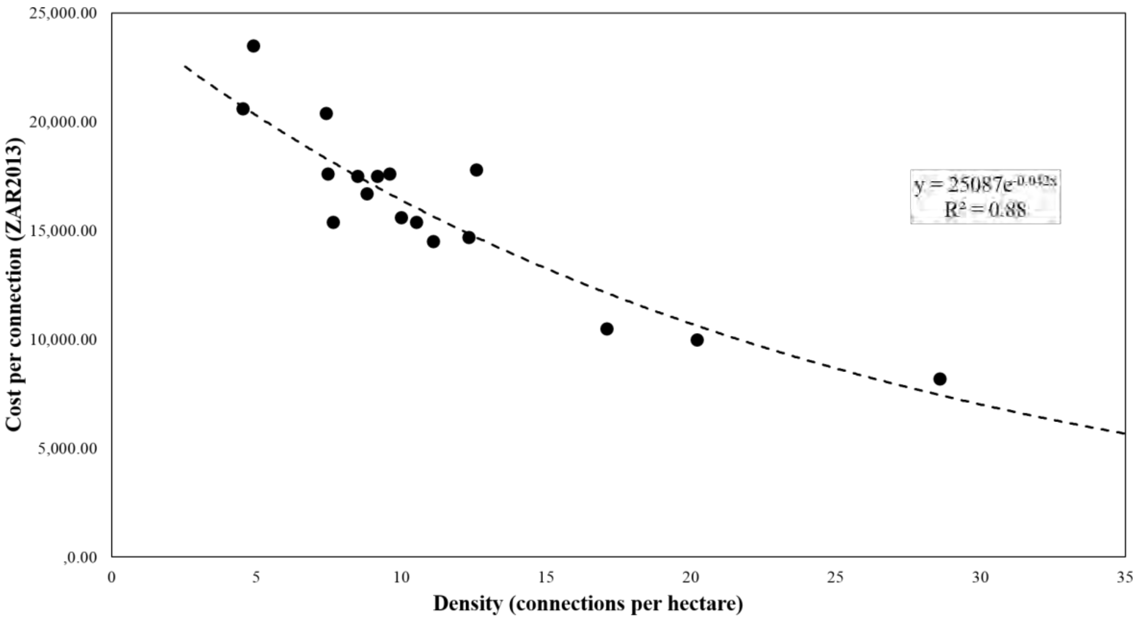

Figure 4-18: Cost versus density of a dual reticulation network

4.2.7.3 Maintenance

RWH and SWH systems require ongoing maintenance in order to ensure the successful short-term operation and system performance. Maintenance typically comprises inspections, routine maintenance and corrective maintenance (when systems are damaged or fail) (Lampe *et al.*, 2005). Importantly, as presented in, amongst others, Lampe *et al.* (2005) the level of maintenance affects the cost of RWH and SWH over the system's life cycle. How maintenance was accounted for in this research is briefly discussed below.

The frequency of inspections, routine maintenance and corrective maintenance was largely based on consultation with officials from the CoCT's stormwater branch (Austin, 2012) and experts working in the industry, and supplemented by work done by Lampe *et al.* (2005) – who completed an extensive study on the maintenance requirements of different SuDS options. Lampe *et al.* (2005) present estimates of maintenance requirements for the USA and UK separately. The estimates for the USA were considered in conjunction with the above mentioned local expert advice.

The cost of maintenance was based on a CoCT stormwater maintenance cost study (Austin, 2012), an infrastructure asset management guideline by the RSA Department of Provincial and Local Government (DPLG, 2009), costs reported in the CoCT's 'Water Services Development Plan' (CoCT, 2011b), and cost estimates provided by suppliers (e.g. replacement cost of a UV bulb for a domestic UV water filter).

4.2.7.4 Price adjustment

The value of money changes over time as a result of, *inter alia*, inflation – as highlighted in Section 2.6.6. Therefore where cost data from different years is to be used together, it is necessary to adjust all cost data to its effective value at a specific point in time. The price data for RWH were collected between May and August 2013, and so no adjustment to the collected data were undertaken – since any adjustment would be negligible. The price data for SWH were, however, collected over a period of years. It was therefore necessary to adjust the cost data to equivalent values at a specific point in time. This was accomplished using Contract Price Adjustment Formulae (CPAF) – specifically the CPAF factors for 'Water and Sewerage Reticulation' and 'Earthworks (with Culverts and Drainage)' (SAFCEC, 2014), which are most appropriate for stormwater infrastructure – as typically used for engineering projects in the RSA. All prices were adjusted to July 2013.

4.2.7.5 Life cycles

Appropriate estimates of life cycles for different components and systems were derived from discussions with experts working in the industry and those found in the economic fact sheets in the South African Guidelines for Sustainable Drainage Systems (Armitage *et al.*, 2013), which made use of numerous sources from the literature. These fact sheets were developed by this author.

4.2.8 Summary: Data collection and processing

The data collection process resulted in over 500 Gigabytes of raw data that was subsequently used to analyse the viability of RWH and SWH in the Liesbeek River Catchment. The methods of processing the data, described above, provide a means of overcoming some of the data gaps in the RSA. Of most relevance in this research was the generation of indoor demand data, dual reticulation cost estimates and data on roof areas.

The methods developed for collecting and processing the data may be used in situations where data is limited (e.g. elsewhere in the RSA / developing countries), rather than relying on data / studies from outside the RSA. These methods make use of commonly available software packages (*Excel*, *ArcMap*, *SWMM*, *EPANET*) which are widely used in the RSA, are cost effective, and for which there is adequate support, – see Section 2.6.1 (selection of modelling tools).

4.3 Catchment stormwater model

A stormwater master planning study for the Liesbeek River Catchment was completed for the CoCT in 2012. As part of the stormwater master planning study, a *SWMM* model of the catchment was 'calibrated' for the peak flow of two single storm events. 'Calibration' resulted in the model peak flows varying from 18 to 46% higher than the observed floods; the higher values being produced by the SCS Type 1 rainfall. This was attributed to the shape of the observed rainfall in comparison to the smoothened SCS curve. The stormwater master planning study notes that the effect can be reduced by increasing the catchment size.

The model was not appropriate for this research for a number of reasons, including, *inter alia*:

- The model was not set up for continuous modelling.
- The model did not consider base flow.
- For the purposes of this research, the subcatchments were not detailed enough. The reasons for this included, *inter alia*, the subcatchments were too large; there were errors in the delineation; and catchment boundaries ran indiscriminately through properties. For the RWH analysis it was necessary to identify which catchment each property formed part of.
- Certain catchments were incorrectly routed (e.g. flowed uphill – clearly incorrect).
- The calculation of imperviousness was based on typical land-use characteristics and so did not necessarily represent local conditions. While this may be acceptable for a high-level planning model, it does not allow for the analysis of on-site technologies at a finer resolution.
- Additionally, input parameters such as the catchment slope and flow directions were based on old contour data which had become outdated due to the subsequent collection of LiDAR data (only available subsequent to the completion of the previous model).

The model did however have some useful basic data which was collected as part of the completed stormwater master planning study. This included: pipe diameters, channel dimensions, invert elevations, and the Liesbeek River catchment boundary.

Due to the shortcomings – for use in this research – in the above mentioned *SWMM* model of the Liesbeek River, it was necessary to overhaul the model. This included, *inter alia*, the following:

- Determining dry weather flows in order to be able to set up a continuous stormwater model.
- Re-delineating the Liesbeek River Catchment into subcatchments appropriate for this research – as discussed in Section 4.3.1.
- Making use of the data captured in Section 4.2 (roof areas, road areas, parking areas etc.), to estimate the level of imperviousness.
- Making use of the DEM, discussed in Section 4.2.2, to estimate the average slope for each subcatchment.

It was decided to continue to use *SWMM*, in the form of *PCSWMM*, to model stormwater runoff in the catchment, as *SWMM* is freely available, widely used in the RSA and internationally recognised as a standard for modelling stormwater runoff. *PCSWMM* was selected owing to the fact that, whilst fully compatible with *SWMM,* it has additional analysis functionality which assists in calibration of the developed model. The following sections detail the development and calibration of the Liesbeek River Catchment stormwater model (CSM).

4.3.1 Subcatchment delineation

Subcatchments within the Liesbeek River Catchment were delineated manually as they need to take into account the pipe network, local catchment topography and road network. DEM (Section 4.2.2), stormwater, sewer and roads networks were overlaid in ArcGIS. This resulted in the delineation of the Liesbeek River Catchment into 171 subcatchments – Figure 4-19. Within the urban areas, subcatchments were typically between 1 and 20 hectares, depending on the density of development, establishment of a stormwater network, etc. Outside of the urbanised areas (e.g. on the mountain), the subcatchments typically ranged from 20 to 40 hectares.

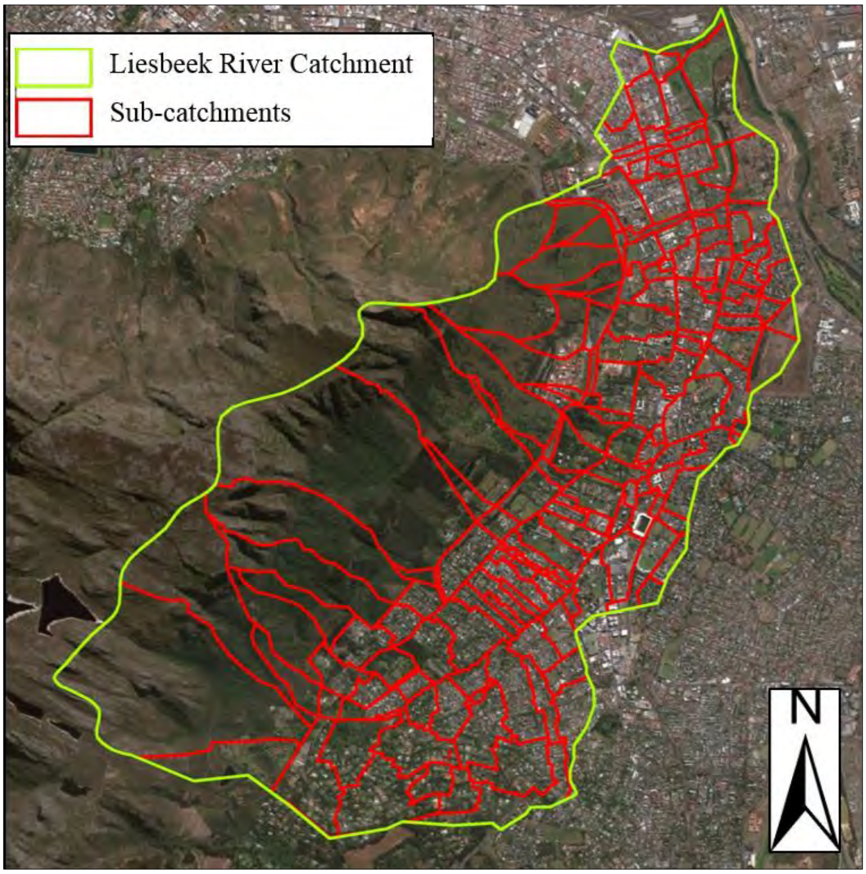

Figure 4-19: Delineated subcatchments

4.3.2 Stormwater conveyance network

The stormwater conveyance system was modelled, building upon previously developed stormwater models, to include all stormwater pipes with a diameter equal to or greater than 300mm. This was done using data provided by the CoCT (CoCT, 2009c). In some cases, attribute data (e.g. cover levels, invert levels, pipe diameters) were missing. Missing data were patched using the methods presented in Table 4-6.

Table 4-6: Conveyance system data patching

Missing data	Approach to patching
Conduit / pipe diameter	The downstream pipe / conduit diameter was assumed to apply.
Conduit length	All conduit lengths were recalculated using the GIS functionalities of *PCSWMM*.
Flow direction	*PCSMM* used to automatically calculate flow direction.
Manhole cover level	All cover levels were estimated from the DEM to account for uncertainty as to the datum used for height measurements. (Currently mean sea level is used as the datum but, historically, other data have been used. Since the DEM provides measurements within 0.1m, it was felt this would limit the uncertainty.)
Manhole depth / invert elevation	The DEM was used to estimate the cover level. The manhole was assumed to be 1.6m deep and then, using *PCSWMM* functions, it was adjusted to ensure the up and down stream pipes met the minimum gradient / slope requirements.

4.3.3 Dry weather flow

Using *PCSWMM*'s monthly and daily 'pattern' analysis tool, estimated monthly dry weather flow (Table 4-7) was calculated from the observed flow data for days without precipitation. The dry weather flow included groundwater inflows, infiltration and inflows from the stormwater sewer network – which was observed to include swimming pool backwash water, car washing, runoff from cleaning at construction sites, and intermittent discharges for industry (South African Breweries) located on the banks of the river.

The calculated flow was weighted across the nodes used to model the main Liesbeek River channel in *SWMM*, and assigned to the nodes as external inflows.

Table 4-7: Average Monthly dry weather flow in the Liesbeek River at Lies03hS

Month	Flow (m^3/s)	Month	Flow (m^3/s)	Month	Flow (m^3/s)	Month	Flow (m^3/s)
January	0.246	April	0.333	July	0.797	October	0.583
February	0.112	May	0.313	August	1.004	November	0.374
March	0.081	June	0.869	September	1.127	December	0.345

4.3.4 Assigning attributes

In order to model the runoff from a catchment, it is necessary to provide a range of parameters including, *inter alia*, area, slope, land use, soil type, impermeable area and depression storage. These parameters, and how their relevant values were estimated, are described in Table 4-8.

Table 4-8: Parameters for modelling runoff

Parameter	Derived by
Slope	Digital Elevation Model (Section 4.2.2)
Area	Subcatchment areas were calculated using *PCSWMM's* GIS capabilities.
Soil type and infiltration parameter	Descriptive information on the types of soil found in the catchment was available from DAFF (2013). Using this information, initial estimates for infiltration (using Green & Ampt infiltration model) were assigned. These were subject to calibration, but the model was found to be insensitive to changes in these parameters – discussed in Section 4.3.5.
Impermeable area / land use data	Based on the results of data collection and processing. Described in Section 4.2.1.
Depression storage	Each land use category was assigned an estimated depression storage based on typical values in the literature. Using *PCSWMM*'s area weighting tool a catchment composite depression storage was calculated. Typical values are included in Appendix G.
Subcatchment width	Subcatchment length was estimated as the subcatchment's effective flow path length. The subcatchment width was calculated by dividing the subcatchment area by the estimated subcatchment length.
Manning's N for overland flow / pipes	Based on values used in SRK (2012), and general recommendations in Rossman (2008). Estimated parameters were subject to calibration, but the model was found to be insensitive to changes in these parameters – discussed in Section 4.3.5.
Subcatchment outlet	Assigned to the stormwater manhole nearest the lowest point of the catchment.
Subarea routing	Impervious to pervious.
Percentage routed (Indicates – in this case – the percentage of impervious area routed to pervious areas)	Based on calibration. Initial estimate considered 25% of impermeable surfaces were routed to pervious surfaces.
Rainfall	Based on the results of data collection and processing. Described in Section 4.2.3.2.
Evaporation	Based on the results of data collection and processing. Described in Section 4.2.3.3.
Base flow	Base flow was estimated from historical records within the Liesbeek River. Described in Section 4.3.3.

4.3.5 1D calibration

As with most models, it is essential for stormwater models to be calibrated (James, 2005). This research was primarily interested in the potential to harvest stormwater for reuse, and the potential benefits (such as peak flow attenuation) that may result from stormwater harvesting. The CSM needed to accurately represent the total storm runoff volume (water available for harvesting) and ensure that the modelled peak flows were reasonable and reflective of the range in the observed data. Therefore, it was decided to focus calibration on total runoff and only then, as a secondary interest, on peak flows.

The observed flow data was analysed and a total of 35 storm events were identified. In line with other studies (e.g. Ashbolt *et al.*, 2013; Mancipe-Munoz *et al.*, 2014), two-thirds of the available data were used for calibration (30 events), and one-third used for verification (14 events). The calibration of the model, however, posed a significant challenge due to:

i) The limited flow gauging information (one station) – Section 4.2.6.

ii) The disaggregation of the rainfall data had resulted in a time series that was not necessarily a representation of what occurred over the calibration period, but rather, based on statistics, a realistic approximation of the type of pattern that could be expected to have occurred – discussed in Section 4.2.3. This posed a challenge to calibrating the model, particularly with respect to peak flow.

iii) The limited availability of infiltration (soil properties) data – Section 4.2.1.

Calibration parameters were identified from recent studies – e.g. Mancipe-Munoz *et al.* (2014) and James (2005). *PCSWMM*'s 'SRTC calibration tool' was used to assess the sensitivity of each parameter. This tool uses a known uncertainty percentage and completes two model runs, one for each extreme high and low percentage of the selected uncertainty range. *PCSWMM* linearly interpolates between the two extreme values for the parameter being tested. This allows for an assessment of each parameter's sensitivity, and for the calibration of the model. The more parameters considered simultaneously, the less certainty there is in the SRTC predicted output. However the model can be re-run to verify the results.

In terms of total runoff the model was most sensitive to changes in (in order): the percentage impervious area; percentage routed to pervious; and depression storage (pervious and impervious). The model was least sensitive to changes in (in order): the Manning's coefficient (subcatchment and conduits); catchment slope; and catchment width. In terms of peak flow the model was most sensitive to changes in (in order): Percentage impervious area; percentage routed to pervious; catchment width; and Manning's coefficient (pervious and impervious area). The model was least sensitive to changes in (in order): Manning's coefficient (conduits); and Green and Ampt infiltration parameters. The total volume of runoff and peak flow were also sensitive to changes in the percentage of impervious area with no depression storage, particularly for smaller events. However, just because a model is sensitive to a change in parameter does not necessarily mean the parameter is incorrect or needs to be adjusted. It was therefore decided that the catchment slope and percentage of impervious area would not be subject to calibration for the following reasons:

i) The DEM (Section 4.2.2) was considered to be accurate as the processing of the LiDAR data resulted in a surface DEM which was not affected by distortions from buildings, bushes etc. The average catchment slope was calculated using an area weighting tool in ArcGIS which ensured that it was accurately calculated.

ii) Significant time was invested in collecting accurate data for percentage of impervious area (Section 4.2). Additionally, for reasons discussed in Section 4.4, it was important that the total impervious area remain as captured during the data collection process. It was also considered more likely that the percentage routed to pervious was a better candidate for calibration since it was unknown.

The calibration parameters ultimately selected were Manning's coefficient for impervious area (N Imperv), Manning's coefficient for pervious area (N Perv), depression storage for impervious area (Dstore Imperv), depression storage for pervious area (Dstore Perv), percentage routed to pervious (Percent Routed), Green and Ampt infiltration parameters; and catchment width (Width).

Once the model was calibrated for total runoff, it was calibrated for peak flow. This was done in two steps:

i) Seven events were selected where the disaggregated rainfall data (hyetograph) reflected a similar pattern of peaks to the runoff hydrograph of the observed data. While a hyetograph is not equivalent to a hydrograph, the two are related. It is reasonable to assume that if a hyetograph has a pattern of peaks, that the hydrograph would have a similar pattern – although the relative magnitude may vary. These seven events were used to fine tune the calibration of the peak flows.

ii) The peak flows were assessed to ensure they typically fell within the range of peak flows in the observed data.

Once the model was calibrated all parameters were checked to ensure they were within an acceptable range. Depression storage was found to exceed the typical range (reported in James, 2005)) in a number of catchments. This was expected, and considered reasonable due to the significant tree cover found in the catchment. As highlighted in a number of sources, trees typically offer more than 1 mm of interception storage, and over 10mm interception storage for specific storms (Xiao *et al.*, 1998, 2000; Xiao & McPherson, 2002; Guevara-Escobar *et al.*, 2007). This interception storage is partly accounted for in the depression storage in the CSM, since SWMM does not explicitly model interception storage.

Summary results of the calibration are presented in Table 4-9, Table 4-10, Table 4-11 and Appendix H. The typical runoff continuity error (<0.1%) and routing continuity error

(<0.5%) were deemed acceptable. The results in Table 4-9, Table 4-10 and visual inspection provide reason for confidence in the model.

Table 4-9: Calibration of total runoff

Error function	Observed vs. Calibrated	Observed vs. Verified
Integral Square Error	2.41	6.45
Integral Square Error rating*	Excellent	Good
Nash Sutcliff Efficiency	0.99	0.956
R^2	0.992	0.983

*Shamsi (1997)

Table 4-10: Calibration of peak flow

Error function	Observed vs. Calibrated
Integral Square Error	4.03
Integral Square Error rating	Very Good
Nash Sutcliff Error	0.942
R^2	0.959

* Shamsi (1997)

Table 4-11: Calibration of hydrograph

	Hourly min.		Daily	
Error function	Observed vs. Calibrated	Observed vs. Verified	Observed vs. Calibrated	Observed vs. Verified
Integral Square Error	1.46	1.09	2.8	3.55
Integral Square Error rating*	Excellent	Excellent	Excellent	Very good
Nash Sutcliff Error	0.487	0.653	0.853	0.719
Nash Sutcliff Error rating **	N/A	N/A	Good	Acceptable

* Shamsi (1997) ** Moriasi *et al.* (2007)

4.3.6 2D catchment stormwater model

RWH and SWH, as discussed in Section 2.2, may offer benefits including the attenuation of peak flows. Larger storm events typically take place after a day or more of rainfall, which would result in all storage (RWH and SWH) being full. Whilst it was suggested in Section 2.6.9 that RWH and/or SWH might attenuate peak flows, it was uncertain what the risk associated with flooding – defined in the CoCT's '*Floodplain and River Corridor Management Policy*' (CSRM, 2009a) as a function of the velocity and the depth of flood water – would be.

In order to assess whether there were any changes in the risk associated with the flooding it was decided to make use of *PCSWMM*'s two-dimensional (2D) modelling capabilities. It also allowed for a more detailed assessment of the flooding. While other modelling software also offers 2D modelling, the 1D model had been developed in *PCSWMM,* and so it made the process of converting to 2D modelling simpler. The process for undertaking 2D modelling in *PCSWMM* is detailed in CHI (2014) and the basic input parameters described in Appendix I.

The calibration of the 2D stormwater model was again hampered by limited data. Nevertheless, between 2003 and 2012 there were two major flooding incidents, one on the 4th of August 2004 and another on the 12th of July 2009. These two events were documented (photographed) by an employee of Aurecon Consulting Engineers – then Ninham Shand (Whittemore, 2014). It was concluded that the 2004 flooding '*was not caused by high flow rates in the Liesbeek River itself but rather by a back-up of flood water in the Salt, Black and Liesbeek Rivers which occurred as a result of the limited capacity of the Salt River canal*' (Whittemore, 2005). This meant that the 2004 flooding event could not be used for calibration – unless the Salt, Black and Liesbeek catchments were also modelled. Fortunately, unlike the 2004 flooding, the 2009 flooding resulted, in part at least, from significant rainfall in the Liesbeek River Catchment. It was therefore decided to make use of information from the 2009 flooding event to calibrate the 2D model.

Many photographs showing the extent of the 2009 flooding, including debris / water marks, were obtained from Whittemore (2009). Using these photos, flood levels were estimated at different points in the flood plain. Most of the photos focused on buildings at the River Club – see Appendix J. Four potential calibration points were identified from the photographs, but on inspection, only one was suitable. The other three points were rejected because they were in localised low points and the 2D modelling would not be able to account for them.

The model was run with three sets of rainfall data: disaggregated rainfall data; SA-SCS Type 2 rainfall data, using the actual daily records for scaling the SCS hyetograph; and 50-year recurrence interval SCS Type 2 rainfall data based on the CoCT's design storm parameters. The results for the calibration point are shown in Table 4-12.

Table 4-12: 2D model calibration results

Scenario	Maximum depth at calibration point (m)
Actual maximum depth (as measured from photographs)	≈ 0.5
Disaggregated rainfall data	≈ 0.42
SCS Type 2 rainfall data using the actual daily records	≈ 0.53
SCS Type 2 rainfall data based on the CoCT's design storm parameters (100yr Recurrence Interval)	≈ 0.67

Considering the uncertainty inherent in the input data, especially rainfall data, the impact of downstream controls and errors in estimating measurements from photographs, the results reflected in Table 4-12 nonetheless suggest that the 2D catchment models using SCS Type 2 with actual data and disaggregated rainfall data together provide a reasonable set of results for assessing the potential impacts of RWH/SWH on flooding and associated risks.

4.3.7 Water quality

Section 4.2.5 highlighted that the available water quality data is of interest, but not adequate for calibrating water quality models. It was also decided that using EMC's from the literature – outside of the RSA – would offer little benefit. Therefore in this study it was decided that there was no benefit in modelling the water quality for the following reasons:

i) A model capable of reasonably accounting for the complexities associated with runoff water quality would require continuous water quality data which was not available for this study.

ii) While the Event Mean Concentration (EMC) approach has been used internationally, there is very limited water quality data available in the RSA and no data that could be used to guide the inputting of reasonable estimates. In particular there are no data, nor studies, of the Liesbeek River catchment that could be used for calibration.

iii) The modelling of EMC's does not provide insight into the runoff quality during different recurrence interval storm events nor does it provide any insight into the changes in runoff quality at different times of the year.

iv) In order to develop an informative water quality model, other than one based on EMCs, it would be necessary to have multiple water quality monitoring points that measure water quality through a number of storms. This sort of data is not available; nor are there resources to collect such data.

In terms of this research, the most important question relating to water quality is to what degree runoff will require treatment. The available water quality results (Sections 4.2.5 and Appendix F) clearly indicate high levels of *E.coli* and suspended solids, which indicate the need for filtration and disinfection. Lim *et al.* (2011), as highlighted in Section 2.5.1, showed that, by making use of alternative stormwater management practices (e.g. WSUD/SuDS) and urban planning, it is possible to ensure relatively high-quality urban runoff. However, in order to manage risk in RWH, it was decided that, in line with Mitchell *et al.* (2007a), harvested water for use indoors would need disinfection (UV disinfection) – and this would require the water to be filtered to reduce the suspended solids. Furthermore, since the other water quality parameters (Sections 4.2.5 and Appendix F) were within acceptable levels for non-potable end uses, it was felt that modelling water quality, without any form of calibration, would be unhelpful.

4.4 Urban rainwater / stormwater harvesting model (*URSHM*)

There are currently a number of models and software packages available for evaluating RWH and SWH systems. However, it was decided that in order to reasonably represent the impacts of RWH and SWH have on water demand and runoff characteristics it would be necessary to develop a new model, termed the '*Urban rainwater / stormwater harvesting model (URSHM)*', for assessing the viability of RWH and SWH in the Liesbeek River Catchment, for the following reasons:

i) The available models and software packages (highlighted in Section 2.6.1) either assume linear upscaling (e.g. *UVQ*), or require the modelling of every property individually, in an unnecessarily time-consuming manner that would require excessive computing power, in order to represent RWH/SWH (e.g. *SWMM / MUSIC / Urban Developer*); or else they make use of statistical distributions for different modelling parameters, for example Mitchell *et al.* (2008a) – which were not available for the Liesbeek River catchment nor the RSA.

ii) To date, there has been limited uptake of RWH for domestic purposes in urban areas of the RSA, while SWH remains a new concept. There are limited data available as to typical storage sizes used and the relationships between water demand, roof area / catchment and optimum storage size for RWH and SWH systems in the RSA. Therefore, the sizing of optimal storage units for RWH and SWH systems has to be undertaken on a case-by-case basis.

iii) Section 4.2 highlighted that authors such as Mitchell *et al.* (2008a) and Neumann *et al.* (2011) made use of values reported in literature from local studies and personal communication with local water industry employees to estimate the descriptive statistics (mean, minimum, maximum, standard deviation) of roof areas, depression storage, tank size and effective roof area factors. No local data of an equivalent nature exists. Section 2.4.2.1 showed, for example, how roof areas varied between different cities in Australia, and what is considered reasonable in the RSA. It would be inappropriate to use data imported from outside of the RSA.

The *URSHM* is designed to size RWH and SWH systems based on a number of different rational design considerations, as follows:

- Allow for different end uses and treatment options to be considered;
- Identify the optimum design from a number of proposed designs;
- Estimate individual and catchment scale volumetric reliability of RWH and SWH systems;
- Estimate the reduction in runoff volume;
- Estimate the economic costs and, where possible, the benefits of RWH and SWH; and

- Estimate the expected variation in each variable at different levels of adoption of RWH/SWH.

The *URSHM* combines many well-known techniques such as life cycle costing and behavioural analysis modelling, with techniques developed in this research – such as the approaches to optimisation – and applies them in a manner that has not been previously done. The model was developed in a macro-enabled Excel 2014-workbook, due to the ease with which developing and altering are possible.

An overview of the *URSHM* is provided in Figure 4-20. The model contains five main calculation modules: runoff module, storage module, life-cycle costing module, optimisation module and results module. Each module requires input data as highlighted in Table 4-13, and which was collected as explained in Sections 4.2 and 4.3. Each module is discussed in the following sections.

Table 4-13: *URSHM* input requirements

Module	Input data	Applicable to
Runoff	Roof area	RWH
	Runoff coefficient	RWH
	Depression storage	RWH
	Evaporation	RWH/SWH
	Rainfall	RWH/SWH
	Runoff time series	SWH
Storage	Water demand	RWH/SWH
	Proposed alternative systems (storage sizes)	RWH/SWH
Life Cycle Costing	Discount rate	RWH/SWH
	Expected useful life for each component	RWH/SWH
	Maintenance costs and frequencies	RWH/SWH
	Electricity demand	RWH/SWH
Analysis and Optimisation	Chosen analysis method (YAS/YBS)	RWH/SWH
	Time step	RWH/SWH
	Maximum retention days	RWH/SWH
	Days	RWH/SWH
Results	Sensitivity analysis parameters	RWH/SWH
	Monte Carlo simulations	RWH/SWH

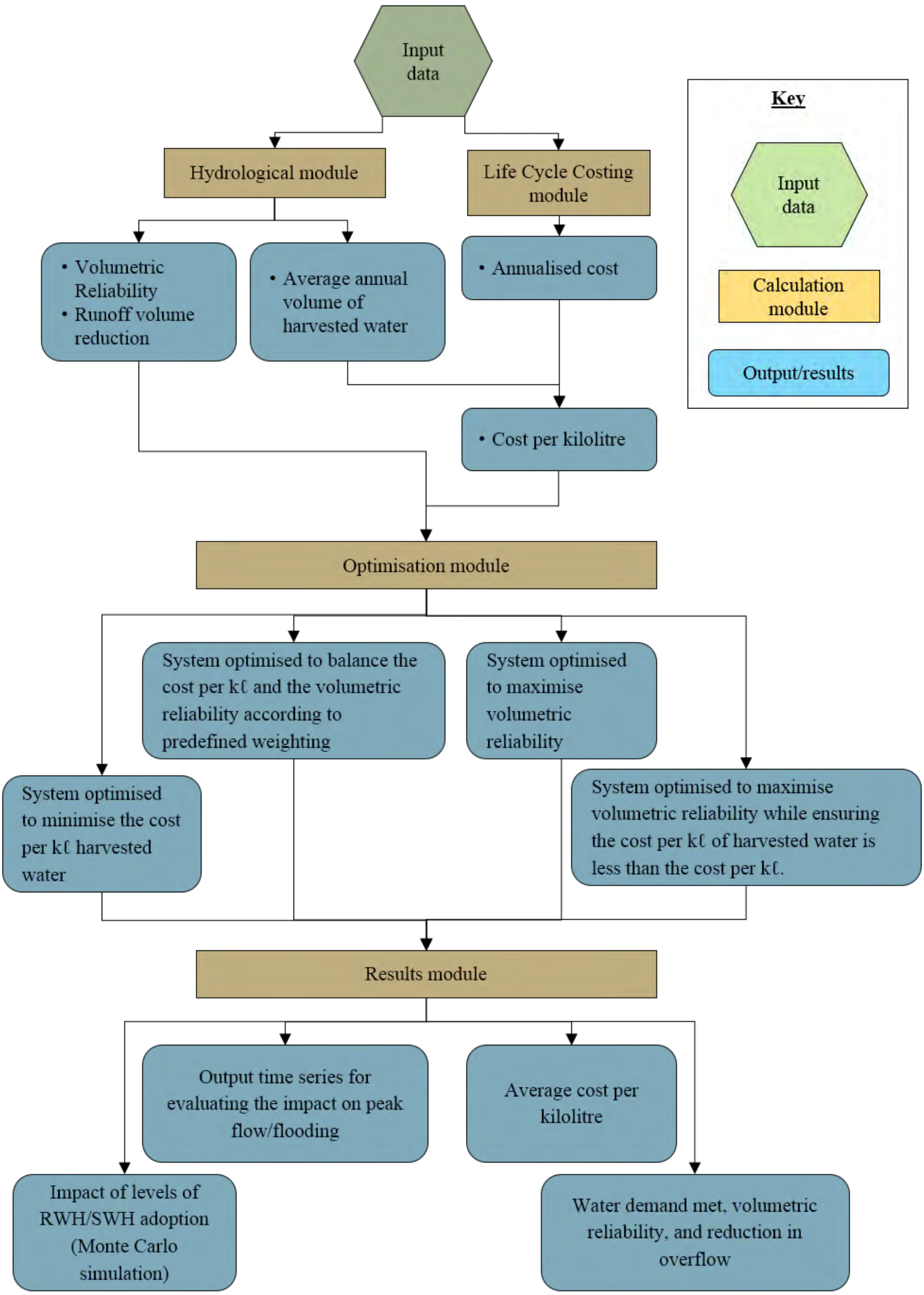

Figure 4-20: General overview of the urban rainwater / stormwater harvesting model

4.4.1 Roof runoff module (RWH)

Section 2.4.5 discusses three different models for simulating the runoff from roofs and inflow into RWH system storage tanks. Section 2.4.5 concludes by showing that, unless the use of a first-flush filter is expressly modelled, there is no difference between the model used by Roebuck (2007) and Mitchell *et al.* (2008a). Roebuck (2007) does not simulate the use of a first-flush filter because the '*use of first flush devices is limited in the UK'*. It was, therefore, decided to use a modified version of Mitchell *et al.* (2008a), which uses a runoff coefficient rather than an 'effective roof area loss factor'. The relationship between the runoff coefficient and roof area loss factor is shown in Equation 4-9.

$$C_R = 1\text{-}L_c \tag{4-9}$$

Where: C_R is the runoff coefficient and L_C is the effective roof area loss factor (%)

The roof is modelled as illustrated in Figure 4-21. The depression storage / initial losses (RC_i) are represented as 'storage', which once exceeded will result in runoff. The depression storage is filled through rainfall and emptied through evaporation (drying), as calculated for each time step. This process is calculated using Equations 4-10 and 4-11. The runoff (as depth (mm) of runoff per time step) is determined by subtracting the depression storage. This is converted to a runoff volume (I_t) (m^3) by multiplying the runoff depth by area (A_T) and a runoff coefficient (C_R) – Equation 4-12. The runoff coefficient, in line with Mitchell *et al.* (2008a), accounts for continuing losses, for example, due to splashing and gutter overflow.

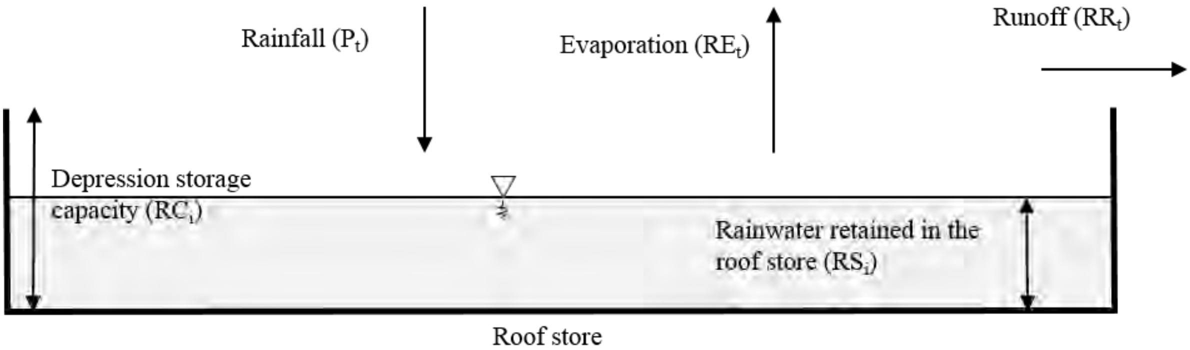

Figure 4-21: Roof runoff model

$$RR_t = \text{Max}\begin{cases}(P_t + RS_{t-1} - RC_i)\\ 0\end{cases} \tag{4-10}$$

$$RS_t = \text{Max}\begin{cases}(P_t + RS_{t\text{-}1} - RR_t - RE_t)\\ 0\end{cases} \quad \textbf{4-11}$$

$$I_t = \frac{A_T \times RR_t \times C_R}{1000} \quad \textbf{4-12}$$

Where RR_t = Runoff (mm); P_t = Rainfall (mm); $RS_{t\text{-}1}$ = Rainwater retained in the roof store from previous time step (mm); RC_i = Depression storage (mm); RE_t = Evaporation during time step (mm); A_T = Catchment / roof area (m^2); C_R is the runoff coefficient; and I_t a runoff volume per time step (m^3/s).

4.4.2 Runoff from an urban catchment (SWH)

The approach that was adopted for RWH could not be used to model runoff for SWH owing to the complexities in accurately modelling runoff from an urban catchment that includes pervious and impervious areas. Section 2.5.5.1 provides an overview of the different approaches and complexities related to modelling runoff from an urbanised catchment. It was decided to make use of an established model, in this case *SWMM* (selected for reasons discussed in Section 4.3), to model the runoff. The time series output of the runoff from each subcatchment, generated by the CSM, was used as an input into the *URSHM*. This time series was considered as the potential inflow into the SWH storage. In terms of the *URSHM*, the runoff time series took the place of the I_t values, which are calculated when considering RWH. Essentially, this approach makes use of *SWMM*'s rainfall-runoff modelling capabilities to provide an estimate of the volume of potentially harvestable runoff.

4.4.3 Storage

The *URSHM* is capable of modelling open and closed storage units, but not managed aquifer recharge (MAR) – see Section 2.5.3. While MAR is considered a relatively cheap option for SWH, it was not an option in the Liesbeek River Catchment due to the steepness of the upper catchment, the levels of development present within the catchment, limited knowledge relating to groundwater conditions, and the risk of polluting aquifers that are currently (or plan to be) used to supply municipal potable water – e.g. Albion Springs (CoCT, 2011b). Therefore, this research only considered open and closed storage. In principle, the reduction in water demand would not change if an alternative storage option were utilised; however, costs and potentially the base flows in the Liesbeek River (as a result of changes to the water table) could change (they would be expected to increase during the wet season and decrease during the dry season).

As noted in Section 2.4, with respect to RWH, the cost of underground closed storage (e.g. tanks or vaults) is significantly more than aboveground storage in the RSA. Thus, the modelling (especially cost) of RWH systems assumed an aboveground tank as this would be the most economical approach. With respect to SWH, it was assumed that the storage unit was an open storage (e.g. retention pond), as again, it was more economical than a below-ground, closed storage unit. 'Typical' RWH and SWH systems are provided in Section 4.5.

4.4.3.1 Modelling

Section 2.6.3 discusses in detail the modelling of storage units using a behavioural analysis approach. The *URSHM* adopts this well-known and widely used approach. For ease of reference, Figure 2-14 is repeated as Figure 4-22 and illustrates how the storage units are modelled. Equation 2-5 describes the balance of flows in each time step.

$$V_t = V_{t-1} + I_t + P_t - E_t - S_t - L_t - Y_t \qquad \textbf{2-5}$$

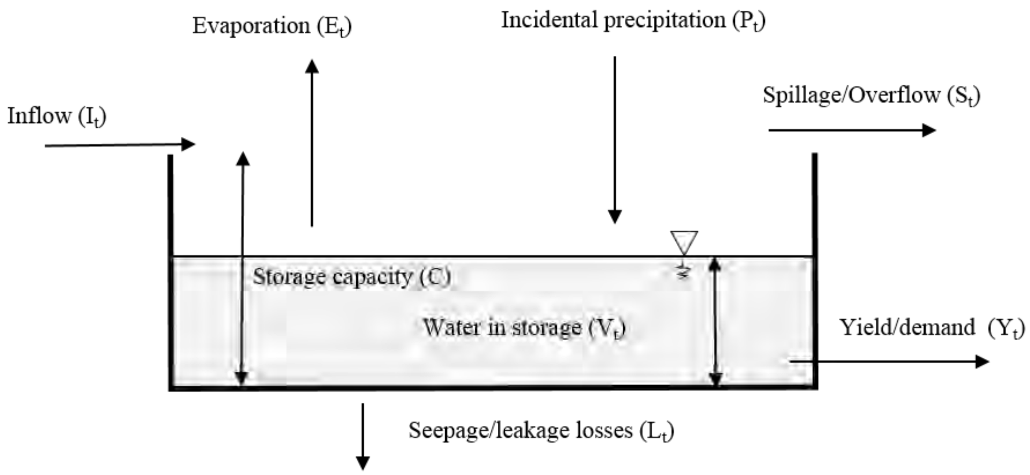

Figure 4-22: Storage tank configuration used in behavioural models
(After Mitchell, 2007; Roebuck, 2007; Mitchell *et al.*, 2008b)

Due to limitations in modelling simultaneous events, as described in Section 2.6.3, the fundamental algorithms – yield after spillage (YAS) and yield before spillage (YBS) operating rules – developed by Jenkins *et al.* (1978) to describe the operation of the storage unit – were included in the *URSHM*. When utilising the model, it was possible to select which algorithm was most appropriate. The YAS operating rule is described mathematically by Equations 2-6 and 2-7. The YBS operating rule is described mathematically by Equations 2-8 and 2-9. The volume of spillage is calculated using Equation 4-13.

$$Y_t = \min(D_t, V_{t-1}) \qquad \textbf{2-6}$$

$$V_t = \min(C_t - Y_t,\ V_{t-1} + I_t + P_t - E_t - Y_t) \qquad \textbf{2-7}$$

$$Y_t = \min(D_t,\ V_{t-1} + I_t) \qquad \textbf{2-8}$$

$$V_t = \min(C_t,\ V_{t-1} + I_t + P_t - E_t - Y_t) \qquad \textbf{2-9}$$

$$S_t = (V_{t-1} + I_t + P_t - E_t - Y_t - V_t) \qquad \textbf{4-13}$$

Where: V_t is the storage volume at the end of the current time step t (m^3), V_{t-1} is the storage volume at the end of the previous time step t (m^3) and I_t is the inflow from the catchment during time t (m^3) (See Sections 2.4.5.2 and 2.5.5.1 for a discussion as to how these values are calculated for RWH and SWH, respectively). P_t is incidental rainfall during time t (m^3). E_t is the evaporation from the storage unit during time t (m^3). S_t is the overflow / spillage during time t (m^3). L_t is seepage and/or leakage losses during time t (m^3), and Y_t is the yield / water demand during time t (m^3).

When modelling RWH, it was assumed that incident evaporation E_t, rainfall P_t and leakage losses L_t were zero as the storage units (tanks) were sealed and that any leakages would be identified by the user and fixed.

When modelling SWH in this thesis, for the reasons discussed in Section 2.5, only open storages were considered. Therefore, incident evaporation E_t and rainfall P_t had to be considered. Leakage losses L_t were considered to be zero, as the design assumed the installation of an impermeable liner.

4.4.4 Treatment and distribution

As noted in Sections 2.4.4 and 2.5.2, the modelling of pre- and post-storage treatment has no impact on the hydraulics but does impact on the capital and operating costs of a RWH/SWH system. The specific costs depend on the scenario that is being analysed and are discussed in Section 4.5 and the related appendices.

4.4.5 Life-cycle costing module

The *URSHM* allows for the entry of life-cycle costing (LCC) data for a range of alternatives. The *URSHM* then uses this to undertake a full LCC analysis. This is undertaken for each alternative system. An overview of the *URSHM*'s LCC module is shown in Figure 4-23.

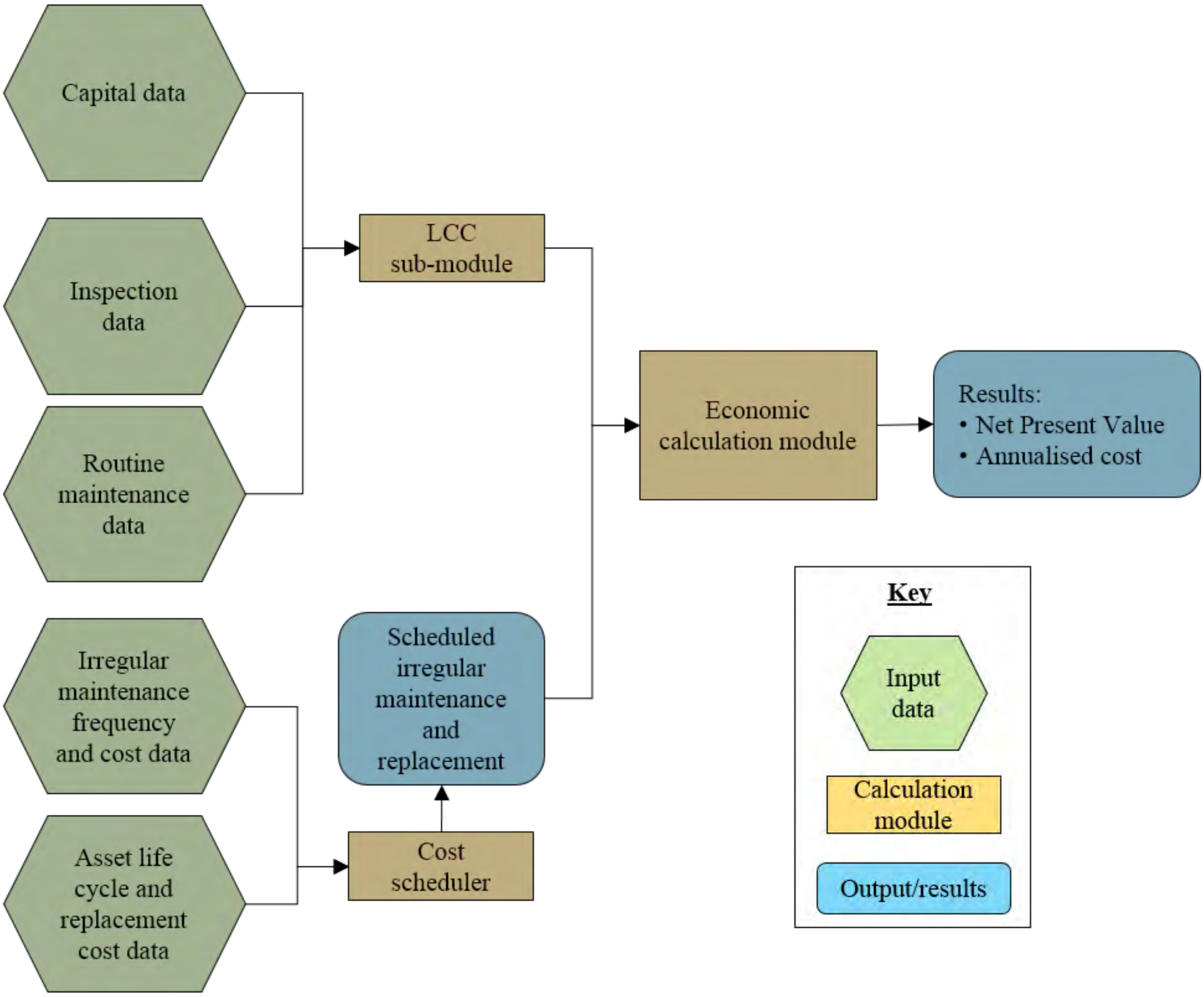

Figure 4-23: Overview of the *URSHM*'s LCC module

4.4.5.1 Economic calculation module

The Economic Calculation Module (ECM) converts all values (future costs) to their present value and calculates the Net Present Value (NPV) and Annualised Cost for each alternative. The ECM shown in Table 4-14 is based on that used in the WERF spreadsheets (Lampe *et al.*, 2005) and adaptions by this author (Fisher-Jeffes, 2011). A discount factor was calculated using Equation 4-14 and entered into Column 2. All 'future costs / values' are converted to their present value (PV) using Equation 4-15 and entered into Columns 6 through 11. The NPV is, in Equation 4-16, the sum of all the PVs in Columns 6 through 11. The annualised cost is calculated using Equation 4-17.

$$Df = \frac{1}{(1+i)^n} \qquad \textbf{4-14}$$

$$PV = Df \times FV \qquad \textbf{4-15}$$

$$NPV = \sum PV \qquad \textbf{4-16}$$

$$AC = \frac{i(1+i)^n PV}{(1+i)^n - 1} \qquad \textbf{4-17}$$

Where: Df = Discount factor; PV = present value; FV = future value; NPV = Net Present Value; AC = Annualised cost; I = discount rate (annual); n = number of years.

4.4.5.2 Discount rate

'*The discount rate is the rate used to convert all future costs and benefits to present value so that they can be compared'* (Lampe *et al.*, 2005). It is considered to be the difference between the rate of return on the open market and inflation. Selecting an appropriate discount rate is an extremely important and contentious issue (Lampe *et al.*, 2005). In the end, the decision is dependent on the person or institution undertaking the analysis (Roebuck, 2007) and may be dictated by the industry or country in which the analysis is being undertaken. For this thesis, the following was taken into account:

- In the RSA, the national treasury does not prescribe the discount rate; however, the treasury states: *'For practical purposes, the discount rate is assumed to be the same as the risk adjusted cost of capital to government. The government bond yield has been used by some institutions as the discount rate for a particular project over a comparable period. The argument in favour of using the government bond yield is that it reflects the actual cost to government of raising funds at any given time. This ignores a number of factors that are difficult to quantify, including: various risk margins relating to increased government borrowing; various tax implications of diverting funds from private to public consumption; and government's time preference of spending'* (National Treasury, 2004).
- As part of an 'Integrated Resource Plan', the RSA Department of Energy noted that good practice requires the consideration of a number of possible discount rates and, as part of the study, made use of a range of discount rates from 0% to 15% (DOE, 2010).
- The Construction Industry Development Board (CIDB) refers to the National Treasury definition when applying a discount rate.
- The CSIR (2005b) make use of a discount rate between 6.8% and 10% for road construction projects.
- In the UK, the discount rate is traditionally set by the national treasury (Lampe *et al.*, 2005).
- The USA Office of Management and Budget (2013) recommended a discount rate of 1.6% and 1.9% for projects of 20 and 30 years (or longer), respectively.

Table 4-14: LCCA Summary Costing Module

1	2	3	4	5	6	7
Year for Graphs	**Discount Factor**	**Capital and Replacement Expenditure**	**Establishment Costs**	**Inspections and Regular Maintenance**	**Irregular and Corrective Maintenance**	**∑ 'Cash' Expenditure for Year SuDS**
0	1.0000	395000	0	0	0	395000
1	0.9259	0	9600	10690	0	20290
2	0.8573	0	9600	10690	0	20290
3	0.7938	0	9600	10690	0	20290
4	0.7350	0	[redacted]	10690	0	10690
5	0.6806	0	[redacted]	10690	8900	19590

Table 4-14 (Continued)**: LCCA Summary Costing Module**

1	8	9	10	11	12	13	14
Year for Graphs	**PV of Capital And Replacement Expenditure**	**PV of Year Establishment Costs**	**PV of Inspections and Regular Maintenance**	**PV of Irregular And corrective Maintenance**	**PV of Years Costs–SuDS**	**Total Cash Expenditure-SuDS**	**∑ Present Value Expenditure– SuDS**
0	395000	0	0	0	395000	395000	395000
1	0	8888	9898	0	18787	415290	413787
2	0	8230	9165	0	17395	435580	431182
3	0	7620	8486	0	16107	455870	447289
4	0	[redacted]	7857	0	7857	466560	455147
5	0	[redacted]	7275	6057	13333	486150	468479

- In the USA, water utilities have traditionally used the average cost of borrowing less inflation as their discount rate (Lampe *et al.*, 2005).

The South African Bond Yields (10yr) and inflation (Consumer Price index) were, in line with the National Treasury (2004) recommendations, are compared (Table 4-15: RSA bond yields and inflation). Data on longer term bond yields was not available. It is worth noting that around 1997 inflation and bond yields were relatively high. Based on this it was decided that a discount rate of 3.1% would be used for this study. A sensitivity analysis between 3.1% and 4.5% was also undertaken for selected scenarios (See Section 4.5).

Table 4-15: RSA bond yields and inflation

Analysis period	Government 10 year bond (%)*	Inflation (%)**	Difference (%) – potential discount rate
1997-2013	10.4	6.0	4.4
2000-2013	9.0	5.8	3.1
2003-2012 (analysis period)	8.6	5.5	3.1

*Trading Economics (2014) **StatsSA (2014)

4.4.5.3 LCC sub-module

The LCC sub-module calculated and entered the data related to capital costs, inspections costs and routine maintenance (annual maintenance) directly into the ECM. The LCC sub-module calculates the total annual 'cash' cost for inspections and routine maintenance. These values are entered into Columns 3 and 4 of Table 4-14.

4.4.5.4 Sub-module for the scheduling of irregular maintenance and replacement

To avoid irregular maintenance (maintenance that does not take place on an annual basis) occurring in the same year or within an unreasonably short period after an asset is replaced, a sub-module for the scheduling of irregular maintenance and replacement was put in place. The sub-module ensures that, once the asset is renewed, the maintenance cycle is reset and the costs recalculated. These results are entered into the ECM and the sub-module then schedules the maintenance at the specified recurrence intervals for each component. For example, if the RWH system requires its pump to be serviced every five years and replaced after every ten years, the sub-module will enter the data as shown in Table 4-16.

Table 4-16: Calculation of corrective and irregular maintenance

Age of System	Age of Component	Task
9	9	
10	10	Replace end of year
11	1	
12	2	
13	3	
14	4	
15	5	Irregular maintenance

4.4.5.5 Life-cycle costing module outputs

The LCC module provided many outputs, including:

- Cumulative 'Cash' Expenditure vs. Time
- 'Cash' Expenditure vs. Time
- Cumulative Present Value of Expenditure vs. Time
- Present Value of Expenditure vs. Time
- Cash Flow Diagram
- Pie Chart (breakdown between Capital, Establishment, Routine Maintenance and Corrective Maintenance) for both Cash and Present Value
- Cumulative Net Present Value vs. Time
- Annualised Cost

While the LCC module provides a range of outputs, the *URSHM* only stores the annualised cost for each analysis, which is used in the optimisation module (Section 4.4.7).

4.4.6 Simulation of RWH and SWH

In order to optimise the RWH/SWH system for each property / subcatchment, it was necessary to simulate multiple designs for each. The *URSHM* was set up to simulate up to ten alternative designs. Each design was determined by the storage size as it was assumed the treatment and distribution would be determined by the end-use demands. Considering that there are roughly 121 subcatchments and 6,000 properties, this equates to between 1,210 and 60,000 design simulations per scenario – see Section 4.5. In order to ease the processing, the simulation of alternative system designs was automated in the *URSHM*. This was done through the creation of an 'input data' sheet, into which the data for all the properties / subcatchments could be entered. A macro was then developed that would, in sequence, simulate each property / subcatchment for each of the ten alternative designs. For each simulation, the following results were stored:

- Total water demand met
- Volumetric reliability
- Total runoff
- Percent of runoff that was harvested
- Average recorded cost per kilolitre.

Once all ten simulations for a specific property / subcatchment had been completed, the results were copied into an 'output and optimisation' sheet. The model then moved to simulating the next property / subcatchment until all the properties / subcatchments had been simulated.

This process took roughly 15 seconds per property / subcatchment, which resulted in runtimes of between 30 minutes (subcatchment) and 24 hours (individual properties).

4.4.7 Optimisation and results module

As noted in Section 4.4.6, the *URSHM* allowed for the simulation of ten systems for each property / subcatchment. For each simulation, the total water demand met, volumetric reliability, total runoff, percent of runoff that was harvested, and average cost per kilolitre was recorded. The optimal RWH/SWH was then selected by choosing one of the four objective functions shown in Table 4-17. These objective functions considered the end-user's options and were based on what criteria might rationally be used to select a system. The optimum system (out of the ten simulated) was then automatically selected for each property. Catchment scale results, by summing up the results of all properties in the catchment, were presented for: total water demand met, volumetric reliability, total runoff, percent of runoff that was harvested, and average cost per kilolitre.

Selecting the storage unit with the greatest volumetric reliability would typically result in the selection of the largest available tank size – as it will store a greater volume of water, overflow less and provide water longer during dry periods. Mwenge Kahinda *et al.* (2010) suggested that: '*the optimum size of the RWH tank should be taken where the curves begin to flatten. A further increase in tank capacity serves no useful purpose since it does not significantly increase the water security* [volumetric reliability]'. However, it is important to determine what '*serves no useful purpose*' means in a particular situation. An increase of 1% in volumetric reliability could be equal to 1Mℓ (substantial additional saving) or 1ℓ (not worth saving). Thus, the use of the '*point where curves begin to flatten*' is a bit misleading. Additionally, in the study by Mwenge Kahinda *et al.* (2010), the roof area was modelled as 20m^2, which would require at least (assuming no losses) 50mm of rainfall to fill the storage unit. Considering that the vast majority of storms are less than 50mm, it is not surprising that

Table 4-17: System optimisation objective functions

Objective Function	Description	Rational motivation for selecting system Using objective function
Objective Function A	System optimised to minimise the cost per kℓ of harvested rainwater	Minimal negative financial impact on the end user if a municipality forces the adoption of RWH/SWH.
Objective Function B	System optimised to maximise volumetric reliability	Provides maximum water supply security. May be appropriate in areas where water supply is intermittent.
Objective Function C	System optimised to maximise volumetric reliability while ensuring the cost per kℓ of harvested rainwater is less than the average cost per kℓ of potable water from the CoCT	Where the adoption of RWH/SWH is left to the end user/s, who is/are motivated primarily through the potential to make financial savings. This objective function may result in a substantial number of individuals not adopting RWH/SWH if the price of water is too low.
Objective Function D	System optimised according to user selected weighting of the cost per kℓ and the volumetric reliability. Default setting assumes equal weighting.	Where financial concerns and water security concerns need to be balanced. Essentially combines objective functions A and B.

increasing storage size made little difference in their study. In this study, the roof areas are comparatively larger, and an increase in volumetric reliability may serve a useful purpose. Therefore, in order to prevent the objective functions (B, C and D) selecting a larger tank that served '*no useful purpose*', the optimisation process ensured that any increase in storage size resulted in a lower overall cost at the household level. This was achieved in the following manner:

i) The difference in capital cost between each tank size was calculated and divided by the difference in storage size – Equation 4-18.

ii) The difference in capital cost was annualised over the storage tank's expected useful life – Equation 4-17.

iii) The annualised cost was divided by the current average cost per kilolitre that the property is paying for municipal water to determine an equivalent volume (EV) of water that would need to be harvested to ensure that the additional cost of increased storage size did not impact negatively on the financial viability of the system over its life cycle – Equation 4-19.

iv) The equivalent volume (EV) was divided by the total water demand (TWD) to provide the required change in volumetric reliability (ΔRVR) (per kilolitre of storage) required to ensure that the additional cost of increased storage size did not impact negatively on the financial viability of the system over its life cycle – Equation 4-20.

v) It was the assumed that any increase in overall volumetric reliability greater than the required change in volumetric reliability (ΔRVR) would, on an economic basis, be acceptable at the household scale. Essentially, the additional cost of increased storage size would either make no difference or improve the financial viability of the system over its life cycle

$$CCK = \frac{\Delta CC}{\Delta SV} \quad \textbf{4-18}$$

$$EV = \frac{A}{PAMWC} \quad \textbf{4-19}$$

$$RVR = \frac{EV}{TWD} \quad \textbf{4-20}$$

Where: *CCK* = Capital Cost per Kilolitre; *CC* = Capital Cost; *SV* = Storage Volume; *EV* = Equivalent Volume; *AC* = Annualised cost; *PAMWC* = Property's Average Municipal Water Cost; ΔRVR = required change in Volumetric Reliability; *TWD* = Total Water Demand (from RWH system)

The change in volumetric reliability as the storage volume increases ($\Delta VRSV$) is then assessed using Equation 4-21. Making use of an If-Else logical statement, the model then removes from consideration systems with storage sizes where increasing the storage tank size would not be an economically rational decision.

$$\Delta VRSV = \frac{\Delta VR}{\Delta SV} \quad \textbf{4-21}$$

This approach to assessing the optimum tank size with respect to volumetric reliability assumes that the only factor changing would be the capital cost of the storage tank. This was reasonable, as the pump and pre- / post-treatment systems required would be determined based on the intended end use. There would not be a significant difference, if at all, in the maintenance costs related to the tank itself.

4.4.7.1 Limiting tank size: Rainwater harvesting

It may be possible that a property has a large roof area and high demand, which would typically equate to a RWH system with a large storage tank. However, it is also possible that such a property may have very little outdoor area available for the tank, in which case it is not practical. The alternative would be to install an underground tank, but as noted in Section 2.4, these are typically very expensive and not economically viable in Cape Town. As a result, it was necessary to limit the maximum storage size, based on the each property's outdoor area.

For each storage size, the footprint of the tank, based on standard tank sizes (Jojo Tanks, 2013a), was calculated as a square, using the diameter of the tank as the length and width dimension – Figure 4-24. This was modelled to ensure that, if more than one storage tank was used to make up the total storage volume, the 'space', indicated in grey in Figure 4-24, would be accounted for.

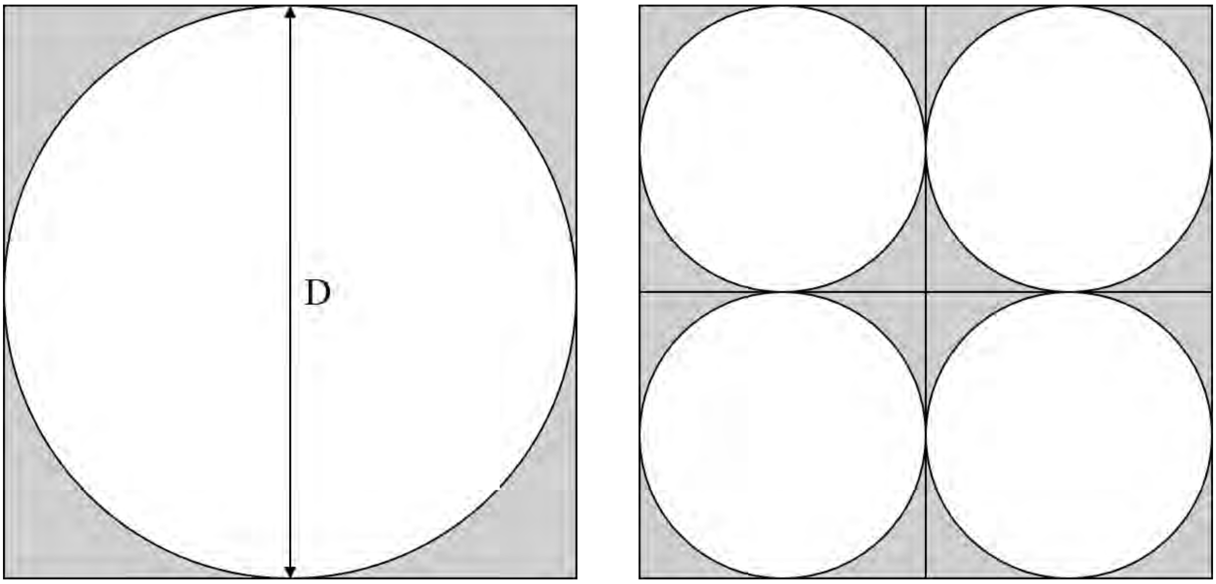

Figure 4-24: Estimating the area taken up by a RWH storage tank

The outdoor area was measured, from the captured land use data (Section 4.2.1), as the property area less the roof area and pool area. When undertaking the modelling, it was possible to limit the tank size by indicating the maximum area (as a percentage of outdoor area) that the storage tank can take. If the area of the storage tank/s exceeded the allowable area for a specific property, those tank sizes were not considered in the analysis.

4.4.7.2 Monte Carlo simulation of adoption

The *URSHM* also allowed an analysis of the potential variation in outcomes for different levels of adoption. This was specifically for analysing the potential variations in water saving and overflow of RWH systems, collectively. The *URSHM* made use of a Monte Carlo simulation (repeated random sampling of properties) to simulate varying rates of adoption from 10%–100%.

The Monte Carlo simulation was undertaken using Excel's random number function. Each property was provided a random number between 0 and 1. If the property's random number was less than or equal to the specified adoption rate, the property was considered to have adopted RWH. The results of all properties that 'adopted' RWH were summed to provide an estimate, at a specified level of adoption, of the: total water demand met, volumetric reliability, total runoff, percent of runoff that was harvested, and average cost per kilolitre. It was, however, recognised that the results may vary, depending on which properties adopt RWH. The procedure was repeated 10,000 times using an Excel macro, and each time, the catchment-wide results were recorded. An analysis of the results, therefore, provides upper, lower and average estimates of the potential impacts of RWH within the catchment.

4.4.7.3 Rational choice simulation

The research assumed that individuals act rationally and will adopt RWH only once it provides owners / occupants with water at a lower cost per kilolitre. In order to understand whether properties would adopt RWH, the *URSHM* calculated the average cost per kilolitre of water used by each household, based on the average monthly water demand and the CoCT's block rising water tariffs. If the cost per kilolitre based on municipal supply was higher than the cost per kilolitre of harvested rainwater, then it was assumed that the property would 'rationally' choose to adopt RWH.

As the results will show (Section 5), RWH was not a viable option for the majority of properties in the catchment, due to the financial implications of RWH systems – that is, the cost of running a system on a per-kilolitre basis is more than using municipally supplied water. In order to understand the extent to which water tariffs need to be increased, an Excel macro was developed that varied the tariff structure (increasing / decreasing) in increments selected for each analysis. The Excel macro recorded, at a catchment scale, the total water demand met, volumetric reliability, total runoff, percent of runoff that was harvested, and average cost per kilolitre.

4.4.7.4 Storage size distribution

For the selected optimisation objective function (Table 4-17), the *URSHM* provided a distribution of the storage size across the different suburbs and catchment as a whole. This was useful, as it provided the first indication of what storage sizes should be used in the RSA, and could be used in other studies that make use of statistics to represent the catchment, for example, Mitchell *et al.* (2008a).

4.4.8 Assessing the impact of RWH/SWH on peak flows and flooding

In order to assess the potential benefits of RWH and SWH with respect to a catchment's runoff, flooding and peak flow, it was necessary to incorporate the results of the *URSHM* into the Catchment Stormwater Model (CSM). Two distinct approaches were used for RWH and SWH respectively, as discussed below.

4.4.8.1 Rainwater harvesting

As noted in Section 2.4.5.3, recent research on the stormwater management benefits of RWH – for example, Steffen *et al.* (2013) – modelled each property in a catchment in detail, including the individual RWH systems. Steffen *et al.* (2013) did not, however, consider the spatial and temporal variability in water demand at the household scale. This might be as a result of the time-consuming nature of assigning individual water demand patterns. Additionally, if the required data is not available it is difficult to set up – and calibrate – the model. While this might be possible in a catchment of 100 properties, going to this detail in a catchment of over 6,000 properties would pose significant calibration challenges, only made worse by the limited available calibration data. Additionally modelling each property in detail would result in extended computational runs. However, in light of recent findings, it would also be unreasonable to assess stormwater management and flooding impacts by linearly upscaling RWH systems within the catchment – as has been established in Section 2.4.5.3.

In order to overcome this, the *URSHM* was set to superimpose the diurnal water demand patterns emanating from the Mayer *et al.* (1999) study, using hourly rainfall data, and calculating a ten-year overflow volume time series based on a specified storage size for each property – dependent on which water demand scenario (Table 4-20) and objective function was being considered (Table 4-17), as discussed in Section 4.5. The *URSHM* computed the total volume of overflow at each time step, for all properties within each subcatchment. The total volume of overflow was divided by the total effective roof area to get an equivalent depth of runoff (mm). This was then imported into *SWMM* as a depth of rainfall time series that was linked to a 'dummy' subcatchment representing the effective roof area of all RWH systems in the catchment (as will be explained below). In order to account for evaporation losses that are calculated in *SWMM* during a storm event, the evaporation losses calculated in the *URSHM* were added at each time step where the depth was greater than zero, i.e. this prevents evaporation losses being accounted for in the *URSHM* and *SWMM*.

SWMM represents each subcatchment conceptually, as illustrated in Figure 4-25a. A directly connected impervious area (impervious area connected directly to the stormwater drainage system) was routed to the outlet, while an impervious area that was not directly connected to the drainage system (assuming subarea routing is set to pervious) was routed to pervious areas before being routed to the subcatchment's outlet.

In order to model the contribution of RWH systems to the reduction of peak flow and flooding, each subcatchment was split into two – Figure 4-25b. The first, a 'dummy' subcatchment, represented the effective roof area (Equation 4-23) connected to the RWH systems. This was created with a catchment area equal to the total roof area being considered, 100% impervious, 100% zero depression storage and a catchment width equal to the original catchment width. Rainfall for this dummy catchment was considered to be that of the time series created above. This approach was tested by letting the *URSHM* calculate (ignoring RWH – demand set to zero) an overflow depth time series that was then entered into *SWMM* for a subcatchment with the properties described above and compared to the runoff of a catchment fully modelled in *SWMM*. As evident in Table 4-18, it made no significant difference. The second subcatchment represented the original subcatchment, except for the effective roof areas connected to RWH systems – which now form part of the 'dummy catchment'. The subcatchment properties were then adjusted to conserve the total depression storage (Equation 4-22), catchment area (not increase it) (Equation 4-24), imperviousness (Equation 4-25) and impervious area routed to pervious area (Equation 4-26) as a result of considering roof areas independently. In order to ensure conservative estimates of the potential impact that RWH might have on reducing peak flows and flooding, it was assumed, in Equations 4-22 to 4-26, that the percentage routed needed to be adjusted assuming that the same (as before adjustment for RWH) total impervious area was routed to pervious areas. Depression storage and total area were conserved (i.e. the total volume of depression storage / total area in the calibrated CSM was equal to the sum of depression storage from the *URSHM* model and the adjusted CSM).

$$AD_{imp}=\frac{A_c\times Imp_C\times D_{Imp}\text{-}A_r\times D_{roof}}{A_c\times Imp_C\text{-}A_r} \qquad \textbf{4-22}$$

$$A_r= C_R\times A_T \qquad \textbf{4-23}$$

$$AA_C= A_c\text{-}A_r \qquad \textbf{4-24}$$

$$AImp_c=\frac{A_c\times Imp_C\text{-}A_r}{AA_c} \qquad \textbf{4-25}$$

$$APERC(1)_{Perv}=\frac{A_c\times Imp_C\times PERC_{Perv}}{AA_c\times AImp_C} \qquad \textbf{4-26}$$

$$APERC(2)_{Perv}=\frac{A_c\times Imp_C\times PERC_{Perv}-A_r}{A_c\times AImp_C} \qquad \textbf{4-27}$$

Where: AD_{imp} = adjusted impervious depression storage (mm); $A_{c/r}$ = Area of the catchment/effective roof area (ha); Imp_c = fraction of catchment that is impervious (%); $D_{imp/r}$ = Depression storage of impervious area/roof (mm); C_R is the runoff coefficient; AA_c = Adjusted catchment area (ha); $Aimp_c$ = adjusted fraction of catchment that is impervious (%); A_T = total roof area (ha); $APERC_{perv}$ = adjusted percentage of impervious runoff routed to pervious (%); and $PERC_{perv}$ = percentage of impervious runoff routed to pervious (%).

From a modelling perspective, the model represents all roofs connected directly to the stormwater system, which – as a result of historic city by-laws requiring properties to connect to the stormwater drainage system – is not uncommon. While this may not always be the case, by ensuring the same total impervious area routes to pervious areas, it ensures the catchment has the same initial storage available and does not affect the results of an analysis without RWH – as indicated by the error functions in Table 4-18 and Table 4-19. However, not all the roof areas are directly connected to the stormwater system and potentially the installation of RWH

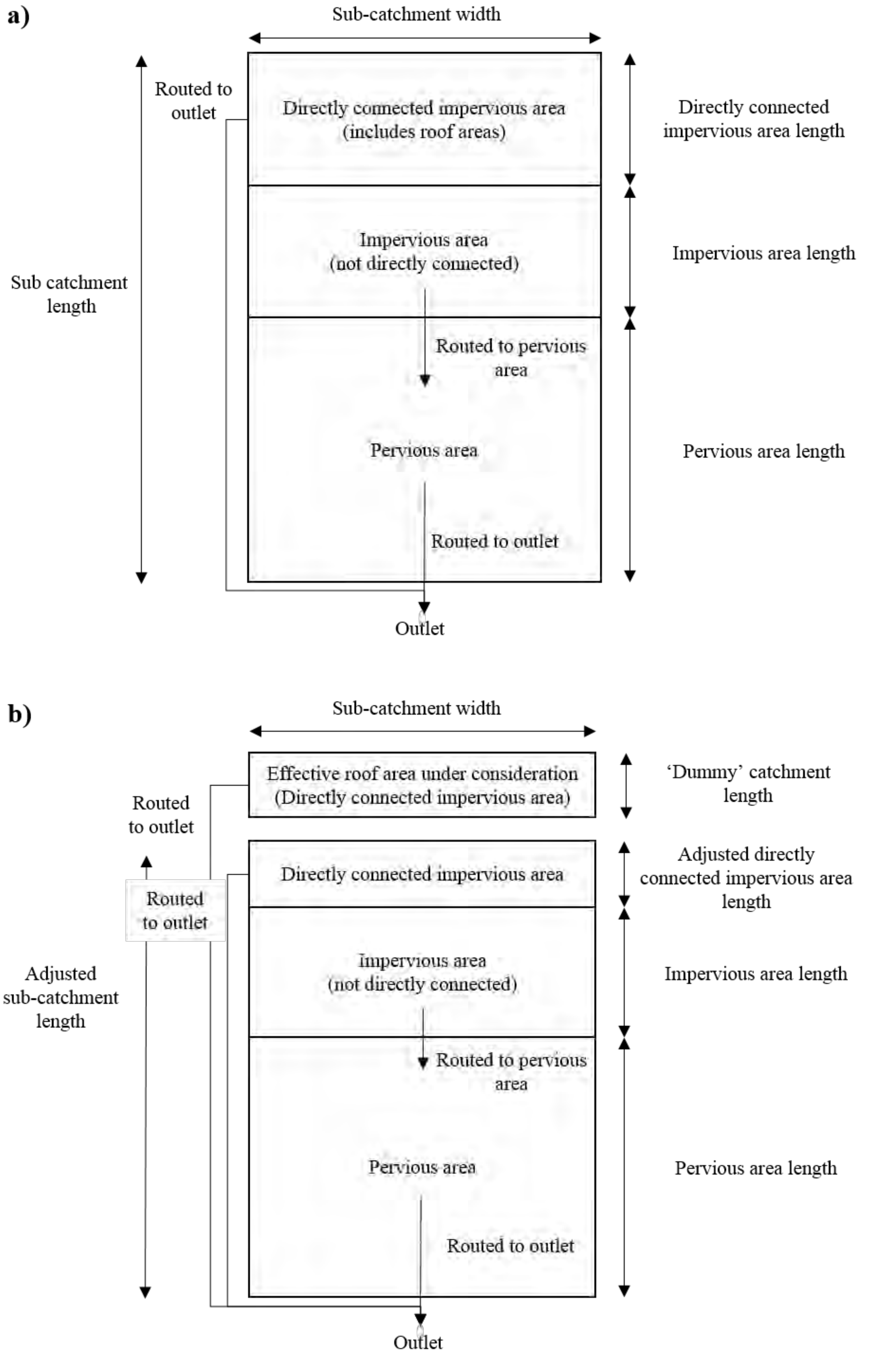

Figure 4-25: a) *SWMM* subarea routing; b) modelling of RWH system in *SWMM*

systems could result in the connection, rather than the expected disconnection, of roof areas from the stormwater system. The approach taken here might, in such cases, overestimate the reduction in peak flows of smaller storms, as it results in attenuating directly connected impervious areas. Therefore, in order to account for this, the analysis was repeated ensuring the directly connected area (impervious area not routed to pervious area) remained constant (by adjusting the percentage routed) – Equation 4-27. This would result in potentially underestimating the impact of RWH systems on peak flows. From the results of the above two analyses, it is possible to draw inferences as to the impact of RWH systems. The alternative approach would require the creation of six 'dummy' subcatchments to represent roofs connected to pervious areas, pervious area connected to the rest of the catchment, directly connected impervious area, roofs (directly connected), impervious area routed to pervious area, and roofs routed to pervious area. As there is no indication of what percentage of roofs are/are not connected directly to the stormwater system and due to the significant impact this will have on computational run time, it was not considered worthwhile, as the above approach provides an upper and lower bound for the impact that RWH may have on a stormwater drainage system. This allowed for the modelling of RWH systems to be reasonably represented within the CSM.

Table 4-18: Results of *URSHM* time series vs. *SWMM* for modelling runoff from a roof

Error function	Result
Integral square error rating*	Excellent
Nash-Sutcliffe efficiency (NSE)	0.999
Coefficient of determination (R^2)	0.999
Maximum difference in peak flows of events	< 1 %
Maximum difference in total runoff volume of events	< 1 %

* Shamsi (1997)

Table 4-19: Results of SWMM model with *URSHM* time series vs. calibrated *SWMM* for modelling runoff for the Liesbeek River catchment

Error function	Result
Integral square error	0.0126
Integral square error rating*	Excellent
Nash-Sutcliffe efficiency (NSE)	0.997
Coefficient of determination (R^2)	0.997
Maximum difference in total runoff volume of events	< 1 %

* Shamsi (1997)

4.4.8.2 Stormwater harvesting

SWH was, in principle, easier to model in the Catchment Stormwater Model (CSM), as it required a maximum of 121 systems (there are 121 urbanised catchments where SWH is considered) to be modelled. The *URSHM* provided the required size of the storage, which was modelled as a retention pond allowing for evaporation losses. The diurnal water demand patterns resulting from Mayer *et al.* (1999) were then superimposed on each subcatchment's / catchment's water demand. This was considered reasonable as Mayer *et al.*'s (1999) diurnal pattern is representative of the average demand of a large sample of properties. An hourly water demand time series was generated for each subcatchment / catchment that was used to control a 'pump', which simulated the extraction from the storage / retention pond. The pump assumed a constant demand over the hourly time step, which is the same as the rainfall data. This allowed for the modelling of a SWH system to be reasonably represented within the CSM.

4.4.8.3 'A steep catchment'

One potential problem with the selection of the Liesbeek River Catchment as a case study is that the catchment is prone to flooding as a result of the high rainfall in the upper catchment and the fact that the catchment flattens out (Brown & Magoba, 2009). This might mean that, while RWH/SWH does have a valuable impact in attenuating peak flows / mitigating flooding, this may not be realised in the Liesbeek River Catchment as a result of the high rainfall and steep upper catchment. The risk, therefore, is that the results of this study could be inappropriately applied in catchments or areas where the catchment is predominantly urbanised. In order to test this possibility, the catchment was adjusted such that only runoff from a hypothetical 'urbanised catchment' was considered. This allows an analysis of the benefits of RWH/SWH in a flatter, fully urbanised catchment.

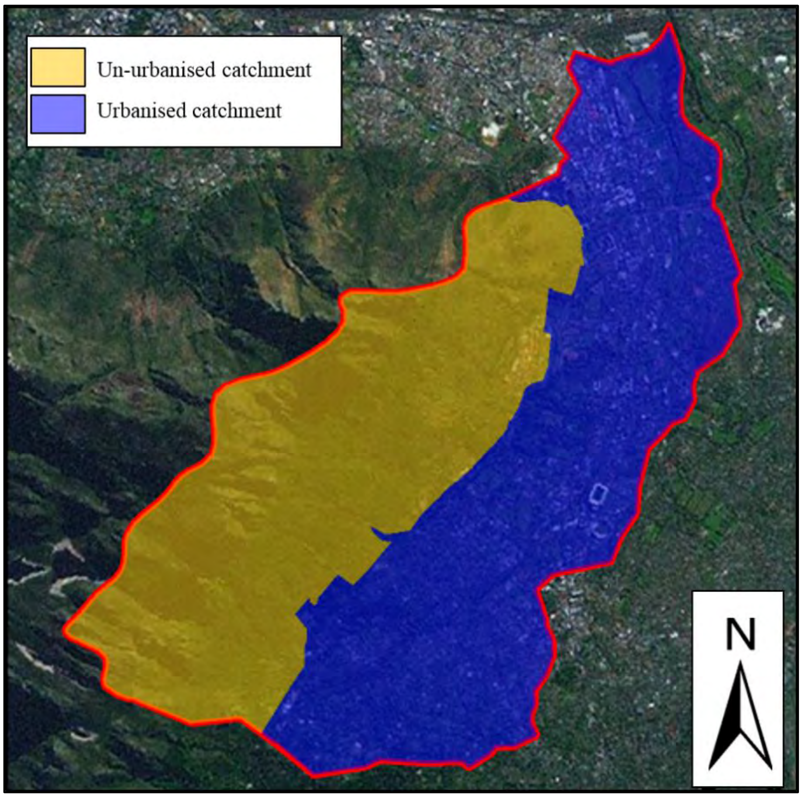

Figure 4-26: Liesbeek River Catchment separated into 'urbanised' and 'un-urbanised' catchment areas

4.5 Assessing the viability of RWH and SWH

The viability of RWH and SWH was assessed in three sets of scenarios. The first two sets considered the viability of RWH (Scenarios 1-20) and SWH (Scenarios 21-26) separately. The scenarios considered each at a subcatchment and catchment scale, for a range of different end-use combinations – as discussed in Sections 4.5.1 and 4.5.2. The analyses were then repeated using data from the 31 downscaled climate change models (discussed in Section 4.2.3.4) in the areas where data was available. As a result, it was only possible to consider subcatchment SWH. Based on the results, conclusions were drawn as to the current and future viability of RWH and SWH within the Liesbeek River Catchment.

The third set of scenarios (Scenarios 27-30) modelled the combination of RWH and SWH at a subcatchment and catchment scale in order to assess the viability of encouraging RWH and SWH concurrently. The analysis considered each at a catchment scale under current climatic conditions for a range of different scenarios and end-use combinations, as discussed in Section 4.5.3. The procedure was repeated using data from the 31 downscaled climate change models (discussed in Section 4.2.3.4) in the areas where data was available. Based on the results, conclusions were drawn as to the current and future viability of concurrently encouraging / implementing RWH and SWH within the Liesbeek River Catchment.

The results from the three sets of scenarios were then used to draw conclusions as to what might be the most appropriate approach to implementing RWH and/or SWH within the

Liesbeek River Catchment. This included critically questioning whether RWH and/or SWH should even be considered as appropriate interventions within the catchment.

4.5.1 Viability of rainwater harvesting

A total of 20 scenarios were analysed in order to assess the viability of RWH, as shown in Table 4-20. Scenarios 1 through 10 were undertaken using the total roof area determined for each property (See Section 4.2.1). Scenario 1 was undertaken in order to allow for comparison with Jacobs *et al.* (2011) who considered RWH for outdoor irrigation only. Scenarios 2 to 10 represent different combinations of what – based on the literature review of current uses of harvested rainwater and similar studies (e.g. Burns *et al.*, 2012) – may be considered appropriate '*fit for purpose'* uses of harvested rainwater in the CoCT.

Table 4-20: Scenarios 1 through 20 (rainwater harvesting)

Scenario (100% roof area)	Scenario (100 m^2 or 50% roof area)	End-use water demand description	RWH system description
Scenario 1	Scenario 11	Supplying pools only	Gravity fed
Scenario 2	Scenario 12	Supplying garden irrigation only	Gravity fed
Scenario 3	Scenario 13	Supplying washing machine only	Directly pumped
Scenario 4	Scenario 14	Supplying toilet flushing only	Directly pumped
Scenario 5	Scenario 15	Supplying showers / bath only	Directly pumped
Scenario 6	Scenario 16	Supplying toilet flushing, washing machine only	Directly pumped
Scenario 7	Scenario 17	Supplying washing machine, shower/bath only	Directly pumped
Scenario 8	Scenario 18	Supplying toilet flushing, washing machine, shower/bath only	Directly pumped
Scenario 9	Scenario 19	Supplying toilet flushing, washing machine, shower / bath, pool only	Directly pumped
Scenario 10	Scenario 20	Supplying toilet flushing, washing machine, shower / bath, pool, garden, irrigation only	Directly pumped

For each scenario shown in Table 4-20, ten possible systems were considered – defined by the storage capacity (0.5 kℓ, 1 kℓ, 1.5 kℓ, 2.2 kℓ, 5 kℓ, 10 kℓ, 15 kℓ, 20 kℓ, 25 kℓ and 30 kℓ) – based on the standard sizes offered by suppliers. Figure 4-27 provides a schematic of a typical RWH system, which would be scaled (e.g. larger roof area or bigger pump) depending on the property's characteristics. For Scenarios 1 and 2, no post-storage treatment was considered.

For Scenarios 3 through 10, post-storage treatment included UV filtration. The size of the pump and filtration units was determined by the intended use of the water and the size of the household. The minimum unit allowed a flow of 10 ℓ/minute with the maximum allowing 40 ℓ/minute – equivalent to the flow rate provided to residential properties (CSIR, 2005b). For example, for Scenario 1, no pumping or filtration was considered, as filling a pool could be accomplished by gravity flow. For Scenario 9, the 40 ℓ/minute pump and filtration units were incorporated, as the water was effectively being used in place of municipal supply.

Scenarios 11 through 20 repeated Scenarios 1 through 10, assuming that the connected roof area was 100 m^2 or 50% of the roof area, whichever was the lesser. This is in line with the MP 4.2 planning requirements of Queensland, Australia (DLGP, 2008) – see Section 2.4.2.1.

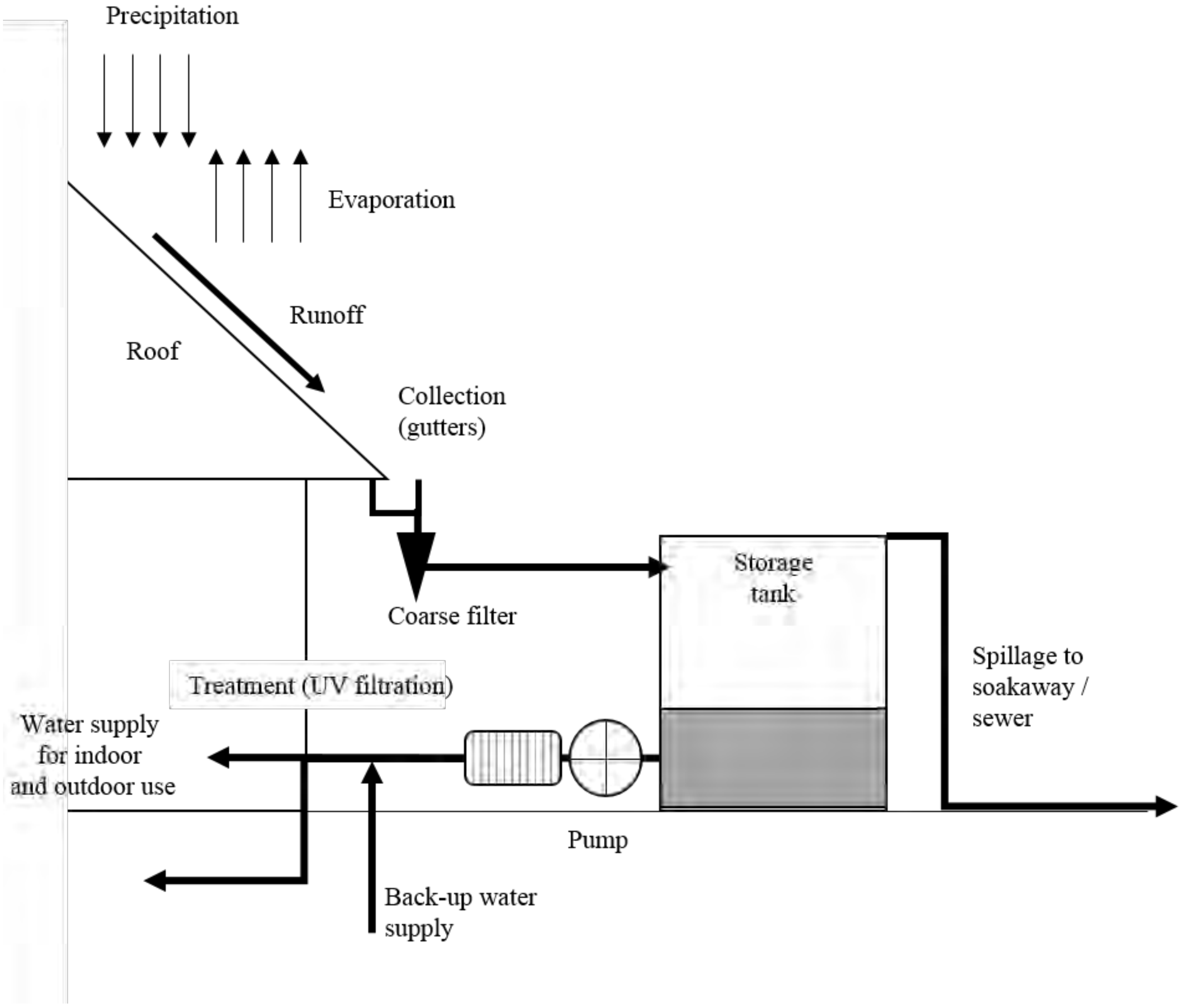

Figure 4-27: Typical RWH system

In line with Mitchell *et al.* (2008a) and Neumann *et al.* (2011), amongst others, it was assumed that depression storage and runoff coefficients factors could be assumed to be normally distributed, as these are the products of an '*infinite number of independent random events'*

(StatSoft Inc., 2013). Each property was thus assigned a once-off value based on a normal distribution and modelled with its assigned runoff coefficient and depression storage values for all analyses.

4.5.2 Viability of stormwater harvesting

A total of six scenarios were analysed in order to assess the viability of SWH, as shown in Table 4-21. These scenarios were analysed considering SWH at a subcatchment and catchment scale. The scenarios were based on a review of the literature, which indicated that harvested stormwater is most appropriate for outdoor uses and/or toilet flushing (See Section 2.6.9). While stormwater could be treated to potable standards, this would likely be too costly.

For each scenario shown in Table 4-21, the *URSHM* was used to optimise SWH system storage. The *URSHM* calculated the storage size as a percentage of the total catchment area. This was done to allow for a degree of comparison between the storage sized for catchments of different sizes. Figure 4-28 provides a schematic of a typical SWH system, which would be scaled (e.g. larger catchment or bigger pump) depending on the subcatchment's characteristics. This approach was taken assuming a retention pond with an average depth of 1.5m – in line with the standard design in the South African Guidelines for SuDS – which is typically used as the minimum depth which prevents the growth of reeds. An average depth of 1.5m implies that in areas the pond depth is greater and less than 1.5m. For all scenarios, post-treatment was considered as basic sand filtration and UV filtration – in order to reduce the risks as a result of pathogens and prevent the sedimentation of suspended material within the distribution system.

Table 4-21: Scenarios 21 through 26 (stormwater harvesting)

Scenario	End-use water demand description
Scenario 21	Gardens (at subcatchment scale)
Scenario 22	Gardens (catchment scale)
Scenario 23	Gardens and pools (at subcatchment scale)
Scenario 24	Gardens and pools (catchment scale)
Scenario 25	Gardens, pools and toilets (at subcatchment scale)
Scenario 26	Gardens, pools and toilets (catchment scale)

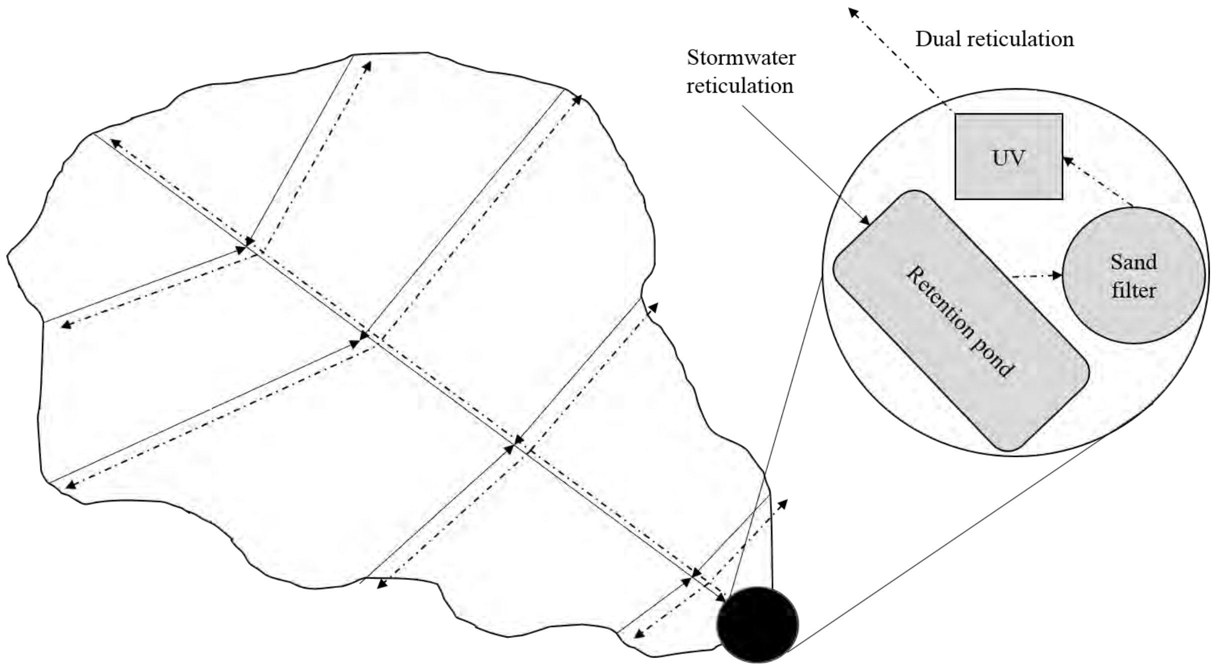

Figure 4-28: Typical stormwater harvesting system

The *URSHM* does not expressly consider whether there is available land on which to develop a SWH facility or whether the land is appropriately positioned. It is recognised that the retrofitting of SWH facilities within the Liesbeek River Catchment may not be possible in many of the subcatchments. The purpose of these analyses are to investigate whether, with similar levels of development, SWH would have been a viable option if considered by the city planners. The scenarios in Table 4-21 are undertaken with the life-cycle costing excluding and then including the cost of the land the SWH facilities would require. This provides some insight into the costs of land and, potentially, the costs of retrofitting.

4.5.3 Scenarios for assessing the viability of concurrent implementation of rainwater and stormwater harvesting

A total of eight scenarios were analysed in order to assess the viability of RWH and SWH in combination, as shown in Table 4-22. These scenarios were analysed considering SWH at both a subcatchment and catchment scale. The scenarios essentially combined the scenarios completed in Sections 4.5.1 and 4.5.2. From a practical implementation point of view, if properties were to make use of both RWH and SWH, it would make sense that RWH be considered for indoor uses while outdoor demand is met by SWH. There may, however, be concern over the quality of SWH for use in swimming pools, in which case SWH would likely only be used for irrigation.

Table 4-22: Scenarios 27 through 30 (rainwater and stormwater harvesting)

Scenario	Rainwater Harvesting		Stormwater Harvesting	
	End-use water demand description	**Scenario from Table 4-20**	**End-use water demand description**	**Scenario from Table 4-21**
Scenario 27	Supplying toilet flushing, washing machine, shower / bath only	Scenario 8 or 18	Gardens and pools (at subcatchment scale)	Scenario 23
Scenario 28	Supplying toilet flushing, washing machine, shower / bath only	Scenario 8 or 18	Gardens and pools (catchment scale)	Scenario 24
Scenario 29	Supplying washing machine, shower / bath only	Scenario 7 or 17	Gardens, pools and toilets (at subcatchment scale)	Scenario 25
Scenario 30	Supplying washing machine, shower / bath only	Scenario 7 or 17	Gardens, pools and toilets (catchment scale)	Scenario 26

4.6 Summary of method

This Chapter has presented the methods that were used and developed in this thesis. The most important aspects of Chapter 4 are: the disaggregation of water demand data (Section 4.2.3.5) and the development of the Urban Rainwater Stormwater Harvesting Model (*URSHM*) (Section 4.4) as these methods and the tools developed could provide the basis for future studies of a similar nature.

The disaggregation of water demand data provides a method for reasonably assessing the per-capita and household demand for properties with and without water demand data. The *URSHM* offers a means of assessing the viability of RWH and SWH in areas where these are proposed as interventions, and for which no data as to the size of proposed – or installed – RWH or SWH systems exist. The *URSHM* makes use of accepted methods for hydraulically modelling RWH and SWH systems, in combination with a whole life cycle costing approach to assess the economic viability of a RWH/SWH system. The results of the hydraulic and economic calculations are used in four objective functions that rationally size a RWH/SWH system, dependent on the selected objective function.

Chapter 4 also describes the methods used to overcome the challenges of modelling the impacts of climate change, including estimating evaporation. It concludes with an overview of the 30 scenarios that were analysed in order to assess the viability of RWH and SWH in the RSA. The analysis of these 30 scenarios form the basis of the results and conclusions with respect to the viability of RWH and SWH in the RSA.

5. Results and discussion

This chapter consists of four sections. It is structured to provide the results of RWH and SWH separately and then in combination. A brief overview of Sections 5.1 through 5.4 is provided below.

Section 5.1 discusses the analyses of the viability of RWH in the Liesbeek River Catchment.

Section 5.2 discusses the analyses of the viability of SWH in the Liesbeek River Catchment.

Section 5.3 discusses the analyses of the concurrent use of RWH and SWH in the Liesbeek River Catchment.

Section 5.4 is a summary and discussion of the analyses and what implications they might have for the implementation of RWH and SWH within the Liesbeek Catchment and throughout the RSA.

In Section 4.1 it was noted that in this research RWH is considered to supply only single residential (houses) water demand (results presented in Section 5.1), whereas SWH is considered to supply all residential properties – including houses and blocks of flats. The percentages for reduction in water demand in Section 5.1 are reported as a percentage reduction in single residential water demand, whereas the percentages in Section 5.2 are calculated as a reduction in total residential water demand. Therefore, while the percentage reduction reported in Section 5.2 may be lower than the reductions reported in Section 5.1, quantity-wise, they may be substantially more. This is due to the inclusion of water demand from blocks of flats and university residences in the analysis.

5.1 Viability of rainwater harvesting

For ease of reference, Table 4-17 which details the different Objective Functions (OF) incorporated into the *URSHM*, and Table 4-20 which details the different scenarios used to assess the viability of RWH have been repeated below.

Table 4-17: System optimisation objective functions

Objective Function	Description	Rational motivation for selecting system using objective function
Objective Function A	System optimised to minimise the cost per kℓ of harvested rainwater	Minimal negative financial impact on the end user if a municipality forces the adoption of RWH/SWH.
Objective Function B	System optimised to maximise volumetric reliability	Provides maximum water supply security. May be appropriate in areas where water supply is intermittent.
Objective Function C	System optimised to maximise volumetric reliability while ensuring the cost per kℓ of harvested rainwater is less than the average cost per kℓ of potable water from the CoCT	Where the adoption of RWH/SWH is left to the end user/s, who is/are motivated primarily through the potential to make financial savings. This objective function may result in a substantial number of individuals not adopting RWH/SWH if the price of water is too low.
Objective Function D	System optimised according to user inputted weighting of the cost per kℓ and the volumetric reliability. Default setting assumes equal weighting.	Where financial concerns and water security concerns need to be balanced. Essentially combines objective functions A and B.

Table 4-20: Scenarios 1 through 20 (rainwater harvesting)

Scenario (100% roof area)	Scenario (100 m^2 or 50% roof area)	End-use water demand description	RWH system description
Scenario 1	Scenario 11	Supplying pools only	Gravity fed
Scenario 2	Scenario 12	Supplying garden irrigation only	Gravity fed
Scenario 3	Scenario 13	Supplying washing machine only	Directly pumped
Scenario 4	Scenario 14	Supplying toilet flushing only	Directly pumped
Scenario 5	Scenario 15	Supplying showers / bath only	Directly pumped
Scenario 6	Scenario 16	Supplying toilet flushing, washing machine only	Directly pumped
Scenario 7	Scenario 17	Supplying washing machine, shower / bath only	Directly pumped
Scenario 8	Scenario 18	Supplying toilet flushing, washing machine, shower / bath only	Directly pumped
Scenario 9	Scenario 19	Supplying toilet flushing, washing machine, shower / bath, pool only	Directly pumped
Scenario 10	Scenario 20	Supplying toilet flushing, washing machine, shower / bath, pool, garden, irrigation only	Directly pumped

5.1.1 RWH in the Liesbeek River Catchment

The following section presents and discusses the results relating to the viability of RWH in the Liesbeek River Catchment.

5.1.1.1 System optimisation

For each scenario, the system was optimised based on the objective functions (OF) in Table 4-17. It was evident that there was a clear trend towards selecting the largest available storage, typically between 5-15 kℓ, as illustrated in Figure 5-1 for Scenario 10. Larger storage tanks were favoured as a result of the following:

- An increase in storage volume, where demand is sufficient and the roof is large enough, increases volumetric reliability and demand met (OF B/D).
- An increase in demand met results in a lower cost per kilolitre (OF A/C/D).

It is, therefore, generally favourable to have a large storage tank; indeed, for most properties in the Liesbeek River Catchment, a storage tank of between 5 and 15 kℓ in size seemed to be the optimal choice. Storage tank sizes greater than 15 kℓ were typically only appropriate for sites with very high demand and for those that had large roof areas. This is also significant from the perspective of stormwater management, as the larger the tank, the greater the opportunity for a benefit in terms of peak flow attenuation and volume reduction.

Of the different objective functions used for optimisation, each has its own advantages and disadvantages. OF A, which optimises a system to minimise the cost per kℓ of harvested rainwater, is appropriate where RWH is the primary source of water. However, it does not consider the total cost to the household and therefore where RWH is not the primary source of water it can lead to a higher overall cost of water for the household. OF B, which optimises systems to maximise volumetric reliability, provides a rational approach to optimising a RWH system. It ensures that the maximum benefit is accrued (note the increasing volumetric reliability decrease cost per kℓ), but ensures that selecting a larger storage size will not result in an overall increase in cost to the household, as explained in Section 4.4.7. OF C maximises volumetric reliability while ensuring that the cost per kℓ of harvested rainwater is less than the cost per kℓ of potable water from the CoCT. OF C would therefore be the most rational means of optimising a RWH system for a household with access to another source of water – as it ensures the household's water costs do not increase. However, as will be shown below, the cost of municipal water is typically cheaper than RWH. To illustrate this, Figure 5-1 shows that the RWH systems were optimised using OF C with the municipal water tariffs assumed to be triple their current levels – using municipal tariffs less than this meant that it would be better for most of the properties in the catchment not to use RWH. OF D requires a significant social survey to be of any value, which is beyond the scope of this research, but it did allow for an assessment of the impact that social perceptions and values might have on the adoption of RWH. Based on the above discussion, OF B was considered the most rational approach to optimising RWH systems for the analysis of the potential catchment-scale impacts of RWH.

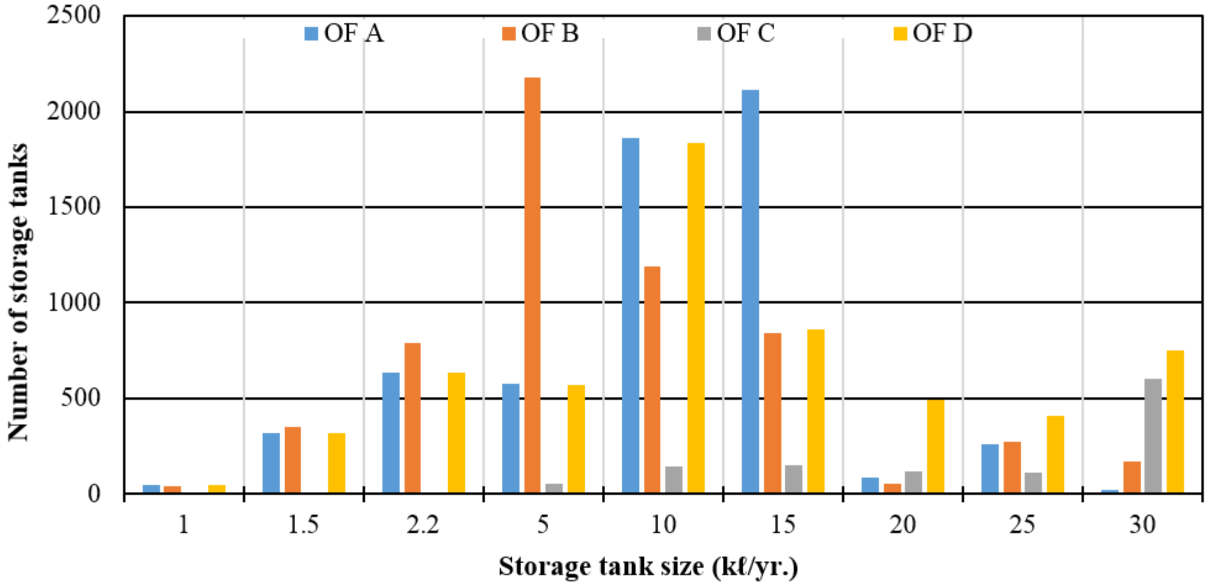

Figure 5-1: Distribution of storage tank sizes for OF A – D, Scenario 10 (Supplying toilet flushing, washing machine, shower / bath, pool, garden, irrigation only)

Figure 5-2 and Figure 5-3 compare the distributions of storage tank sizes optimised using OF B for scenarios 1 to 20. Figure 5-2 highlights that, as demand increases – Scenario 1 has the lowest demand, while Scenario 10 has the highest demand – the optimal storage tank size increases. However, when Figure 5-2 and Figure 5-3 are compared – bearing in mind that Scenario 1 corresponds to Scenario 11, except that Scenario 11 has a reduced catchment (roof) area connected to the storage tank – the catchment size is, as would be expected, a limiting factor.

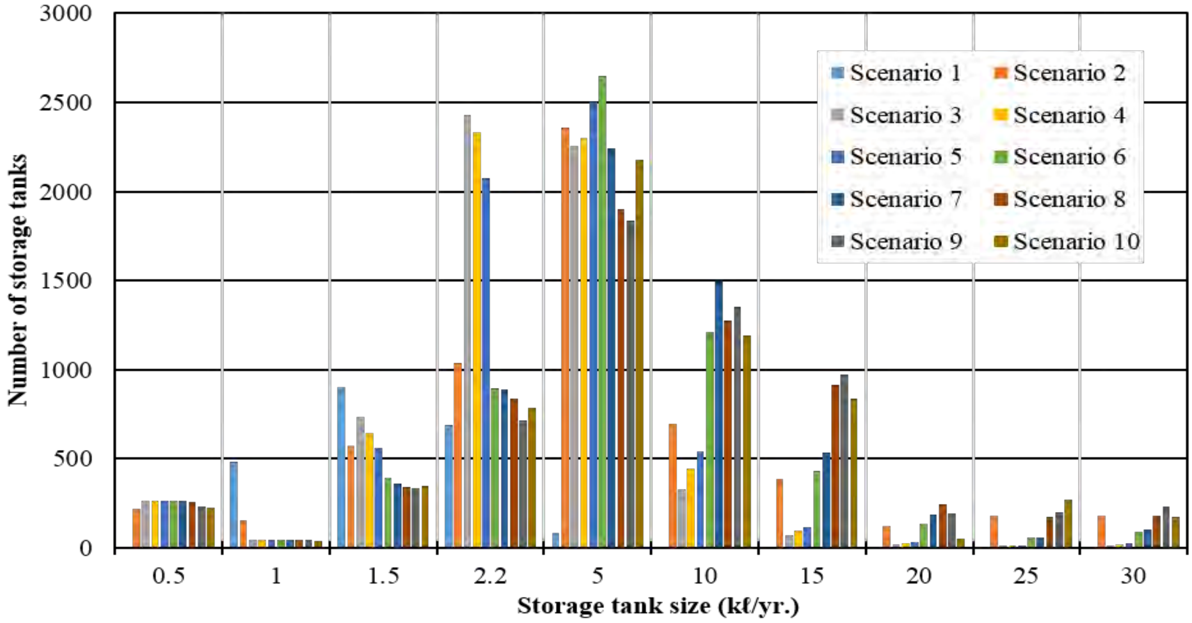

Figure 5-2: Distribution of storage tank sizes, scenarios 1 through 10

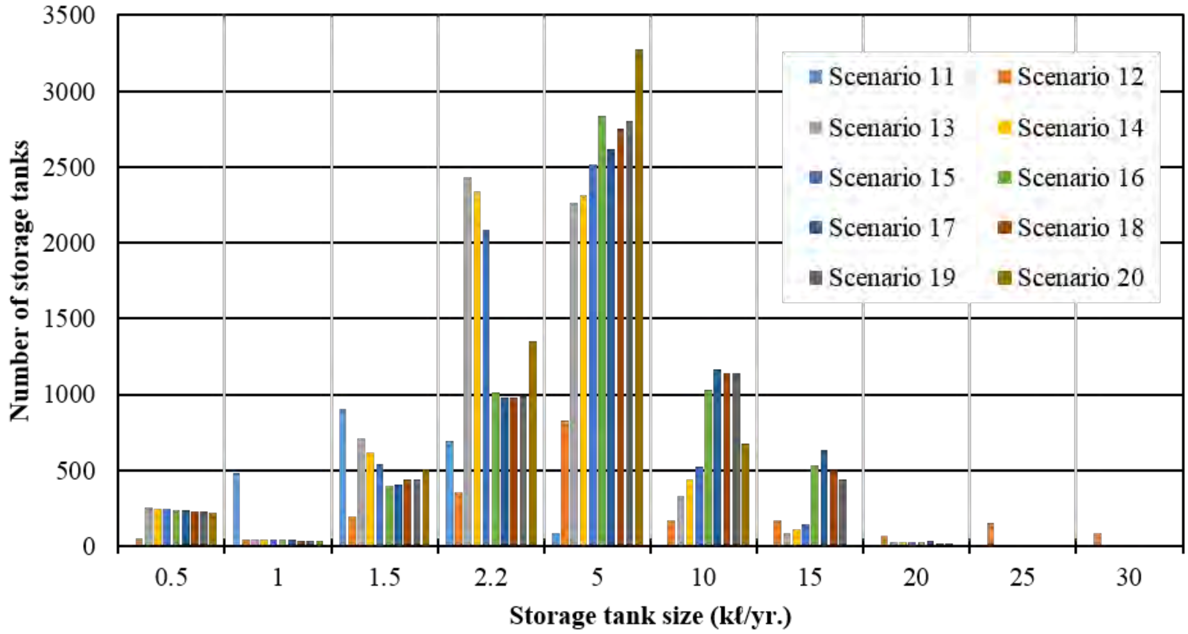

Figure 5-3: Distribution of tank sizes, scenarios 11 through 20

5.1.1.2 Analysis of the postulated scenarios

Figure 5-4 presents the results of an analysis assuming 100% adoption of RWH throughout the catchment, with systems optimised using OF B. Figure 5-4 indicates that Scenario 1, where RWH is considered for filling pools, could reduce annual water demand by a maximum of 1.3% (28 Mℓ/yr.), while Scenario 2, which considers RWH for garden irrigation, could reduce annual water demand by up to 9.5% (200 Mℓ/yr.). Further analysis indicates that, throughout much of the winter, the storage tank will be quickly filled and will overflow, offering little beneficial use as there is sufficient precipitation not to require the topping up of swimming pools. This is also evident in the relatively low volumetric reliability for Scenarios 1 and 2, even though the demand is significantly less than in other scenarios. This confirms the finding of Jacobs *et al.* (2011) that *'the major limitation with RHSs* [RWH] *in South Africa, particularly in the Western Cape winter rainfall region, stems from lack of synchronisation between rainfall and garden irrigation demand'.*

Figure 5-4 however also indicates that, as RWH is considered for more end uses (Scenarios 3 through 10), up to 29% of the annual demand from single-residential properties may be met through RWH as a consequence of the use of rainwater for various purposes such as toilet flushing, washing machine use, shower / bath use, as well as pool and garden irrigation. As RWH is considered for more end uses, the volumetric reliability decreases. While Scenarios 8 through 10 meet roughly the same water demand, Scenario 10 has a significantly lower volumetric reliability (water demand met divided by total water demand) due to the additional outdoor irrigation demand which is typically present during the dry season (see the discussion of Scenarios 1 and 2). As a result, the choice of end uses is important for the system operator (homeowner), who may wish to have a system with a higher volumetric reliability – ability to meet water demand – and thus finds an alternative source of water for irrigation that may be more '*fit for purpose*'. For example, this could include greywater or groundwater. This would also prevent the homeowner needing to switch back and forth between backup supply – although this could also be automated.

Figure 5-4 further indicates that the use of RWH for the topping up of pools will have limited stormwater management benefits, because even with 100% adoption by pool owners (Scenario 1), only a 2% reduction in runoff volume will be realised, and this will likely be early on in the winter / rainfall season. As the water demand for a particular scenario increases, a reduction in runoff is realised.

Results for Scenario 10 (maximum demand), indicate that a reduction of up to 47% of runoff volume could be realised (Figure 5-4). However, Figure 5-4 does not show whether the cost of developing and operating individual RWH systems is affordable. The small differences between Scenario 8 (supplying toilet flushing, washing machines, showers / bath only), Scenario 9 (supplying toilet flushing, washing machine, shower / bath and pools) and Scenario 10 (supplying toilet flushing, washing machine, shower / bath, pools and garden irrigation). This again supports the notion that RWH for the purposes of irrigation in a winter rainfall area provides little benefit.

Figure 5-5 presents the same analysis as per Figure 5-4 for Scenarios 11 through 20 (Scenarios 11 through 20 assume the RWH system is connected to the lesser of 100 m^2 or 50% roof area). The same trends in performance in Figure 5-4 are evident in Figure 5-5, however, the demand met, volumetric reliability and reduction in spillage are significantly less. It is evident (and logical) from Figure 5-4 and Figure 5-5 that RWH systems should be connected to as much of a property's roof area as possible. In doing so, it will be possible to more easily maximise the benefits of the system.

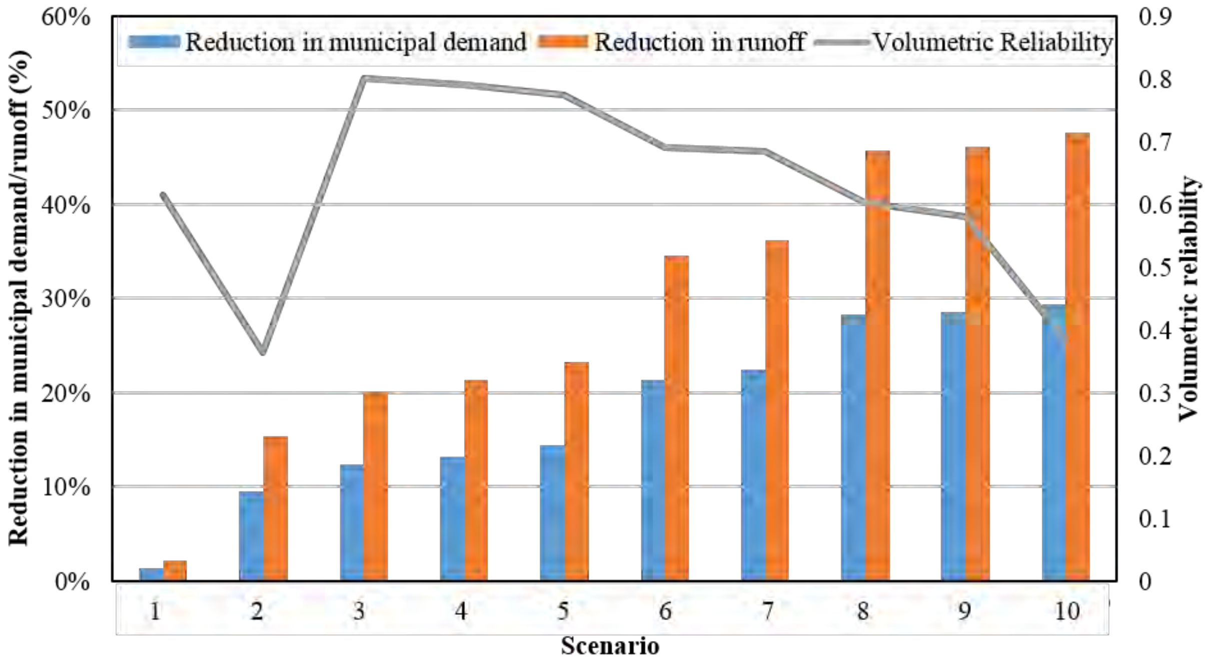

Figure 5-4: Potential reduction in demand through water savings as a percentage of total demand (Scenarios 1 through 10)

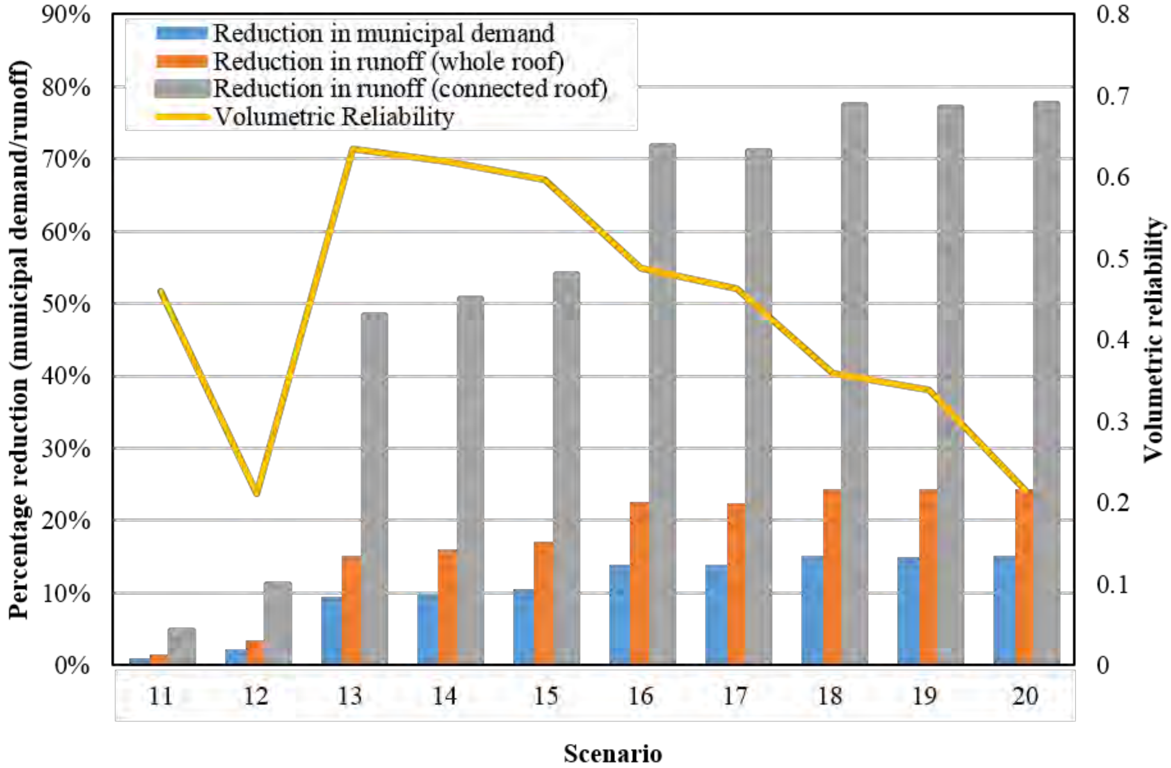

Figure 5-5: Potential reduction in demand through water savings as a percentage of total demand (Scenarios 11 to 20)

A significant challenge to the wider adoption of RWH in the RSA (and elsewhere) is that the cost of RWH typically has an inverse relationship with water demand, as highlighted in Figure 5-6. As a result, the CoCT's current block tariff structure, which has no charge for the first 6 kℓ/hh.mnth and then increasing unit rates as the monthly demand increases, acts as a disincentive to small users of water to harvest rainwater, even if, at a per capita level, they are using large amounts of water. Consequently, at current (2013) water tariffs, only 488 (8%) of the households within the catchment will likely be financially incentivised to install RWH systems assuming the adoption of Scenario 8 (supplying toilet flushing, washing machine, shower / bath only). This increases to 590 (9.5%) households, assuming the adoption of Scenario 10 (supplying toilet flushing, washing machine, shower / bath, pools and garden irrigation). Essentially, as a result of the relatively cheap cost of municipal water at the moment, RWH is not financially viable for the majority of households. However, to take only a financial view of RWH fails to recognise the additional purported benefits (Section 2.6.9) that may be realised when RWH tanks offer supplementary on-site storage that reduces the volume of runoff from the site. Whether RWH offers these purported stormwater management benefits is discussed in Section 5.1.4.

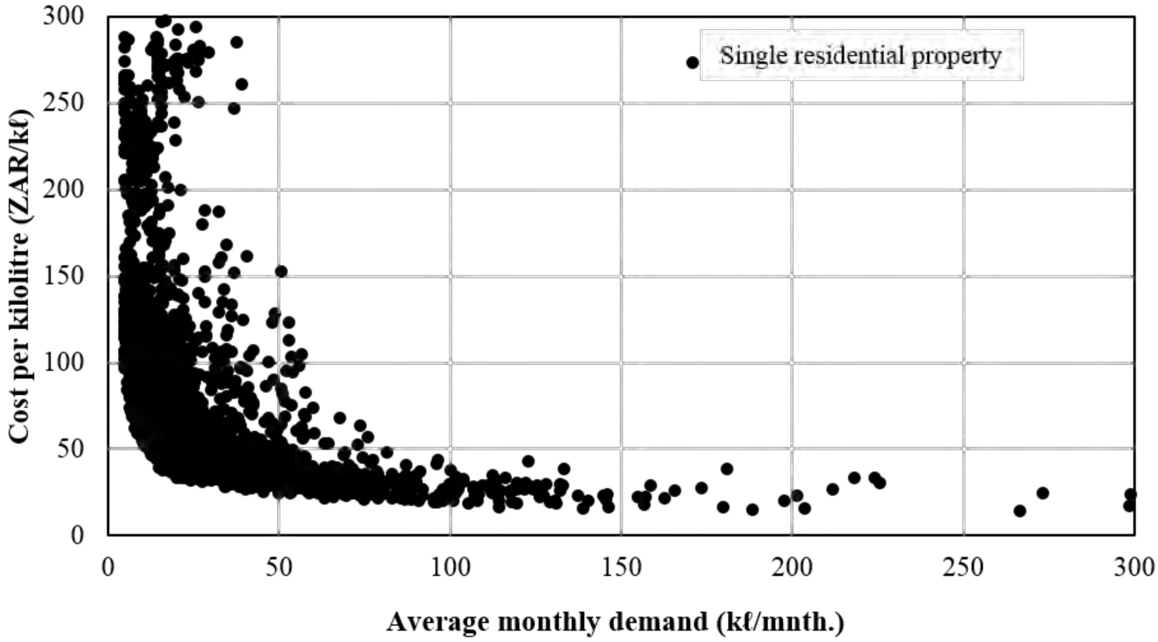

Figure 5-6: Monthly household demand (kℓ) vs. cost of harvested rainwater for each property in the Liesbeek River Catchment, Scenario 8 (Supplying toilet flushing, washing machine, shower / bath only)

Assuming the cost of installing and operating a RWH system is a significant driver for the adoption of RWH, Figure 5-7 shows the maximum annual potable water savings (Mℓ/yr.) that may be realised in the Liesbeek River Catchment depending on the rational selection of RWH by property owners – and based on the cost of municipal water in the catchment (this includes the sanitation charge, as the CoCT's sanitation charge is based on the volume of water supplied). The cost of water in Figure 5-7 is represented by a 'Tariff multiplier', i.e. the current tariff (Appendix K) multiplied by the tariff multiplier. This analysis was undertaken using OF C (as per the discussion in Section 5.1.1), and 100% adoption of RWH by property owners was assumed, where the cost of harvesting rainwater on their property is less than the cost of water supplied by the municipality. It showed that the current cost of water is unlikely to incentivise many property owners to adopt RWH under virtually any scenario. It is also evident that the more the rainwater is used for functions like flushing toilets and washing clothes, the lower the cost per kilolitre and the less the municipality would need to increase the tariffs in order to incentivise property owners to adopt RWH.

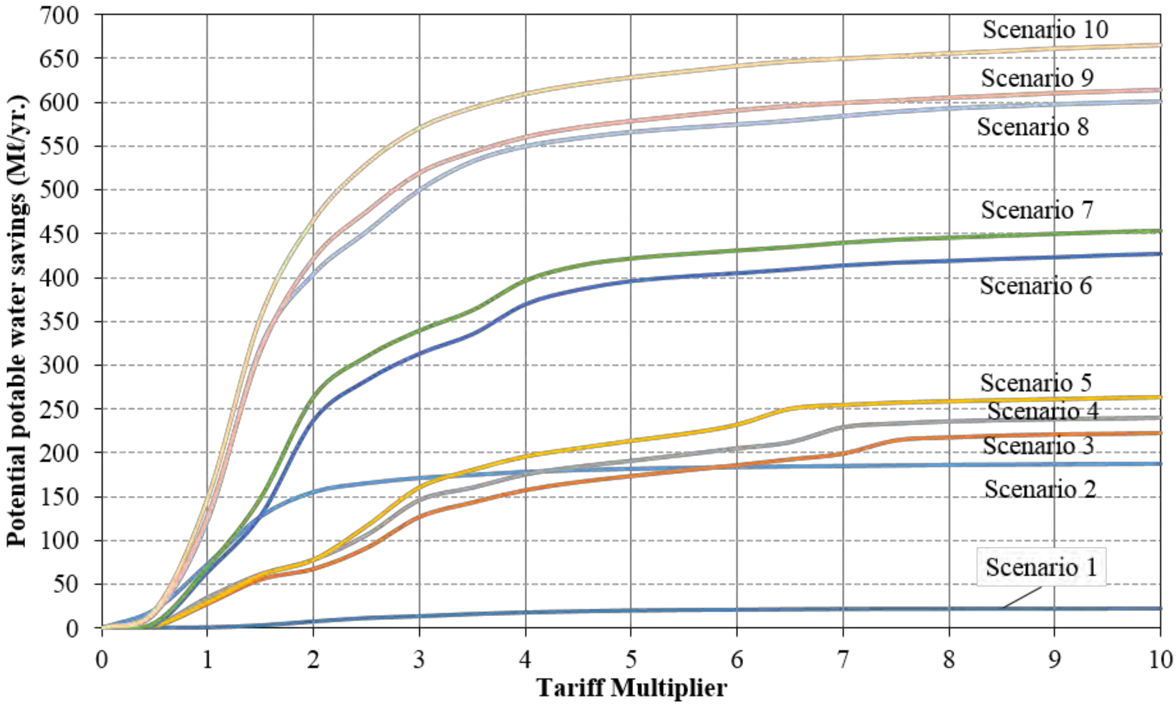

Figure 5-7: Maximum potable water savings (Mℓ/*yr.*) vs. cost of municipal water tariffs (Scenarios 1–10 using the full roof for collection)

Figure 5-7 suggests that Scenarios 1, 3, 4 and 5 would require significant increases in water tariffs before there is much likelihood of a significant reduction in the volume of water that needs to be supplied by the municipality as a result of the voluntary uptake of RWH by individual residents. Figure 5-7 suggests for Scenario 2 that doubling the water tariffs would achieve roughly the same savings as could be achieved by tripling the water tariffs for Scenarios 3 through 5. However, Scenario 2 assumes gravity irrigation and not post storage treatment. The use of gravity irrigation (including carrying buckets of water), from a social perspective, is unlikely in more affluent areas with bigger gardens. As a consequence, should RWH be considered in the CoCT and the rest of the RSA, it would be important to encourage the installation of systems in which water is used as diversely as possible (i.e. Scenarios 6–10).

This would reduce the scale of the required adjustment to the tariffs in order to incentivise users to adopt RWH while concurrently ensuring a greater reduction in demand for municipal water.

Figure 5-4 and Figure 5-5 highlight the importance, from a system performance perspective, of connecting RWH systems to as large a roof area as possible. This is reinforced when comparing Figure 5-7 and Figure 5-8. Figure 5-8 supports the contention that, considering the current water tariffs, not a single household in the catchment would be financially incentivised to adopt RWH, even under Scenario 20. It would require increasing the tariffs 10 times in Scenario 20 to achieve the same level of savings as increasing the tariffs 1.5 times for Scenario 10 (Scenarios 10 and 20 are identical except for the roof area which in Scenario 20 is limited to the lesser of 100m^2 or 50% of the total roof area).

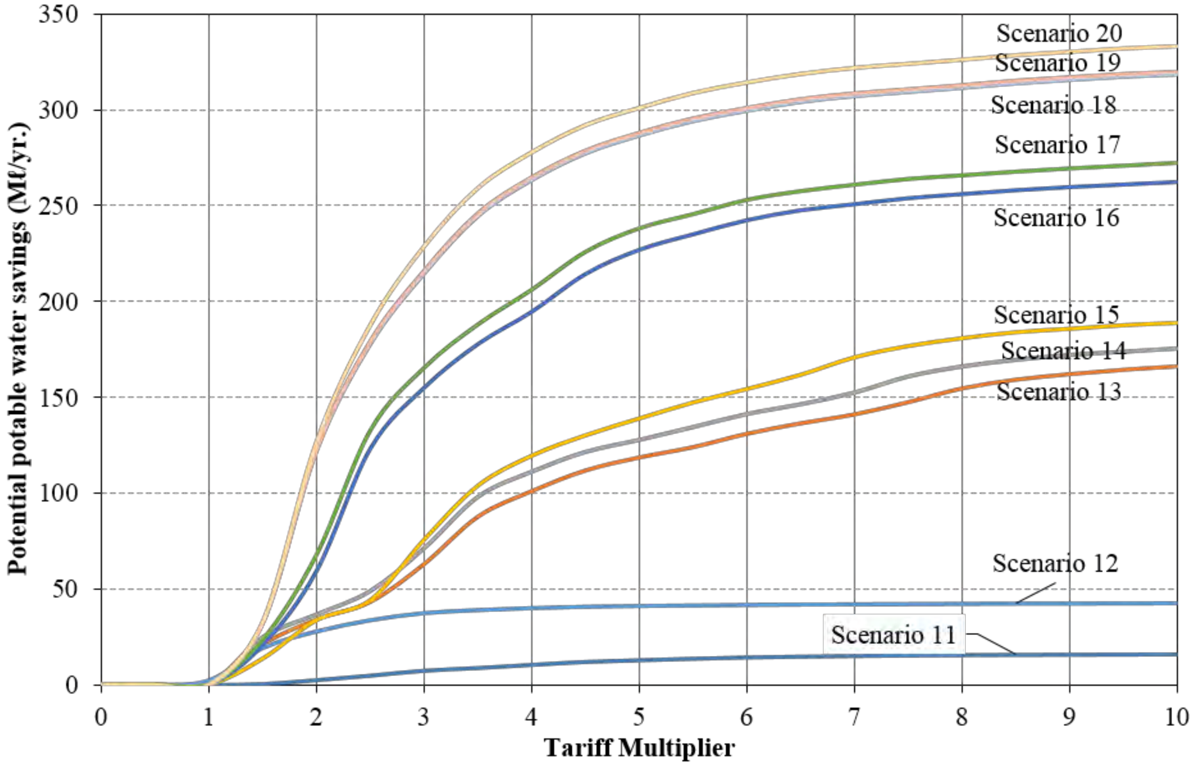

Figure 5-8: Maximum potable water savings (Mℓ/yr.) vs. cost of municipal water tariffs (Scenarios 11–20 using part of the roof for collection)

Table 5-1 shows an analysis with Scenario 8 (supplying toilet flushing, washing machine, shower / bath only) – for discussion purposes, as the trends were common for all scenarios – using OF C (system optimised to maximise volumetric reliability while ensuring the cost per kℓ of harvested rainwater is less than the average cost per kℓ of potable water from the CoCT), and assuming 100% adoption of RWH by property owners. The cost of municipal water is progressively increased as a multiple of the current tariff. Table 5-1 suggests that, in order to achieve a 43.5% adoption rate, assuming that all property owners would make the rational decision to adopt RWH if it were to be financially beneficial for them, would require the CoCT to roughly double the current water tariffs. However, if it is assumed that there is only a 50%

take-up from qualifying property owners, the CoCT would need to quadruple the current tariffs to achieve the same adoption rate.

The rate of increase in the level of adoption decreases with each subsequent increase in the water tariffs. This is because the majority of households would be incentivised to adopt RWH if the tariffs were to double or triple. Further increases in the tariffs incentivise fewer additional households to adopt RWH. This is in part due to the CoCT's stepped tariff structure which allows for the free provision of 6 kℓ of water to each household per month. This means that in the extreme, some households will simply never be financially incentivised to adopt RWH no matter what the scale of increase in tariffs is, unless the whole tariff structure were to be adjusted. The same trend is apparent in the increase in water demand met and reduction in runoff.

Table 5-1: Potential rainwater harvested as a result of incentivising property owners by increasing the cost of municipal water, Scenario 8, Objective Function C

Multiple of current tariff	Reduction in total domestic water demand (%)	Reduction in roof runoff (%)	Approximate level of adoption (%)
1	5.6	9.0	7.9
2	18.1	29.3	43.5
3	22.4	36.2	63.6
4	24.6	39.8	77.1
5	25.3	41.0	82.3
6	25.7	41.6	85.1
7	26.1	42.3	86.8
8	26.5	42.9	88.3
9	26.7	43.2	89.9
10	26.9	43.5	91.3

Figure 5-9 and Figure 5-10 graphically illustrate Table 5-1 (only Scenario 8) for Scenarios 1 through 10. Figure 5-9 presents the water demand met through RWH as a percentage of the maximum water demand for a scenario, assuming 100% adoption of RWH by property owners.

Figure 5-10 shows that doubling the tariffs will, for Scenarios 6 through 10, incentivise savings of between 60% and 80% of what is possible, and that this could be achieved through adoption rates as low as 30% to 50% of households adopting RWH.

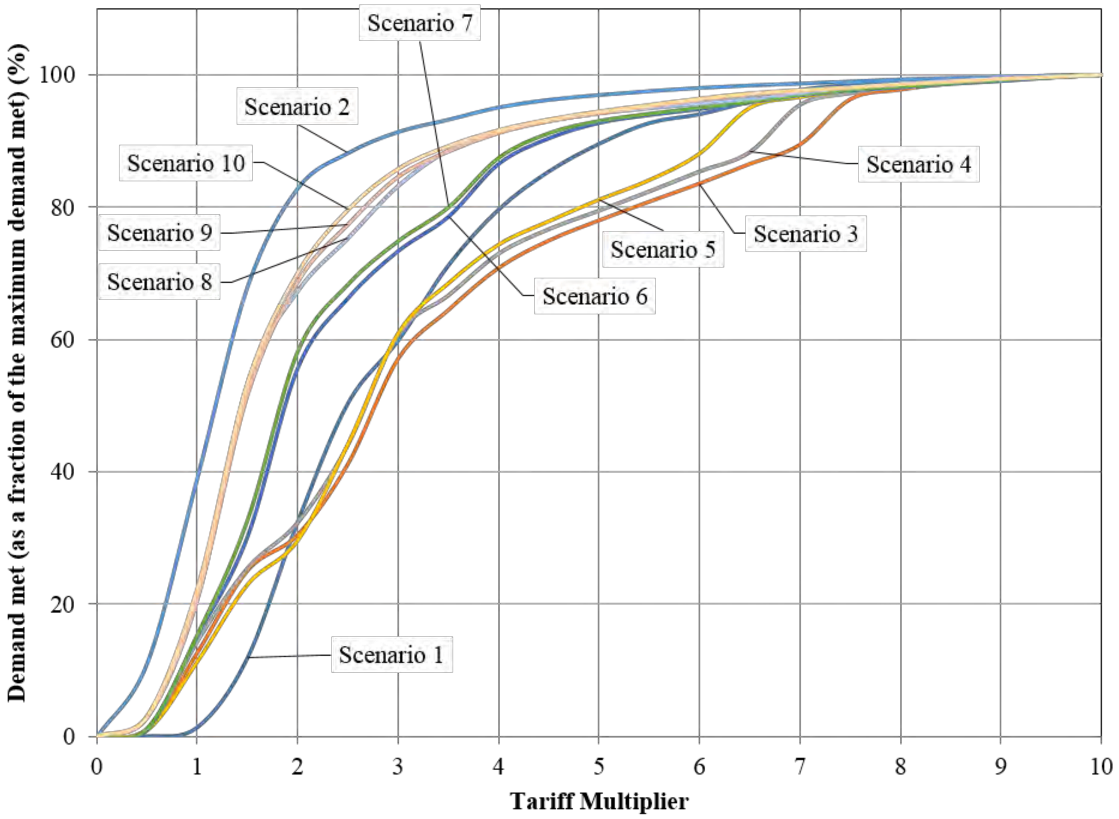

Figure 5-9: The impact of increasing tariffs on the water demand met through RWH. The demand met is given as a percentage of the maximum water demand maximum that could be met for a scenario, assuming 100% adoption of RWH by property owners.

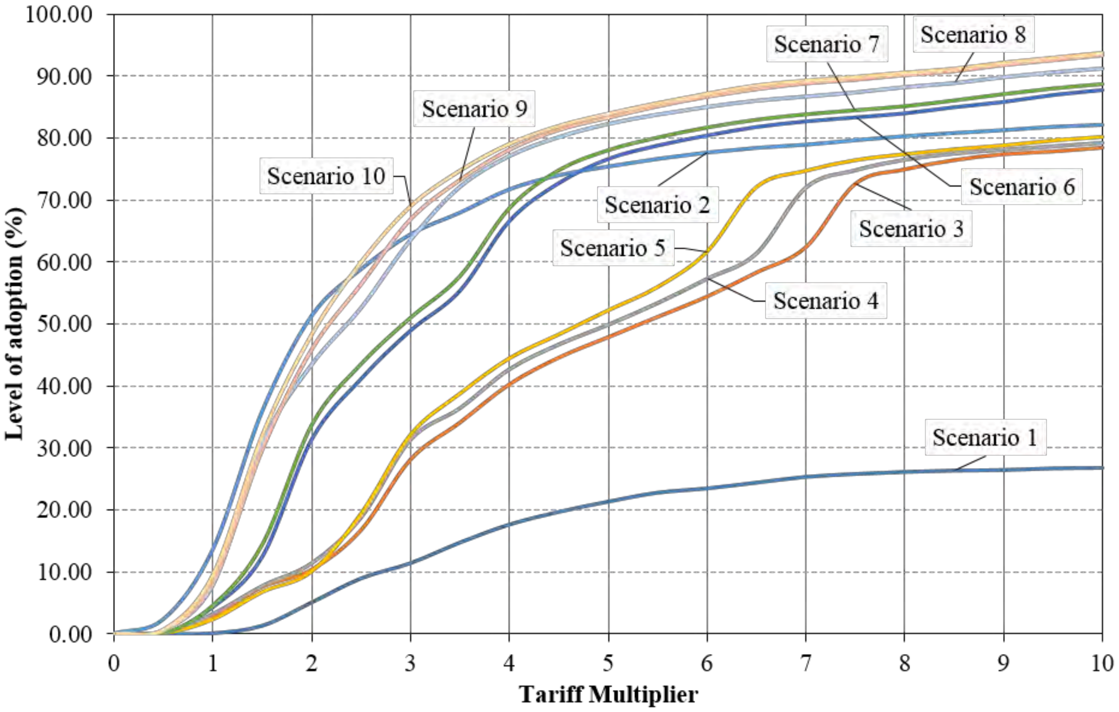

Figure 5-10: Impact of increasing tariffs on the adoption of RWH

Figure 5-11 indicates that the properties that are incentivised through the increase in tariffs are typically those that have higher water demands and are in wealthier suburbs, as would be expected. This is important, as it indicates that RWH is more appropriate for wealthier households. The focus of any educational, marketing or incentivisation scheme should be targeted at wealthier households.

An assessment of the overall results from the analysis of Scenarios 1 through 20 indicates that RWH is a viable option under the following conditions: harvested rainwater is used for as many end uses as possible, and the largest possible catchment area (as much of the roof area as possible) is connected to the RWH storage tank. If RWH were to be encouraged in the RSA, and the Liesbeek River Catchment in particular, it would need to be done in line with Scenarios 8 through 10, ideally Scenario 10.

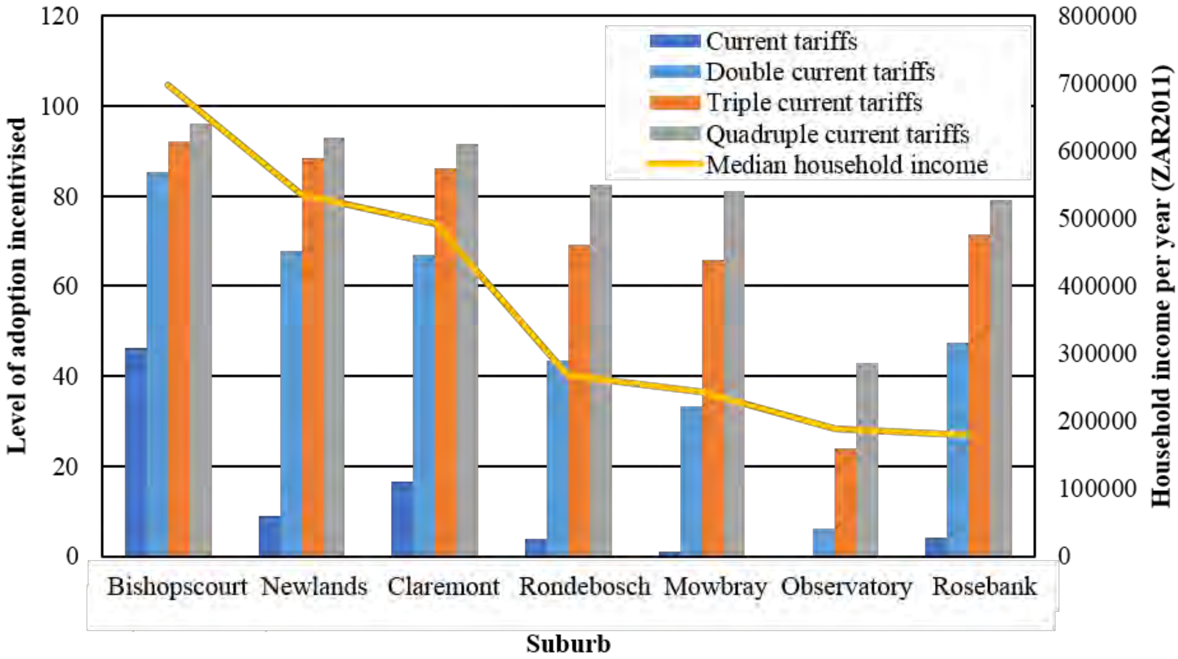

Figure 5-11: Suburb (as a proxy for wealth) vs. level of incentivised adoption

5.1.1.3 Simulating the adoption of rainwater harvesting

The results presented thus far assume 100% adoption of RWH. This is impossible in the short term and unlikely in the long term. As indicated in Table 5-1, even if the water tariffs were to increase 10 times, only approximately 90% of households would be financially incentivised to adopt RWH. In order to assess the impact of different levels of adoption on the overall performance of RWH at the catchment scale, a Monte Carlo simulation was undertaken, as described in Section 4.4.7.2, assuming different rates of adoption, from 10% to 100%. Figure 5-12 summarises the results for Scenario 10, highlighting that, at lower rates of adoption (percentage of properties in the catchment that adopt RWH), the expected reduction in demand can vary by more than 30% depending on which properties adopt RWH. At 50% adoption, the variation reduces to 15% and at 100% adoption to 0%. The large variation is a result of variation in household demand, which is most notable at low adoption levels. For example, if the 10% of households who adopt RWH are the 10% who use the most water, they will harvest and use significantly more water than if it were the 10% of households who use the least. However, at higher levels of adoption, this variation reduces until it reaches 100%, i.e. where everyone is harvesting rainwater.

The variation increased with scenarios that had fewer end-uses, especially at lower adoption rates. This is due to the selection of which properties have adopted RWH. For example when only 10% of households are simulated to have adopted RWH it is possible that the users simulated using RWH are the 10% using the most or least water. As the adoption rate increases, this effect is balanced out. The maximum variation was for Scenario 1 at 10% adoption, where the variation could be as much as 50%. Typically, though, by 50% adoption, the variation was between 12% and 18%. This is important to recognise, especially from an urban planning perspective, as at low levels of adoption, there can be a significant variation in the reduction in water demand, and attenuation of runoff.

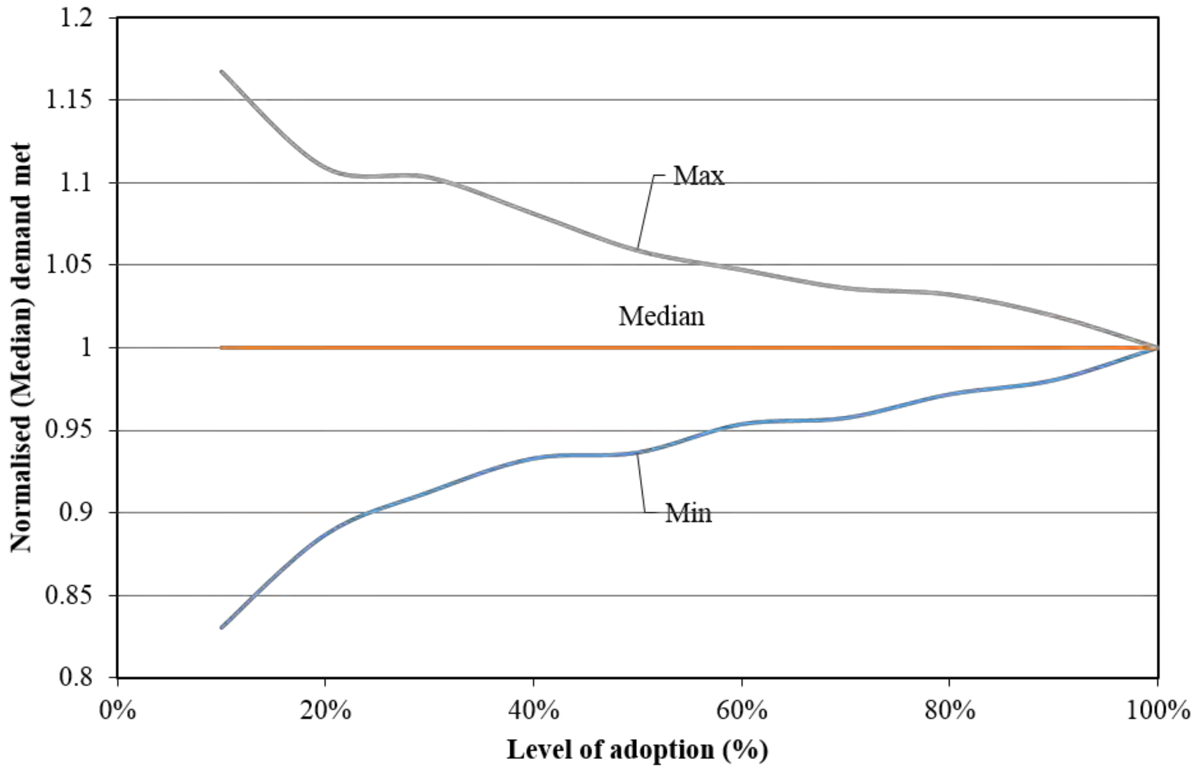

Figure 5-12: Variation around the expected mean water demand met depending on the level of adoption, Scenario 10

5.1.2 Climate change

Section 4.2.3.5 highlighted the fact that the viability of RWH will likely decrease as the climate changes. Consequently, an analysis of the impact of climate change on the viability of RWH, as discussed in Section 4.2.3.4, was undertaken. The results discussed in this section are based on Scenario 10, using set tank sizes optimised using historical climate data and OF B. The analysis simulated the changes in performance under climate change conditions (for the period 2050–2099) of RWH systems designed to optimise volumetric reliability under the current climatic conditions.

The results, unlike the results for rainfall and evaporation, present no clear trend and are rather confusing and contradictory. Figure 5-13 illustrates the potential change in volumetric reliability (2050–2099), compared with the volumetric reliability calculated from historic climate data (1979–2012), for each climate change scenario. While some climate change scenarios are suggesting increases of over 20% in volumetric reliability, there are others that indicate a decrease of the same magnitude. The average change in volumetric reliability is a 4% increase for properties simulated using the Kirstenbosch rainfall station and a 4% decrease for properties simulated using the Observatory rainfall station.

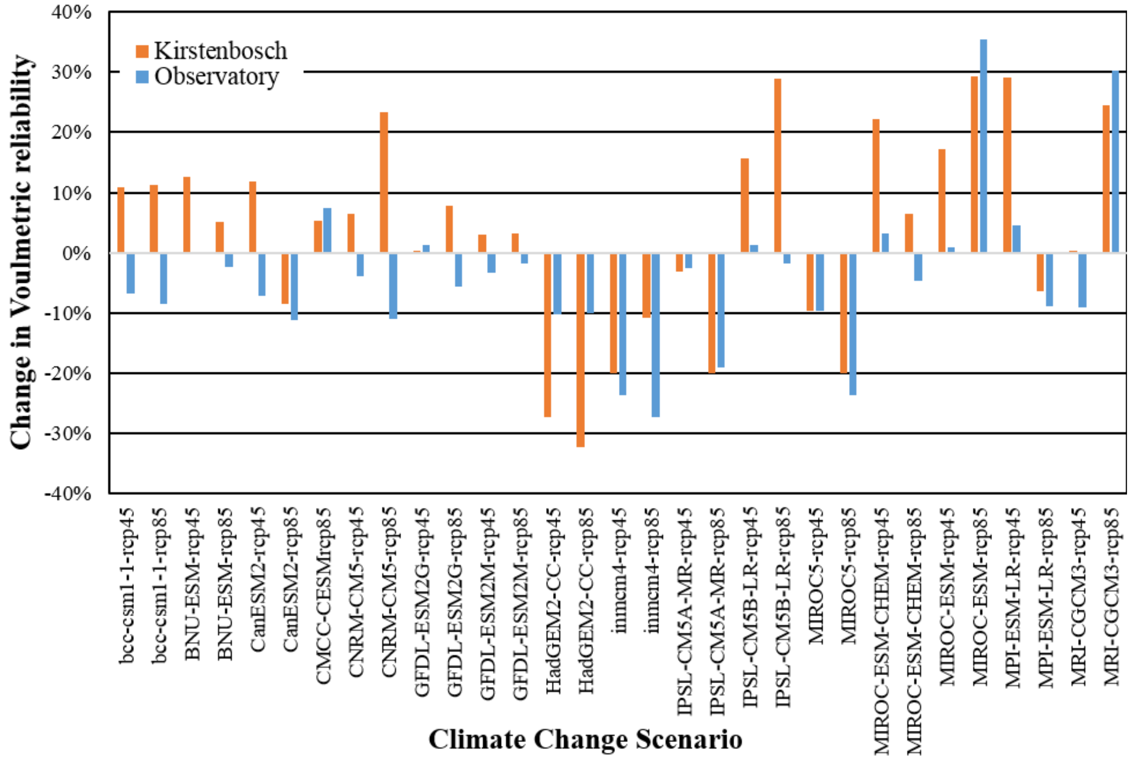

Figure 5-13: The impact of climate change on volumetric reliability (Observatory and Kirstenbosch rainfall stations)

Figure 5-14 illustrates the potential change in the cost per kilolitre of harvested rainwater, compared with the cost per kilolitre of harvested rainwater calculated from historic climate data (1979–2012) for each climate change scenario. Again, the results fail to provide a clear picture, and it seems that there is potential for extremely different results. Since the cost of RWH is closely linked to volumetric reliability, a change in volumetric reliability will therefore have cost implications that will vary from system to system. The change in cost is not linearly related to the change in volumetric reliability.

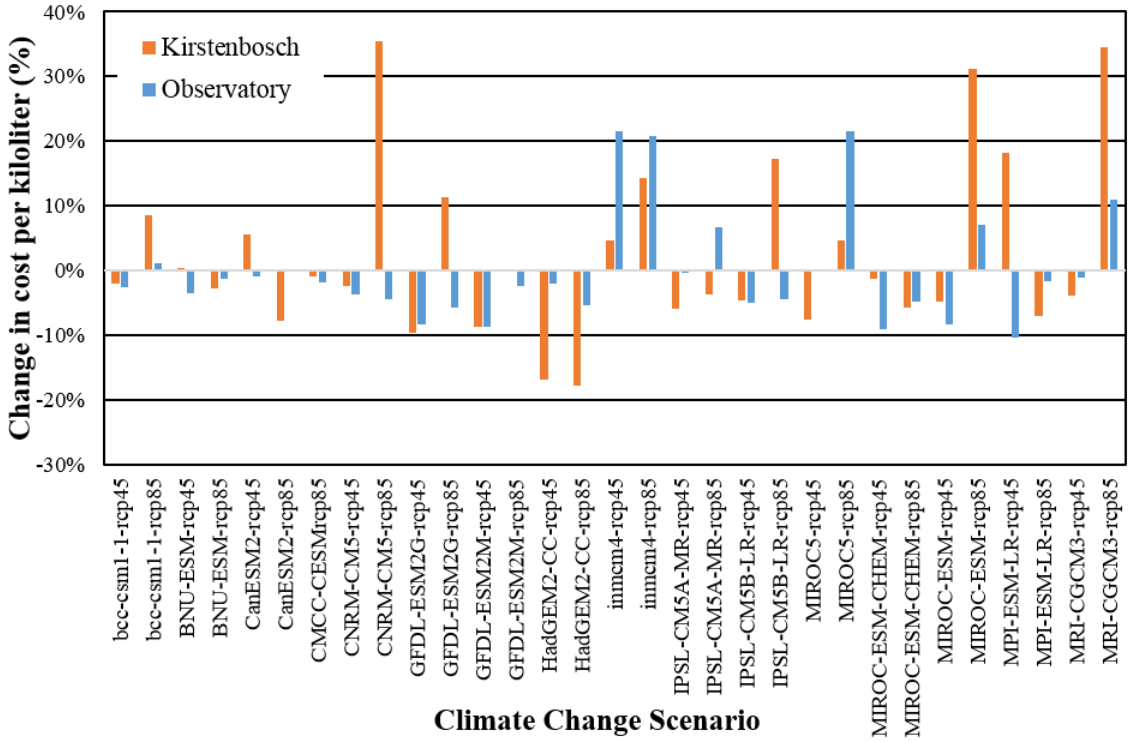

Figure 5-14: The impact of climate change on the cost per kilolitre of harvested rainwater (Observatory and Kirstenbosch rainfall stations)

Figure 5-15 illustrates the potential change in spillage, compared with the spillage calculated from historic climate data (1979–2012) for each climate change scenario. The general trend is for an overall increase in spillage for the Kirstenbosch station and a slight decrease in spillage for the Observatory station. This is expected due to the difference in expected changes in rainfall highlighted in Figure 4-7 and Figure 4-8.

5.1.3 Effect of economic changes

Section 5.1.1.2 identified Scenarios 8 through 10 to be the most appropriate, were RWH to be adopted in the Liesbeek River Catchment in particular and the RSA in general. In order to assess the implications of economic variability on the viability of RWH, a sensitivity analysis was conducted on these scenarios using discount rates of 3.1% to 4.5% (see Section 4.4.5.2). They were also modelled with the *URSHM* using set storage sizes, as calculated using OF B (Table 4-17) and a discount rate of 3.1%. The results presented in Figure 5-16 show the change in average cost per kilolitre throughout the catchment and indicate that an increase in the discount rate will increase the cost per kilolitre. The difference is approximately a 4.5% increase in the cost per kilolitre (between a discount rate of 3.1% and 4.5%). 3.1% was considered the most reasonable estimate of the discount rate (Section 4.4.5.2).

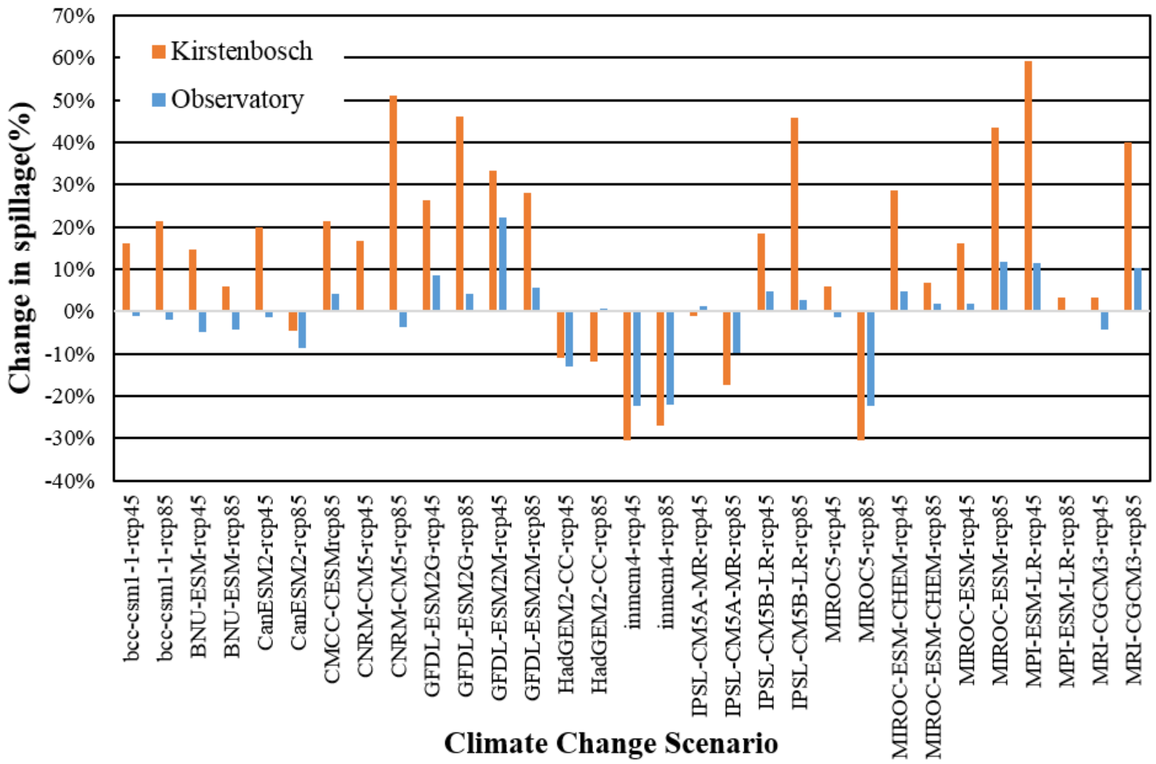

Figure 5-15: The impact of climate change on the volume of spillage (Observatory and Kirstenbosch rainfall stations)

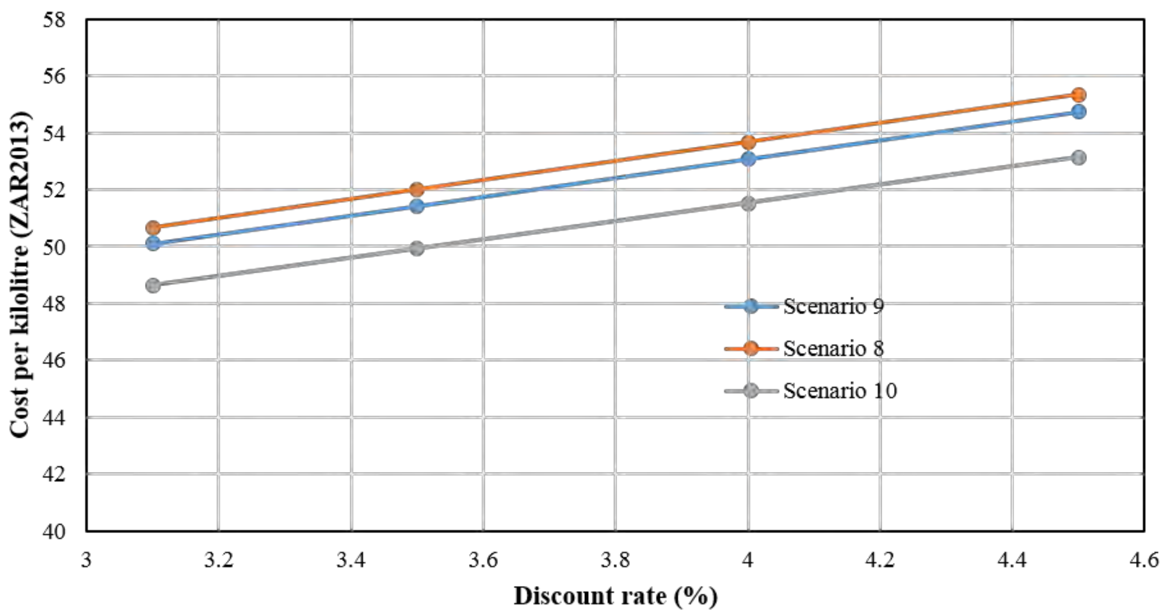

Figure 5-16: Sensitivity to changes in the discount rate

5.1.4 Stormwater management benefits

A commonly cited benefit in the literature, especially stormwater management manuals (e.g. Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013), is that RWH assists in stormwater management through attenuating peak flows and reducing runoff volumes, as discussed in Sections 2.6.9 and 2.4.5. As demonstrated in Sections 5.1.1 and 5.1.2, RWH has the potential to significantly reduce the total runoff volume from roofs in the catchment by up to 44%. However, while a reduction in total runoff is valuable from a conventional stormwater management perspective, the peak flow rate is the most important consideration from a flooding and risk point of view. The maximum potential reduction in peak flows would occur if there was 100% adoption (i.e. every property was harvesting rainwater) for as many end uses as possible, as this would ensure the maximum available storage for each storm event. Therefore, Scenario 10 was modelled in the Catchment Stormwater Model, as described in Section 4.4.8. Additionally, in order to minimise the effect that runoff from the mountain has on the scale of any reduction in peak flow, the results presented in this section, unless otherwise stated, consider runoff from only the urbanised catchment, as described in Section 4.4.8.3.

Figure 5-17 compares the modelled flow with and without RWH in the catchment. It is apparent that the effect of RWH is not constant and can vary substantially. This is in line with results presented in Campisano *et al.* (2014). Petrucci *et al.* (2012) noted that RWH may '*affect the catchment hydrology for usual rain events,* [but] *are too small and too few to prevent sewer overflows in the case of heavy rain'.* Figure 5-17 indicates that, while this may be true, it is not always so; in some cases, RWH makes no noticeable impact on the catchment hydrology, even for the more frequent rain events.

On the other hand, Figure 5-18 illustrates that, for storm events with a RI of less than one week, RWH could, in more than 50% of events reduce the peak flow by greater than 50%. However, the effectiveness of RWH quickly decreases as the RI increases. For events with a return period of 3 months, the reduction in peak flow in 17% of events is less than 2% and in 58% of events is less than 10%.

Figure 5-17 and Figure 5-18 consider the catchment runoff and peak flows, which includes runoff from all surfaces. Further analysis indicates that, even if only considering the runoff from roofs in a catchment, RWH is an unreliable tool for stormwater management. As an illustration of this, Figure 5-19 compares only the runoff from roofs in a selected subcatchment in the suburb of Newlands (S38 - Appendix O). While RWH manages to fully attenuate the one-week RI events, it is unreliable for RIs of greater than one week. For example, in more than 50% of six-month and one-year RI storm events, the peak flow from roofs is attenuated by less than 20%. This is significant, as the CoCT's 'Management of stormwater impacts policy' (CSRM, 2009b) encourages the use of SuDS (including RWH) to address a range of stormwater management objectives. Two relevant objectives are: firstly, to detain the one-year RI storm event on-site in order to reduce the downstream peak flows; and, secondly, to attenuate the peak flow of the ten-year RI storm event to predevelopment levels. While RWH could be used in conjunction with other SuDS, the problem is that its attenuation capabilities

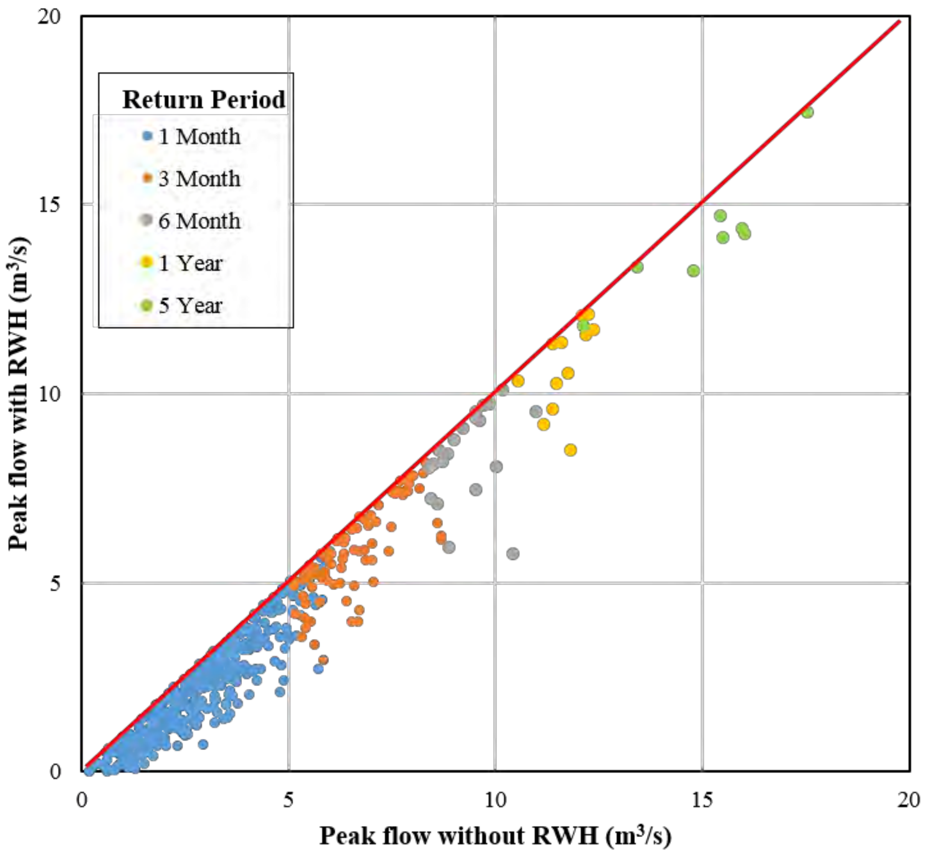

Figure 5-17: Comparison of modelled flows in the Liesbeek River with and without RWH, for events between 2003-2012

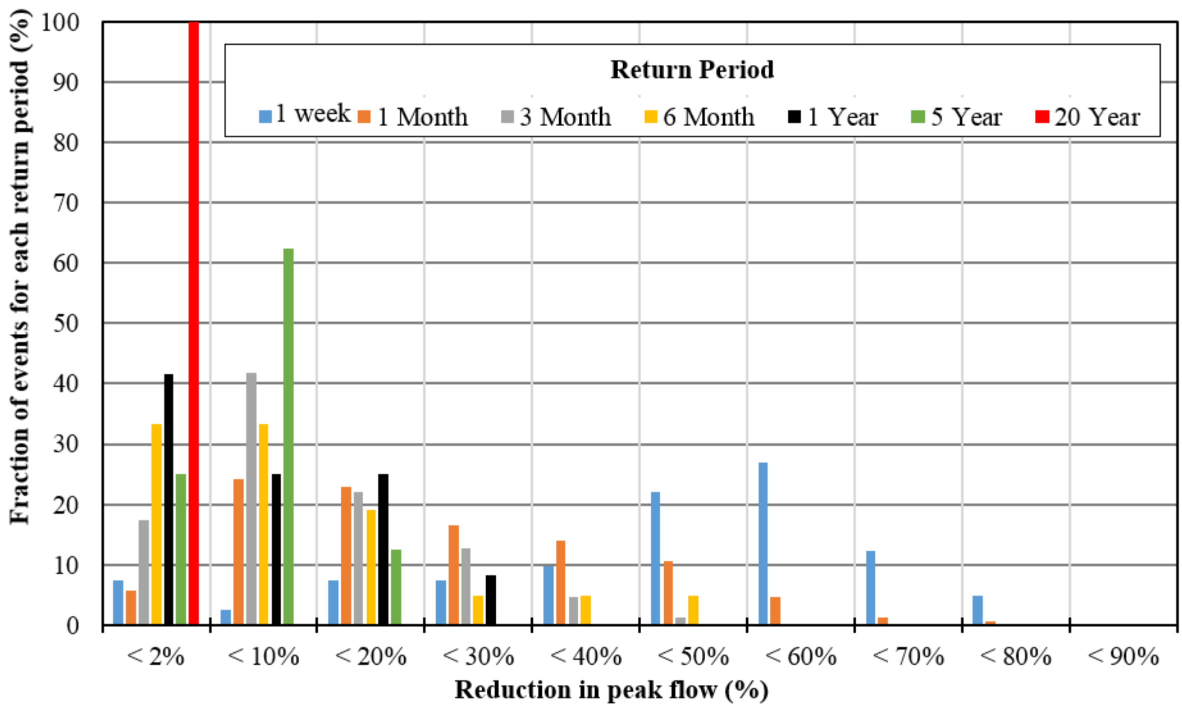

Figure 5-18: Distribution of the reduction of peak flow due to RWH in the Liesbeek River Catchment for different return periods, for all events between 2003-2012

are unreliable. It is apparent that RWH is an unreliable means of achieving the first objective and is incapable of achieving the second. This applies to both the site and catchment scales.

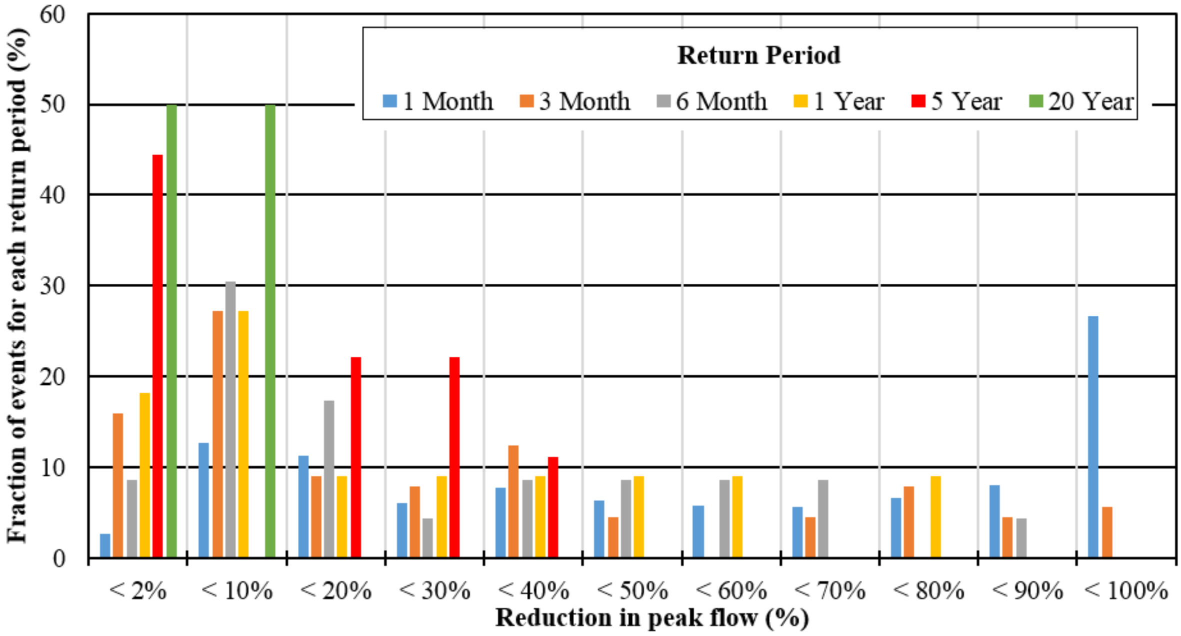

Figure 5-19: Distribution of the reduction of peak runoff from roofs in a selected catchment of the Liesbeek River Catchment due to RWH for different return periods, for all events between 2003-2012

Methods such as those used by Petrucci *et al.* (2012) to model the impact of RWH on a catchment's hydrology, where the initial storage is uniformly adjusted to account for the storage resulting from RWH in a catchment, essentially linearly upscales the functioning of a single RWH system – as previously discussed in Chapter 2, and will be discussed in Section 5.1.5, this has been shown to lead to errors in the modelling of the performance RWH systems.

The results presented thus far in this section assume that the same area remains directly connected to the drainage system (the percentage routed adjusted using Equation 4-26), and for purposes of discussion this is termed the 'expected case'. However, as discussed in Section 4.4.8, a 'worst case' would constitute properties that, prior to the installation of RWH indirectly connected their roofs to the drainage system (i.e. roofs drain to the garden), but as a result of the installation of RWH directly connected the tank overflow to the drainage system. Once the RWH tank is full, this would effectively increase the directly connected impervious area which was simulated using Equation 4-27. Under such conditions, the attenuation potential of RWH decreases significantly.

Figure 5-20 shows the modelled hydrographs for the Liesbeek River Catchment, with and without RWH, for a selected period of 36 hours with three storm events – extracted from the 10 years continuous simulation. The hydrographs labelled 'with RWH' were the result of modelling, where the catchment parameters were adjusted to represent the 'expected case' and the 'worst case' respectively. Figure 5-20 indicates that, for the first storm event (1), both the 'expected' and 'worst case' attenuated the peak flow. For the second storm event (2), however, while the 'expected case' offers limited attenuation, the 'worst case' results in an increase in the peak flow. This is significant because it is assumed that RWH attenuates peak flows. The worst case indicates that if, because of choice or pressure from a municipality, individuals install RWH and then connect the overflow directly to the drainage system – where previously the runoff was directed to a garden or other impervious area – there is the potential to increase the directly connected impervious area and consequently the peak flows. As a result, the CoCT and other municipalities should include in their stormwater management guidelines directions as to where the overflow from a RWH system should be directed.

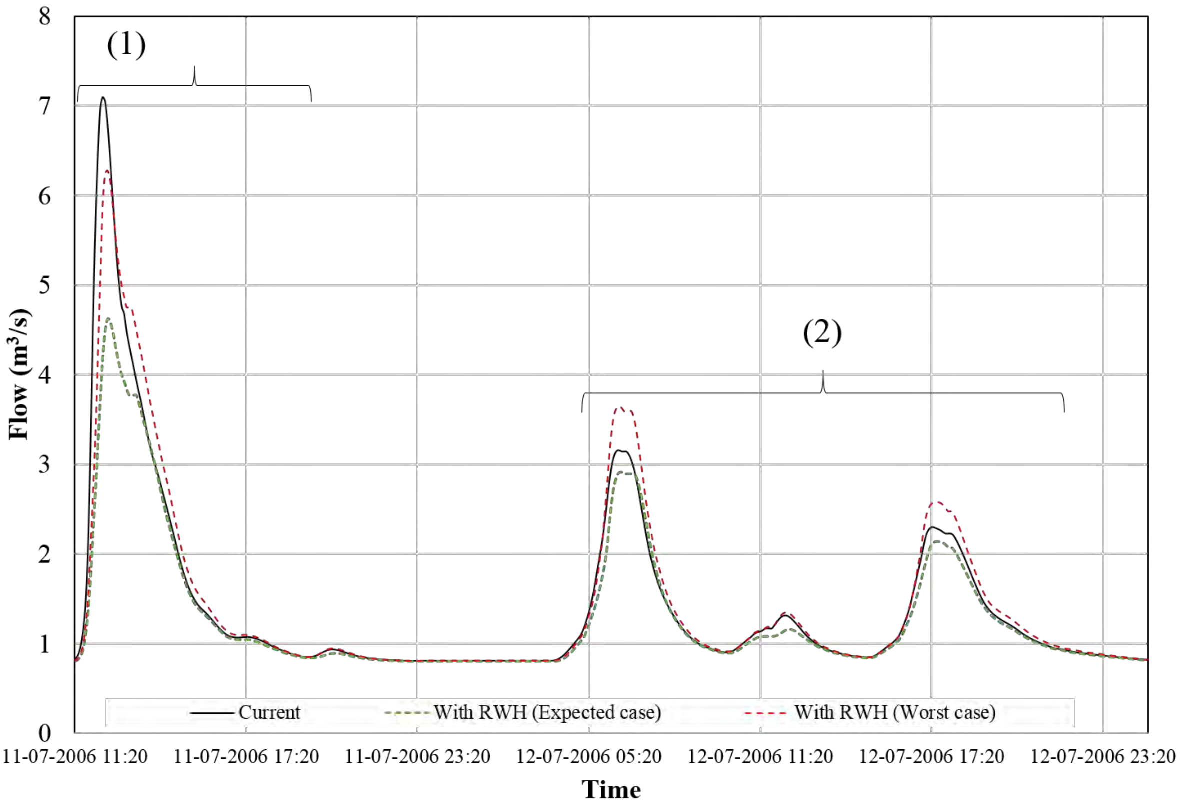

Figure 5-20: Hydrograph of the Liesbeek River with and without RWH

Figure 5-20 also explains the significant variation in attenuation, as illustrated in Figure 5-17 and Figure 5-18. With reference to the 'expected case' in Figure 5-20, the reduction in peak flow (2) is negated as the rainfall event follows a event (1) that has pre-filled the tanks in the catchment. This was observed throughout the 10-year simulation. This further supports the notion that it is unrealistic to consider RWH a reliable means of reducing peak flows (as per e.g. Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013). It is, however, evident that this is not always true. Rainwater butts – which are designed for short-term attenuation – will, if operating properly, be empty at the start of a storm event (assuming there had been sufficient time for the water butts to empty, e.g. 24 hrs), whereas, RWH is dependent on demand to empty the storage unit before the next storm event. As such, rainwater butts will, more consistently than RWH systems, offer attenuation to a design level. This function could always, through regulation, be incorporated into the design of RWH systems. For example, RWH systems could be designed with part of the storage acting as a rainwater butt.

While RWH may offer negligible peak flow attenuation, it will improve runoff water quality by intercepting pollutants prior to any spillage – captured in the coarse filter and/or first-flush filter. Dissolved pollutants will, however, not be removed, although this level of water quality improvement could be achieved in a cost effective manner – e.g. through the installation of coarse filters and/or first flush filters on the gutter downspouts.

5.1.5 Impact of modelling methods

Sections 2.4 and 2.6 highlighted a number of important modelling considerations that may influence the results of an analysis of the viability of RWH. Most important were the selection of an appropriate time step and the fact that the use of linear extrapolation to evaluate catchment-scale impacts of RWH may lead to overestimates of demand met and an underestimation of the spillage volume. These two concerns were investigated and are briefly discussed in this section.

5.1.5.1 Selecting an appropriate time step (Discussion)

The selection of an appropriate time step had a number of important implications for this research, including the optimisation of a RWH system, the accurate modelling of a single RWH system in isolation, and the accurate modelling of RWH in the catchment as a whole.

Table 5-2 presents the catchment-scale results for Scenario 10 (Table 4-20), with the storage size optimised using Objective Function (OF) B (Table 4-17) based on the results of the daily time step model. It is interesting to note the relatively minor difference in demand met (1%), cost per kilolitre (–0.9%) and volumetric reliability (1.4%) calculated using the daily and hourly time step models. There is, however, a significant difference in the percentage of dry periods calculated using the different time step models. This difference is the result of modelling at a finer time scale. For example, if half a day’s demand could be met, but not the whole day’s, the daily model would indicate that the demand was not completely met, and hence, the system ran dry. The hourly model could, however, indicate that, over the course of the day, the demand for 12 periods might well be met, as well as those 12 periods in which demand was not met and the system was dry. The differences in percentage collected are a result of modelling evaporation at a finer time scale, potentially allowing for the wetting and drying of the roof within a day (see Section 2.4.2.3). The system thereby realises greater losses and reduced collection.

It is, however, important to recognise that the differences reported in Table 5-2 are at the catchment scale and that, at the individual property scale, the differences in performance can be a lot more significant. Figure 5-21 illustrates the range in results of modelling using two different time steps at the system scale. It is evident that there can be significant variations in the modelled performance at the system scale. For example, the performance of small systems (e.g. 0.5 kℓ) was typically underestimated. Due to the sizes of these systems, they have little impact on the catchment-scale results, but there could potentially be significant impacts on the cost of operating the system for the individual RWH system owner, since an underestimation of demand met will result in an overestimation of the cost per kilolitre.

Table 5-2: Comparison of modelled performance using daily and hourly time step models at the catchment scale (using storage sizes in optimised daily time step model)

Performance parameter	Daily time step model	Hourly time step model	Difference (%)
Demand met (Mℓ/yr.)	601	596	1.0
Total storage volume (m^3)	37,965	37,965	0.0
Volumetric reliability	0.35	0.34	1.4
% dry periods	56	52	7.5
% collected	44	46	-3.1
Cost per kilolitre (2013ZAR/kℓ)	50	50	-0.9

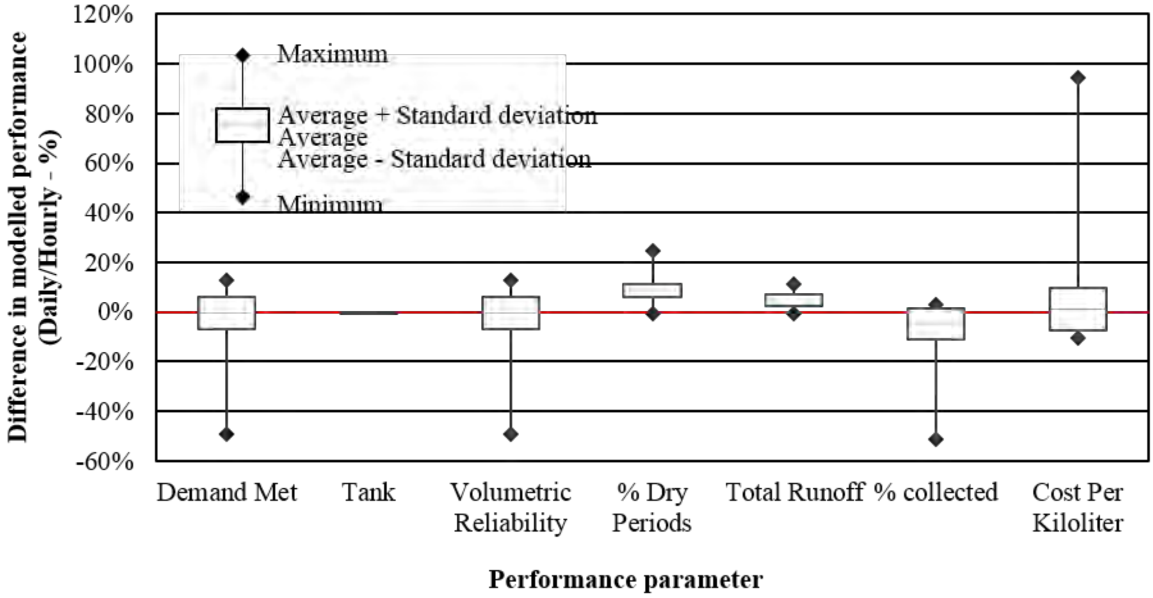

Figure 5-21: Comparison of modelled performance using daily and hourly time step models at the property scale (using storage sizes in optimised daily time step model)

Differences in the modelled performance of an RWH system using the hourly and daily time steps could potentially be minor, but they could still result in the optimisation functions (Section 4.4.7) selecting an alternative RWH system. Table 5-3 presents the catchment-scale results for Scenario 10 (Table 4-20), with the storage size optimised using Objective Function B (Table 4-17) based on the results of the daily and hourly time step models, respectively. It is interesting to note the reduction in storage volume (9.3%) as well as the slight increase, when compared with Table 5-2, in difference in demand met and volumetric reliability, percentage collected and cost per kilolitre between the hourly and daily time step models. On the other hand, the selection of a larger or smaller time step can have a significant impact (up to 200%) on the individual system's storage. This, in turn, impacts all other performance parameters, as illustrated in Figure 5-22.

Table 5-3: Comparison of modelled performance using daily and hourly time step models at the catchment scale (using storage sizes optimised by model)

Performance parameter	Daily time step model	Hourly time step model	Difference (%)
Demand met (Mℓ/yr.)	601	585	2.9
Total storage volume (m^3)	37,965	34,750	9.3
Volumetric reliability	0.35	0.34	3.2
% dry periods	56	53	6.1
% collected	44	45	-1.4
Cost per kilolitre (2013ZAR/kℓ)	50	51	-1.8

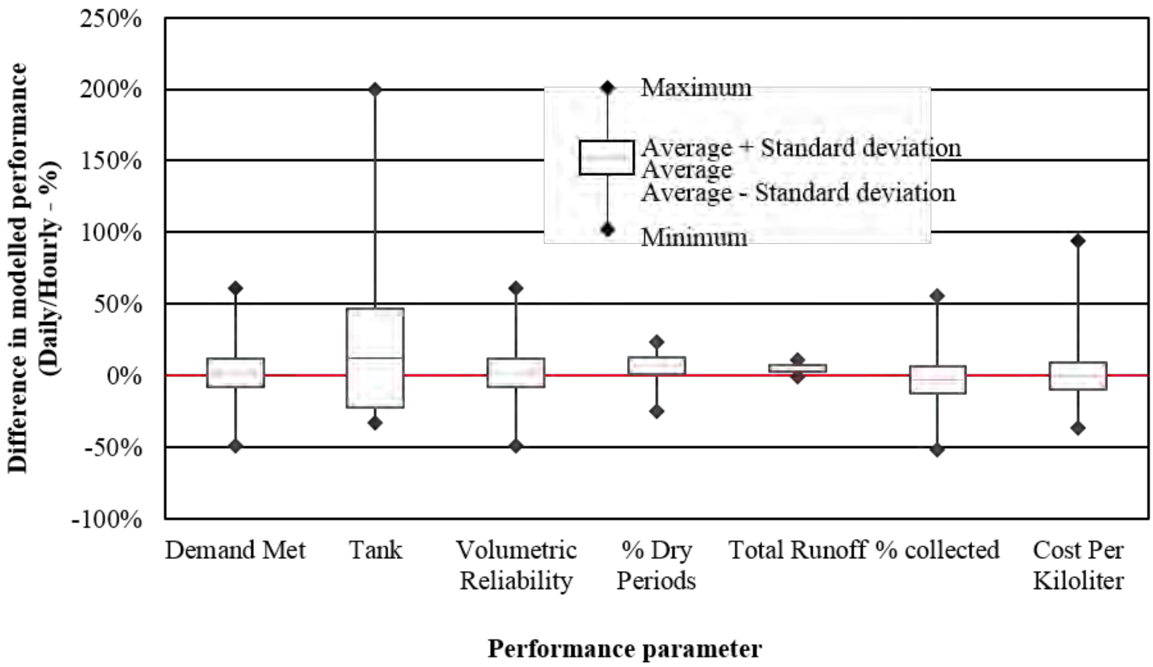

Figure 5-22: Comparison of modelled performance using daily and hourly time step models at the system scale (using storage sizes optimised by model)

5.1.5.2 The use of linear extrapolation

Section 2.4.5 highlighted a number of studies that indicated that linearly extrapolating site-scale results for RWH could lead to potentially significant errors in the modelled performance. In line with Neumann *et al.* (2011), both the geometric mean and arithmetic means – at suburb and catchment scale – were used to estimate the average parameters for modelling a single RWH system. The performance results (e.g. demand met, volumetric reliability etc.) were then linearly extrapolated in two ways:

i) The performance results were linearly extrapolated from the property scale to the catchment scale – termed extrapolating to the catchment scale.

ii) The performance results were linearly extrapolated from the property scale to the suburb scale according to a typical system for each suburb. The performance results from the different suburbs were then aggregated to provide catchment scale performance results. This was termed extrapolating to the suburb scale.

It was decided to investigate the impact that scaling to the suburb scale would have, as it was evident that there were significant variations in the parameters (e.g. roof area) between suburbs. In Section 4.2.4.2, it was assumed that socio-economic conditions within each suburb were homogeneous and, thus, the RWH systems would be more similar in use and operation and could potentially minimise the errors that accrue in extrapolation. The discussion that follows indicates that this assumption was reasonable.

Figure 5-23 presents the error in volumetric reliability that results from linearly extrapolating to the catchment scale for Scenarios 1 through 10 using a daily time step, and for Scenario 10, modelled using an hourly time step – based on the assumption that the modelled performance, with each property modelled separately, reflects the most accurate modelling possible (see Chapter 4). It is evident that the use of the arithmetic mean typically results in an overestimation, while the use of the geometric mean results in an underestimation of the volumetric reliability. Also, extrapolating to the suburb scale and then aggregating to the catchment scale typically provides a better estimate of a system's volumetric reliability than extrapolating directly to the catchment scale, whether using the arithmetic or geometric means of the input data.

Figure 5-24 presents the error in spillage that results from linearly extrapolating performance results to the catchment scale. It appears that linearly extrapolating the site scale results to the catchment scale can lead to a significant underestimation of spillage (3.5–35%, typically 7–18%). Using the geometric mean values to extrapolate to the catchment scale – not shown in Figure 5-24 – resulted in errors exceeding 75%.

The errors presented in both Figure 5-23 and Figure 5-24 are within the ranges of those presented in other studies (Table 2-8).

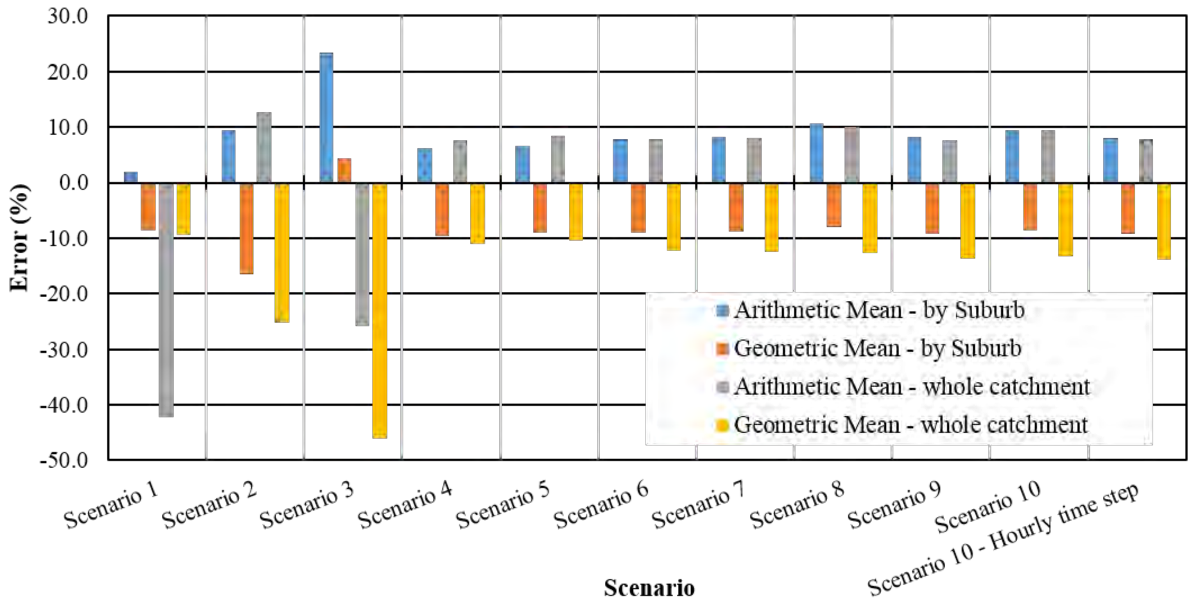

Figure 5-23: Error in volumetric reliability as a result of linearly extrapolating an RWH system's performance to the catchment scale

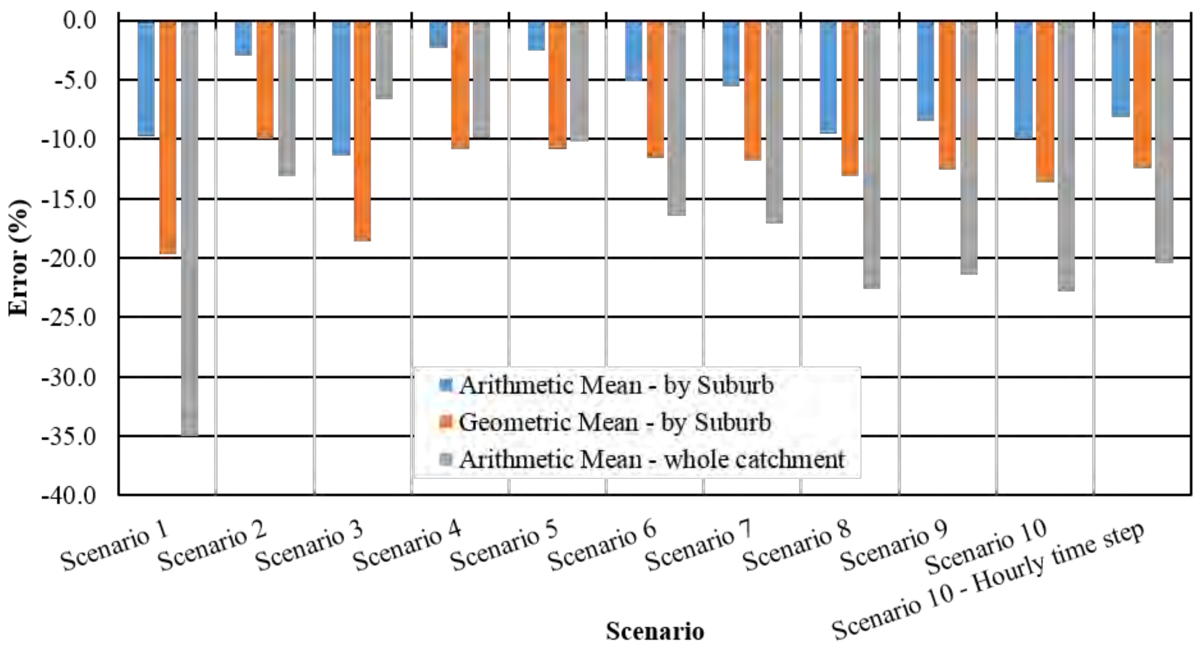

Figure 5-24: Error in spillage as a result of linearly extrapolating an RWH system's performance to the catchment scale

5.1.5.3 Summary and discussion of the impact of modelling methods

The methods and spatial scale used in modelling RWH can have an impact on the results of an analysis. It is apparent that, while at the property scale – using the same storage size – there can be significant differences in performance when using hourly time steps in comparison to daily time steps; at the catchment scale, the differences are small (demand met, 1%; volumetric reliability, 1.4%; percentage collected, 3%) and considered acceptable. The most significant difference is found in smaller systems, as expected (Fewkes, 1999), where the system could potentially fill and empty multiple times in a single day. The time step can, however, have a significant impact on the optimisation and selection of the storage size of an individual system. In the RSA, RWH systems are typically sized based on daily demand simulations; therefore, it would seem rational to size systems based on the results of modelling using a daily time step.

It is evident that the use of linear extrapolation to infer the catchment-scale impacts of RWH is likely to lead to errors. Based on the analysis conducted in the Liesbeek River Catchment, the error in estimating volumetric reliability typically ranges between 8% and 9%. The error in estimating spillage typically ranges between 7% and 18%. While extrapolating to the suburb level improves the accuracy of the results, there remains an inherent error. Additionally, the above linear extrapolations have the advantage of being based on the mean (arithmetic or geometric) data from every household; where only a sample of data is used, the errors could potentially increase.

5.1.6 Viability of RWH: Summary and discussion

It is clear from the analysis that RWH is generally not a financially viable option for the majority of households due to the cost of installing and maintaining RWH systems compared with the benefit of the likely reduced water bills. Nevertheless, if property owners harvest runoff from the majority of their roof areas (as for scenarios 1–10) and use water for a diversity of end uses (e.g. scenarios 8–10), RWH is potentially a financially viable option for between 8% and 9.5% of households in the catchment. This would equate to approximately 7% of total residential water demand. If the municipality wishes to incentivise the wide spread adoption of RWH by making it more financially attractive, it would need to increase water tariffs by between two to four times what they currently are. Increasing the tariffs by more than four times will yield relatively limited additional benefits.

Climate change is typically a concern for water resource planners. The analysis of 31 different climate change scenarios demonstrated that, above all, the future is uncertain. While some climate change scenarios indicated significant decreases in runoff, others showed limited change. Overall, it seems reasonable to expect a slight decrease in volumetric reliability in the lower reaches (Observatory) of the catchment and a slight increase in volumetric reliability in the upper reaches (Kirstenbosch). The change in cost per kilolitre is inversely linked to volumetric reliability; as such, it is likely to decrease wherever volumetric reliability increases and vice versa.

RWH is often considered an on-site stormwater management tool (see Sections 2.6.9 and 2.4.5) and is highlighted as such in some stormwater management guidelines. This study, however, suggests that it would not be particularly effective in doing so in the Liesbeek River Catchment. While it does reduce the volume of runoff and may attenuate peak flows, it fails to consistently attenuate the peak flows of storms with a RI of greater than one week. With this in mind, it is unreasonable in general to consider RWH as having any significant stormwater management benefits. It is true that RWH improves water quality by intercepting pollutants prior to any spillage. However, dissolved pollutants will not be removed, and there are alternative means of removing pollutants that may be more cost effective for the individual.

All things considered, RWH primarily offers a means of reducing municipal water demand, with negligible stormwater management benefits. Currently it is only financially viable for the minority of property owners, most commonly the more affluent households. RWH is generally only financially viable under the following conditions:

- Harvested rainwater is used for as many end uses as possible.
- The largest possible catchment area (as much of the roof area as possible) is connected to the RWH storage tank.

5.2 Viability of stormwater harvesting

The following section discusses the modelling and viability of SWH in the Liesbeek River catchment, including, *inter alia*, the impact of modelling methods, the impact of climate change on the viability of SWH, and stormwater management benefits. The viability of SWH was considered for both a centralised and decentralised approach. The scale of the decentralised approach was found to be important to its economic viability – as is discussed in Section 5.2.1.2. Initially, for the decentralised approach to SWH, each stormwater runoff sub-catchment in the urbanised part of the catchment – equivalent to a watershed – was considered to be a water supply catchment for a separate SWH scheme. Within the urbanised area of the Liesbeek River Catchment there were 130 such subcatchments (Figure 5-25). However, as discussed in Section 5.2.1.2, it was found that larger SWH schemes proved to be economically more viable. As a result, through an iterative process of combining adjacent subcatchments, with careful consideration of the drainage patterns – i.e. the different subcatchments should be able to drain to a single point – the initial 130 subcatchments in the Liesbeek River Catchment were combined into 30 larger subcatchments which were considered as independent SWH schemes with a storage pond situated near the river at each subcatchment's lowest point (Figure 5-25). Throughout Section 5.2, when referring to decentralised SWH – Scenarios 21, 23 and 25 in Table 4-21 –unless otherwise stated, reference is made to decentralised SWH with 30 SWH schemes.

For ease of reference, Table 4-17 which details the different Objective Functions (OF) incorporated into the *URSHM*, and Table 4-21 which details the different scenarios used to assess the viability of SWH, have been repeated below.

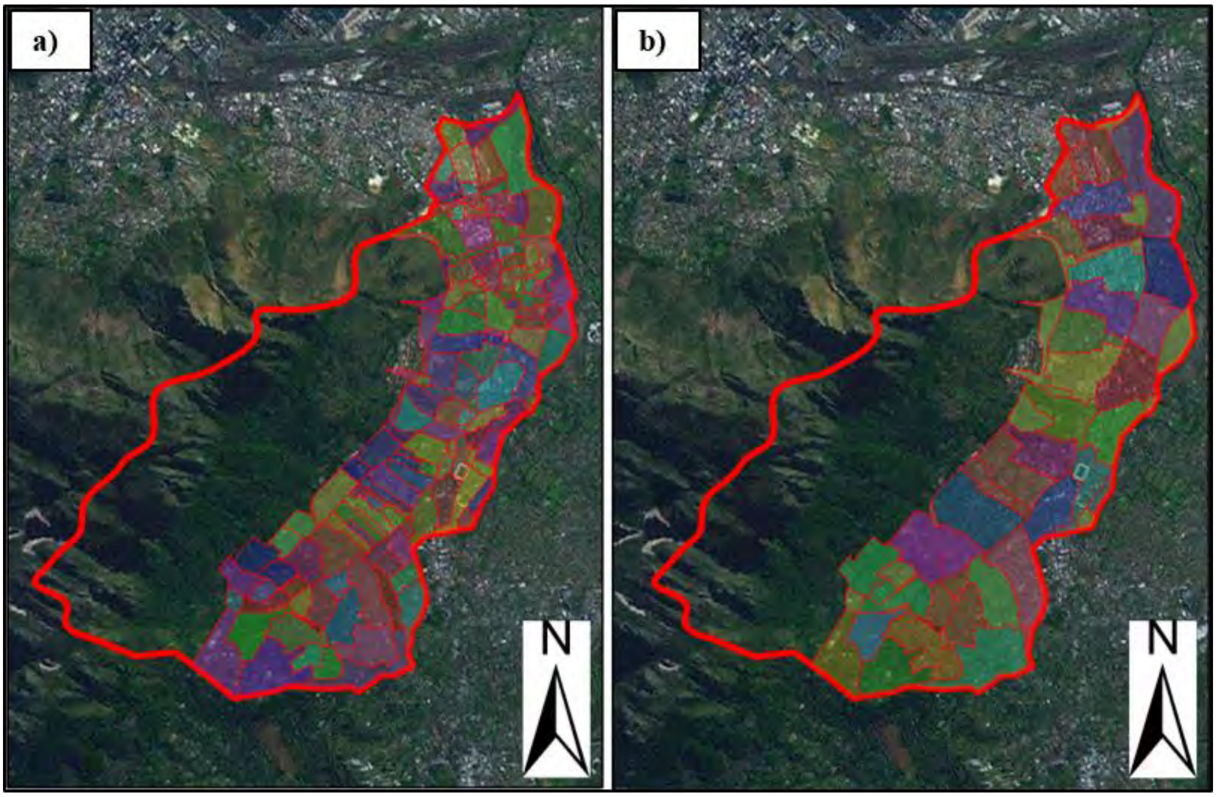

Figure 5-25: Urbanised area of Liesbeek River catchment delineated into a) 130 sub-catchments, and b) into 30 subcatchments as a result of combining subcatchments in (a)

Table 4-17: System optimisation objective functions

Objective Function	Description	Rational motivation for selecting system Using objective function
Objective Function A	System optimised to minimise the cost per kℓ of harvested rainwater	Minimal negative financial impact on the end user if a municipality forces the adoption of RWH/SWH.
Objective Function B	System optimised to maximise volumetric reliability	Provides maximum water supply security. May be appropriate in areas where water supply is intermittent.
Objective Function C	System optimised to maximise volumetric reliability while ensuring the cost per kℓ of harvested rainwater is less than the average cost per kℓ of potable water from the CoCT	Where the adoption of RWH/SWH is left to the end user/s, who is/are motivated primarily through the potential to make financial savings. This objective function may result in a substantial number of individuals not adopting RWH/SWH if the price of water is too low.
Objective Function D	System optimised according to user selection weighting of the cost per kℓ and the volumetric reliability. Default setting assumes equal weighting.	Where financial concerns and water security concerns need to be balanced. Essentially combines objective functions A and B.

Table 4-21: Scenarios 21 through 26 (stormwater harvesting)

Scenario	End-use water demand description
Scenario 21	Gardens (at subcatchment scale)
Scenario 22	Gardens (catchment scale)
Scenario 23	Gardens and pools (at subcatchment scale)
Scenario 24	Gardens and pools (catchment scale)
Scenario 25	Gardens, pools and toilets (at subcatchment scale)
Scenario 26	Gardens, pools and toilets (catchment scale)

5.2.1 SWH in the Liesbeek River Catchment

The following section presents and discusses the results relating to the viability of SWH in the Liesbeek River Catchment.

5.2.1.1 System optimisation

For each scenario, the system was optimised based on the objective functions in Table 4-17. The storage size was presumed to be a pond with an average depth of 1.5m – in line with the standard design in the South African Guidelines for SuDS. The *URSHM* was used to optimise SWH systems. Figure 5-26 shows the SWH storage sizes – optimised using Objective Function A, C and D – as a percentage of total catchment area. Figure 5-26 does not show the results for OF B, which optimises the system for volumetric reliability, as this results in unrealistically large storage units for SWH systems in excess of 10% of total catchment area for all catchments. It is possible to reduce the SWH system size by increasing the depth of the storage pond which would have the added benefit of reducing evaporation losses, however the increased risks of, *inter alia*, drowning would need to be considered. Additionally, near the river, the water table is approximately 0-5 m below ground level. An average depth of 1.5 m implies that in areas the depth of the pond is greater than 1.5 m, while in other areas (typically near the edges) the pond is shallower. The intention of these ponds is not to intercept the water table (which could result in evaporation losses during summer), or impact on groundwater flows (blocking flow paths). It was decided that an average depth of 1.5 m was reasonable for modelling purposes.

OF A, which optimises a system to minimise the cost per kℓ of harvested stormwater, is appropriate because the increase in storage size is a relatively minor cost. Unlike RWH, where the end user may find that increasing the system size and a consequent decrease in the unit cost of harvested rainwater would not increase their overall savings, in the case of SWH a cheaper unit price is better for the end user. This is due to the CoCT's stepped water tariff structure. Owing to the fact that OF B optimisation leads to storage sizes in excess of 10% of the catchment it was deemed not appropriate for optimising SWH systems. This is because it would take up the majority, if not all, of the open space typically allowed for in urban settlements (internationally between 10% to 17% of a development (CSIR, 2005a)). OF C ensures that the maximum benefit is accrued (note the increasing volumetric reliability decrease cost per kℓ) and that selecting a larger storage size will not result in an overall increase in cost, as explained in Section 4.4.7. For SWH systems, unlike RWH systems, the cost of additional storage is a minor contributor to the total cost of the system – unless the land costs are considered. If the land costs are considered, then the optimisation favours ponds with a surface area of between 0.1% and 0.25% of the catchment – assuming the average depth remains 1.5m. The result is that volumetric reliability decreases from 0.58 to 0.33 and the average cost per kilolitre increases from ZAR16,00 to ZAR28,00 (for Scenario 25). Including the cost of the land leads to a distortion in the design, which results in the cost of harvested water being uneconomical when compared to current potable water tariffs. The option remains to increase the average depth of the storage unit. The impact of the cost of the land is further discussed in Section 5.2.4. Finally, OF D requires a significant social survey to be of any value, which is beyond the scope of this research, but it did allow for an assessment of the impact that social perceptions and values might have on the adoption of RWH.

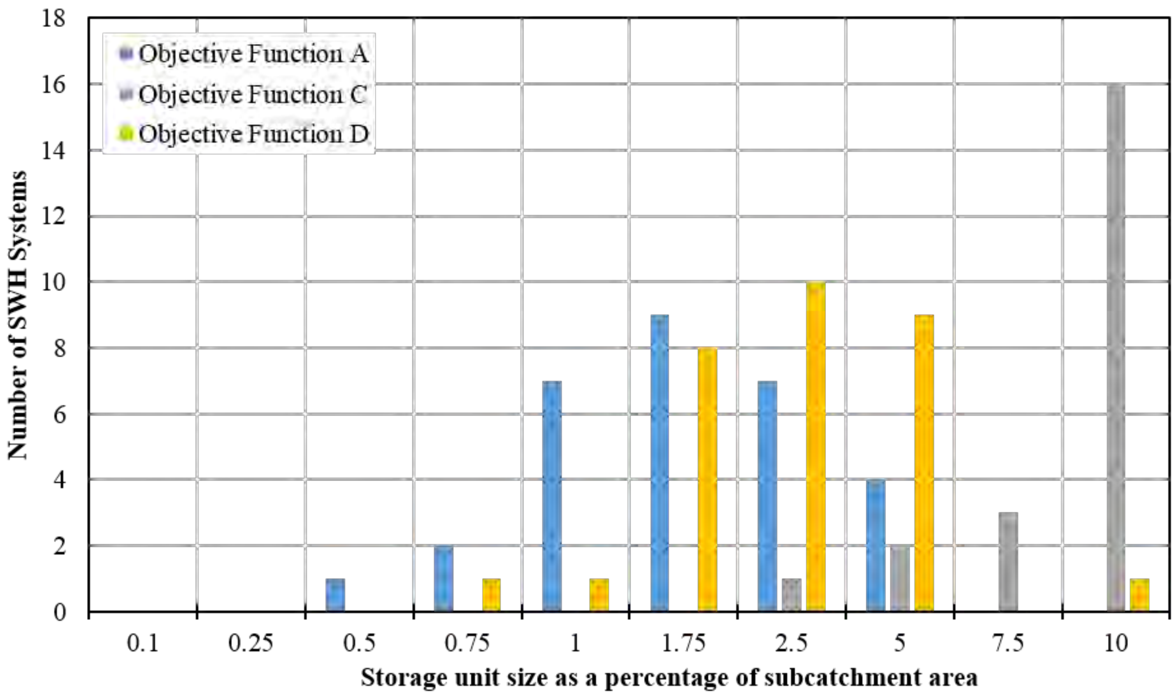

Figure 5-26: Distribution of storage unit sizes - SWH

Figure 5-26 indicates that the selected objective function is relatively important. OF A, which optimises a system to minimise the cost per kℓ of harvested water, typically results in smaller systems, compared with for example OF C.OF C indicates that larger systems (compared with OF A) would typically be required to ensure the cost of harvested water was less than that of the current municipally supplied potable water. Additionally, OF C indicates that SWH is not viable in all areas – roughly 11 out of the 30 water supply sub catchments. As a result of the above discussion, OF A was considered the most rational approach to optimising RWH systems for the analysis of the potential catchment-scale impacts of RWH, and is used throughout Section 5.2 unless otherwise mentioned.

5.2.1.2 Importance of SWH scheme size

The size of a SWH scheme was found to be important to its economic viability. Initially, each stormwater runoff subcatchment (equivalent to a watershed and assumed to be a water supply catchment for the SWH scheme) was modelled as a separate SWH scheme – within the urbanised area of the Liesbeek River Catchment there were 130 such subcatchments. The cost per kilolitre for the harvested stormwater (Scenario 25) varied significantly – as shown in Figure 5-27. Upon investigation of the wide variability, it was noted that subcatchments with higher density populations typically had cheaper SWH systems (when costs were reduced to ZAR/kℓ). This made sense when considering the cost of the dual reticulation network (Section 4.2.7.2) – a major component of the cost of a SWH scheme. It was also evident that, in general, the higher demand associated with a larger population would result in lower cost per kilolitre. Based on an assessment of the cost per kilolitre of harvested stormwater in Figure 5-27, it was decided to combine a number of stormwater subcatchments to create 'water supply subcatchments' in order to increase the demand and improve the economic viability of the SWH systems. This was achieved through combining adjacent subcatchments with careful consideration of the stormwater drainage system, in particular the surface drainage patterns; i.e. the different subcatchments should be connected by the bulk stormwater system and be able to drain to a single point. The Liesbeek River Catchment was redivided into 30 SWH schemes as shown in Figure 5-25. The costs of harvested stormwater across these 30 SWH schemes are presented in Figure 5-28. The costs per kilolitre shown are generally significantly lower than those in Figure 5-27 and are within the range of tariffs currently charged by the CoCT (Maximum CoCT water tariff including the linked sewerage charge is 2013ZAR 31.95 – Appendix K).

Table 5-4 further indicates that, in general, there is benefit to larger SWH schemes, with the cost per kilolitre for Scenario 26 (a single SWH scheme for the whole Liesbeek River Catchment) approximately ZAR12.70. However, when using Scenario 25 (SWH scheme supplying gardens, pools and toilets at subcatchment scale – for all 30 SWH schemes), the cheapest SWH system (out of the 30 subcatchments) could supply harvested stormwater at approximately ZAR10.50 while the average cost per kilolitre for all 30 SWH schemes for Scenario 25, as shown in Table 5-4, is ZAR16.60. The reason why the cost of harvested stormwater is lower for some SWH schemes is because these schemes service densely developed areas with numerous blocks of flats / university residences and, consequently they have a higher intensity water demand (AADD/ha). Simply put, the cost per kilolitre of water supplied by a SWH scheme is most closely associated with the intensity of demand, not the scale of the scheme. In the case of the Liesbeek River Catchment, the higher density subcatchments help in reducing the costs of supplying harvested stormwater to the lower density areas – hence, in this instance, population density is a proxy for water use.

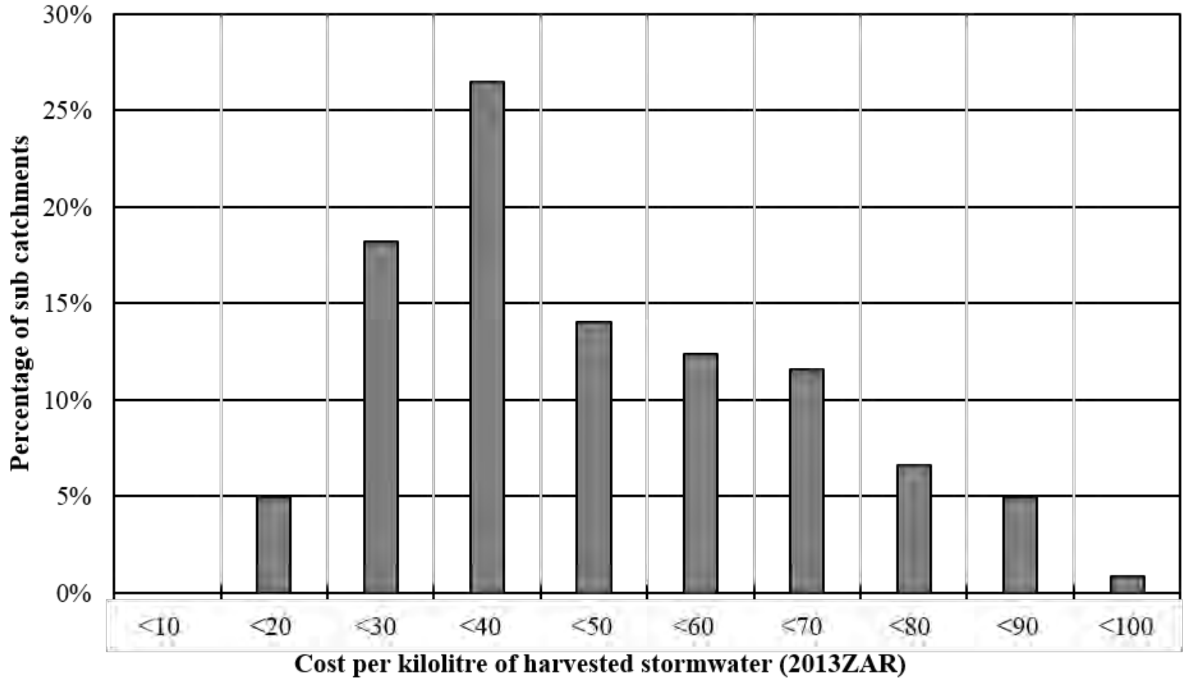

Figure 5-27: Distribution of the costs of harvested stormwater in different subcatchments (Liesbeek River Catchment divided into 130 SWH schemes)

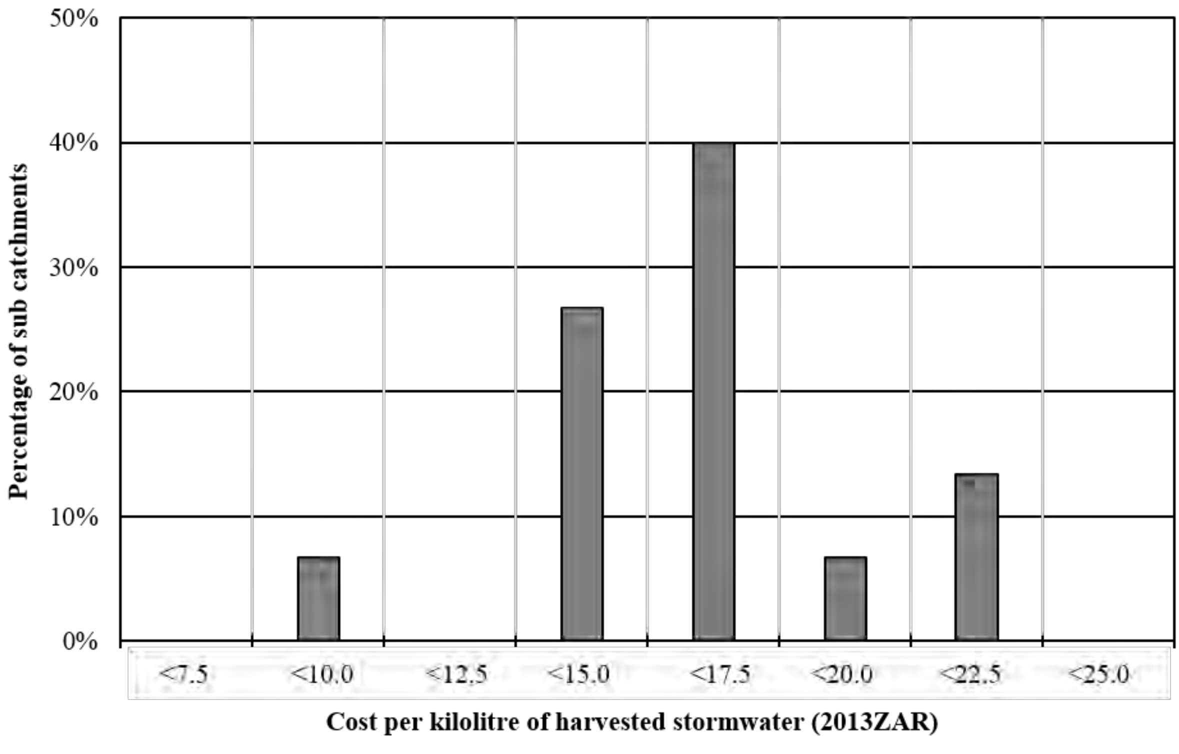

Figure 5-28: Distribution of the costs of harvested stormwater in different subcatchments (Liesbeek River Catchment divided into 30 SWH schemes)

Table 5-4: Cost per kilolitre of SWH schemes in the Liesbeek River Catchment

Scenario	Average SWH catchment area (ha)	Total number of SWH catchments	Average cost per kilolitre (2013ZAR)
Scenario 25 (with each drainage subcatchment treated as a SWH scheme)	10	130	30.00
Scenario 25 (With 30 SWH schemes – the result of combining smaller drainage subcatchments)	45	30	16.60
Scenario 26	1300	1	12.70

A further reason for larger, more centralised SWH schemes being more economical is as a result of the way in which distribution systems are designed. Smaller, decentralised systems are typically designed with higher peak flow factors (see CSIR, 2005b). As a result, at a catchment scale (when the capacity of all the decentralised systems is combined), there is significantly more capacity in certain infrastructure (e.g. pumps) than if the system had been designed in a centralised manner – e.g. one system for the whole Liesbeek River Catchment. Consequently, in order to offset the additional costs, the decentralised systems need to supply more harvested water and so typically require larger storages, which enable the system to harvest more stormwater, in order to minimise the cost per kilolitre of harvested stormwater – as illustrated in Figure 5-29. If the centralised system were to be designed with a storage equal to the total storage offered by all the decentralised systems, the effect would be to have a system where the available storage is excessive, and not optimised, and the increased available supply does not offset the increased the cost resulting from developing a larger storage unit – as illustrated in Figure 5-29. When comparing optimised centralised and decentralised SWH systems, the larger available storage offered by decentralised systems results in decentralised systems having, on average, a higher volumetric reliability.

5.2.1.3 Analysis of the postulated scenarios

Figure 5-30 and Figure 5-31 present the results of an analysis assuming 100% adoption of SWH throughout the catchment for Scenarios 21 to 26 with systems optimised using OF A (System optimised to minimise the cost per kℓ of harvested rainwater). Scenarios 21, 23 and 25 represent SWH using a decentralised approach (30 SWH systems in the catchment), while Scenarios 22, 24 and 26 represent SWH using a centralised approach (one SWH scheme for the Liesbeek River Catchment), with different end uses – see Table 4-21. Scenarios 23 and 24, which consider outdoor irrigation and pool demand, show very similar results in all performance parameters to Scenarios 21 and 22, which only consider outdoor irrigation demand. Scenarios 25 and 26, which consider all outdoor and toilet demand, show a greater reduction in runoff and demand for potable water in the catchment than the other scenarios, as a result of the substitution for harvested stormwater. The volumetric reliability of the SWH system, as was the case for RWH systems, decreases as the end uses increase. As discussed in

Section 5.2.1.2 and shown in Figure 5-29, as the end uses increase and the volume of demand met increases, the cost per kilolitre of the system decreases, making the system more economically viable.

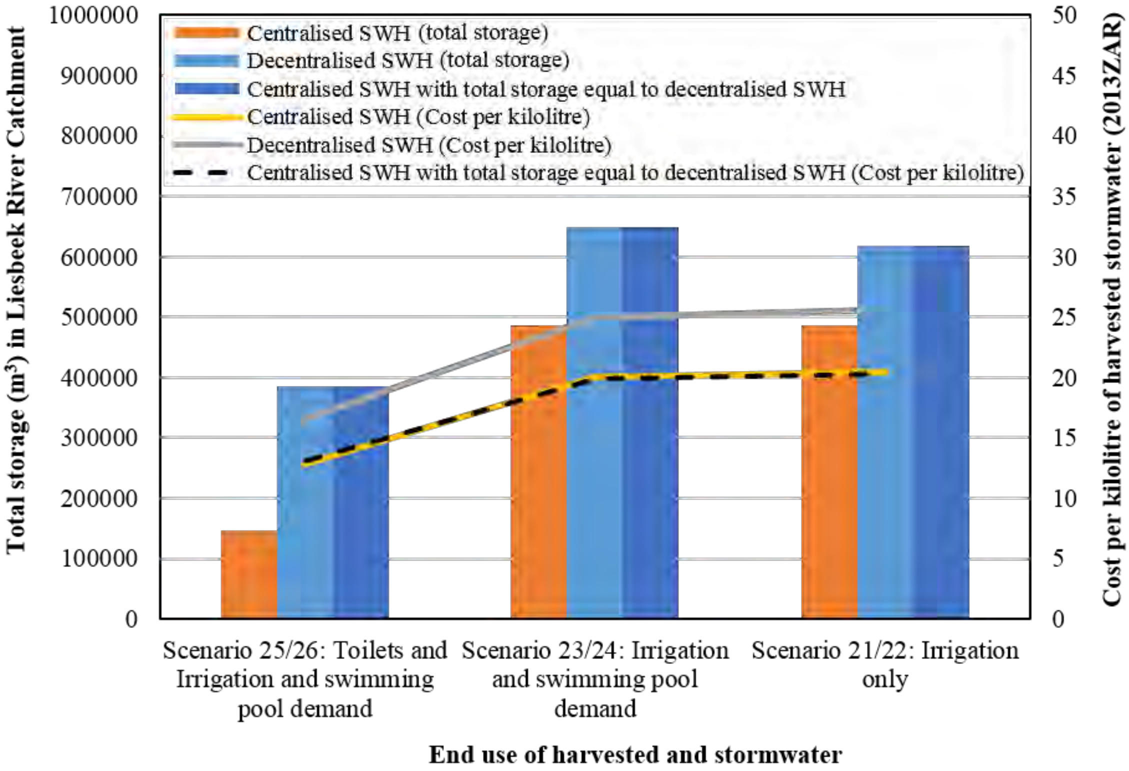

Figure 5-29: Stormwater harvesting systems optimised to minimise cost (OF A) at a decentralised (Scenarios 21, 23, 25) and centralised (Scenarios 22, 24, 26) scale, compared with centralised SWH with equivalent storage to decentralised SWH

Figure 5-30 and Figure 5-31 seem to indicate that decentralised SWH can meet a greater demand than centralised SWH. This is not true, but is a result of optimising the system to minimise the cost per kilolitre. As discussed in Section 5.2.1.2, and shown in Figure 5-29, the centralised optimised system has a much lower storage volume. Overall, increasing the storage volume of the centralised SWH system to the cumulative storage volume of the decentralised systems (e.g. Scenario 23) increases the volumetric reliability of the centralised system for the comparative scenario (e.g. Scenario 26) to greater than the volumetric reliability of the decentralised system. This would be a design decision, as it has financial and economic cost implications – as illustrated in Figure 5-29.

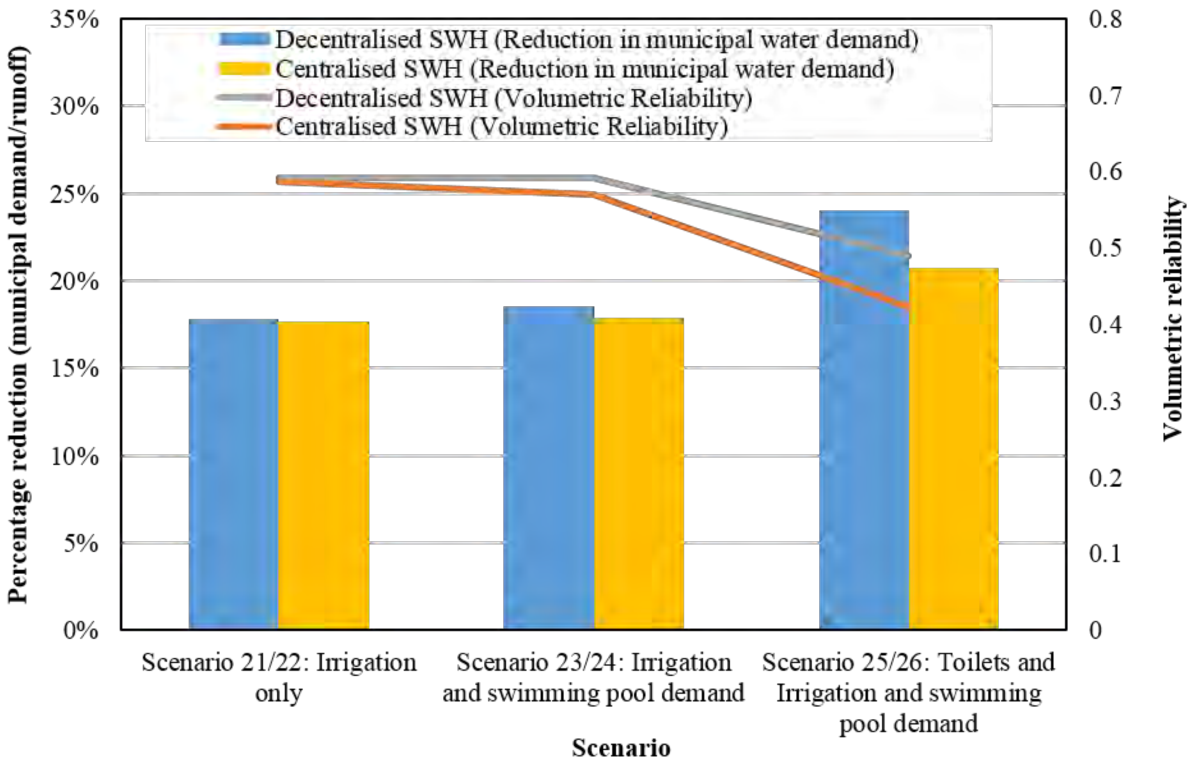

Figure 5-30: Reduction in water demand and volumetric reliability at a decentralised (Scenarios 21, 23, 25) and centralised (Scenarios 22, 24, 26) scale

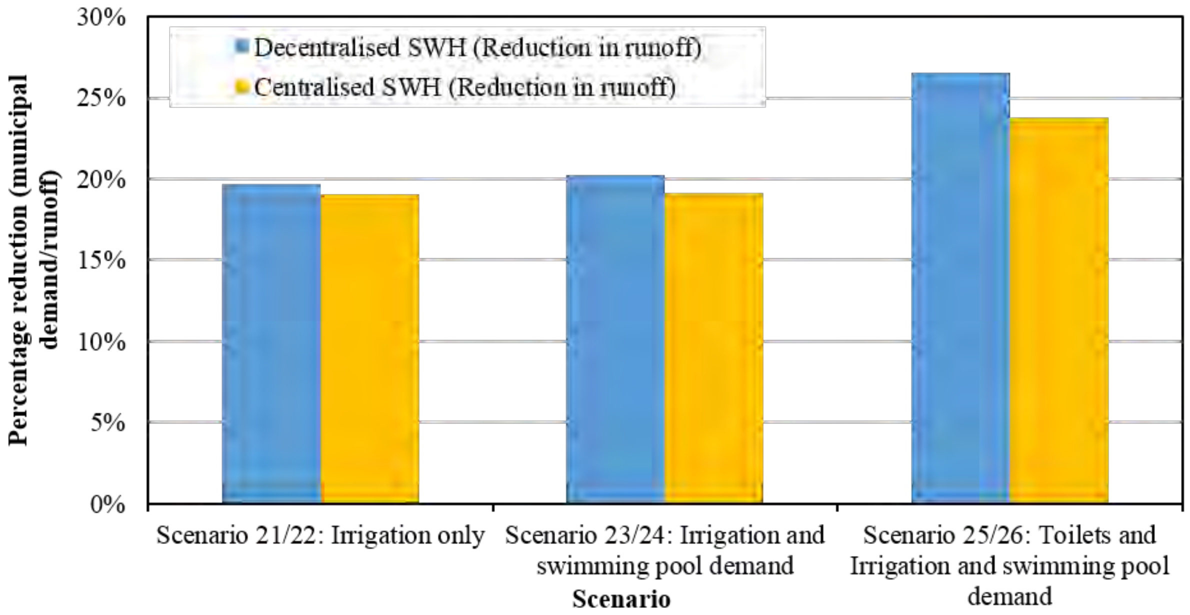

Figure 5-31: Reduction in runoff and volumetric reliability at a decentralised (Scenarios 21, 23, 25) and centralised (Scenarios 22, 24, 26) scale

5.2.1.4 The impact of varying levels of SWH adoption

The results presented thus far assume 100% adoption of SWH. If a SWH system has to be developed, a very high level of adoption would be required for the system to be economically viable. No matter what legislation and regulations are put in place, it is not reasonable to expect that everybody would make use of harvested stormwater for residential purposes. This would, in the short to medium term, be especially true for a retrofit situation where buildings would need to be retrofitted to accommodate a dual supply. Section 5.2.1.2 discussed the importance of scale and its relationship with demand. However, an important issue that needs consideration is: what would the impact be of demand being less than expected? Assuming accurate water demand data were used in developing the design estimates, this would likely be the result of lower than expected adoption of the use of harvested stormwater and/or the widespread installation of water-efficient devices.

Figure 5-32 provides the results of an analysis assuming 10%, 25%, 50%, 75% and 100% adoption of SWH, which was equated to a demand of 10%, 25%, 50%, 75% and 100% for Scenarios 25 (supplying garden irrigation, pools and toilets in a decentralised manner for the decentralised SWH schemes) and 26 (supplying garden irrigation, pools and toilets in a centralised manner – a single SWH scheme). The results were best described using a power function, and this could imply that a small change in the level of adoption will have a relatively minor impact on the cost, but a larger change in the level of adoption will have a proportionally larger impact on the cost. For example, if it is assumed that a 90% level of adoption was realised, the cost for both Scenarios 25 and 26 would increase approximately 3% in comparison to a scenario of 100% adoption. With 75% adoption, the cost would increase approximately 16%. However, as the level of adoption decreases, the relative increase in cost becomes significant. With only 50% adoption, the cost would increase approximately 50%, and with only 10% adoption, the cost would increase roughly 350% when compared to the cost at 100% adoption. This is significant for the development of SWH schemes and highlights the importance of access to credible end-use water demand data for estimating water demand for such schemes as well as in-depth social studies (beyond the scope of this study), which assess the communities' willingness to adopt alternative water supplies.

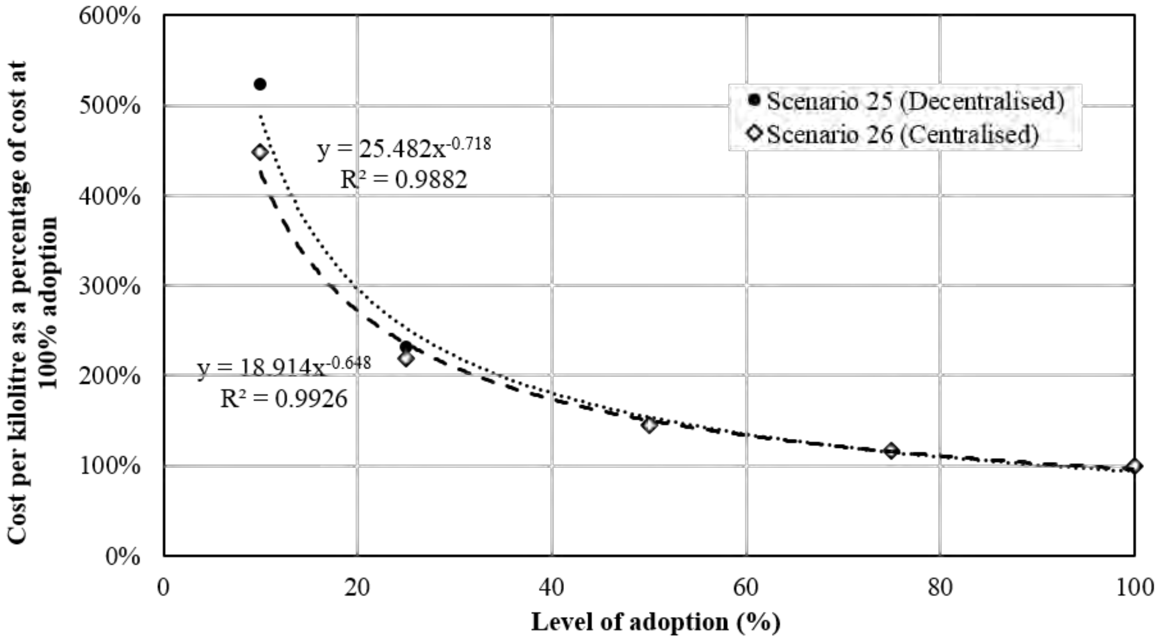

Figure 5-32: The impact of adoption levels on the cost of SWH

Figure 5-33 indicates that the change in potable water savings as a result of a SWH system is directly related but not directly proportional to the change in the level of adoption. This is logical, when considering the local climate, as the storage units of SWH schemes with lower demand will draw down slower, which results in a higher volumetric reliability. It is also important to recognise, as highlighted for RWH in Section 5.2.1.3, that at low levels of adoption, there can be a significant variation in the water demand and consequently savings, depending on which residents adopt SWH.

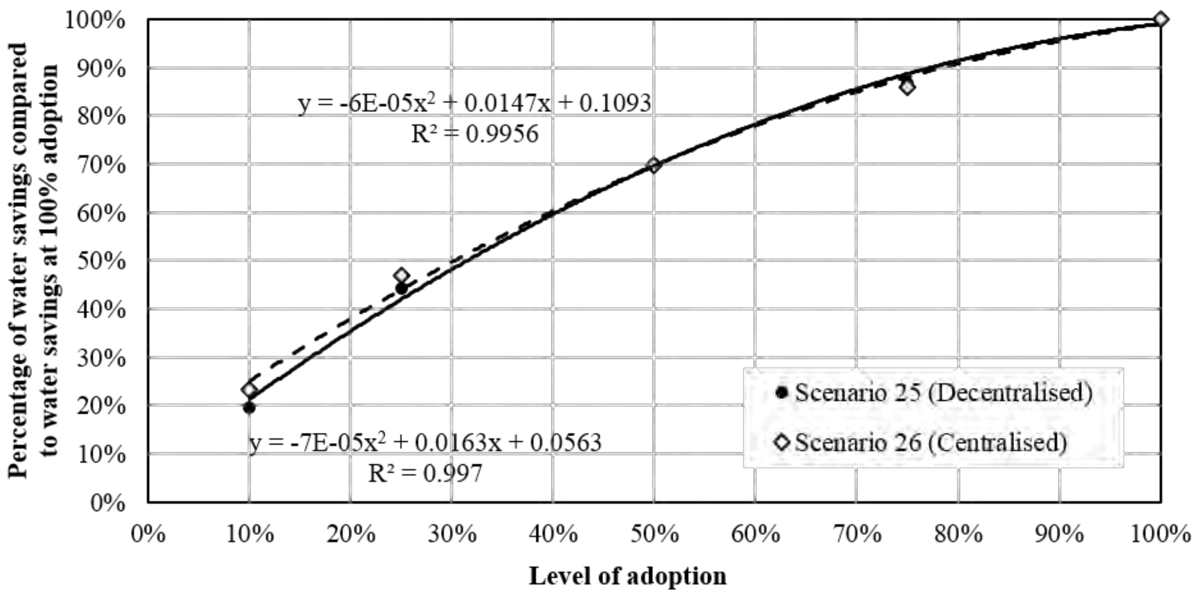

Figure 5-33: Potable water savings at different levels of adoption as a percentage of potential savings at 100% adoption

5.2.2 Climate Change

Section 4.2.3.5 indicated that climate change could affect the viability of SWH. As a result, an analysis of the impact of climate change on the viability of SWH, as discussed in Section 2.6.4.5, was undertaken. The results discussed in this section are based on Scenario 25 (supplying garden irrigation, pools and toilets in a decentralised manner for the decentralised SWH schemes), using set storage sizes optimised using historical climate (2003-2012) data using Objective Function A. This simulates the changes in performance under climate change conditions (for the period 2050–2099) of SWH systems designed to optimise volumetric reliability under the current climatic conditions.

The results are presented in Figure 5-34. In the majority of scenarios, the total runoff increases within the Liesbeek River Catchment, as is expected due to climate change scenarios predicting that annual rainfall will increase. It is, therefore, surprising that volumetric reliability and the water demand met are expected to decrease, while total water demand is expected to increase in the vast majority of scenarios. This is in part due to increased evaporation, and an inability to store the additional runoff. This results in the cost per kilolitre of harvested stormwater increasing by between -1% and 10% (average of 4%), equivalent to 2013ZAR -0.16 to 2013ZAR 1.60 per kilolitre (average of 64c per kilolitre).

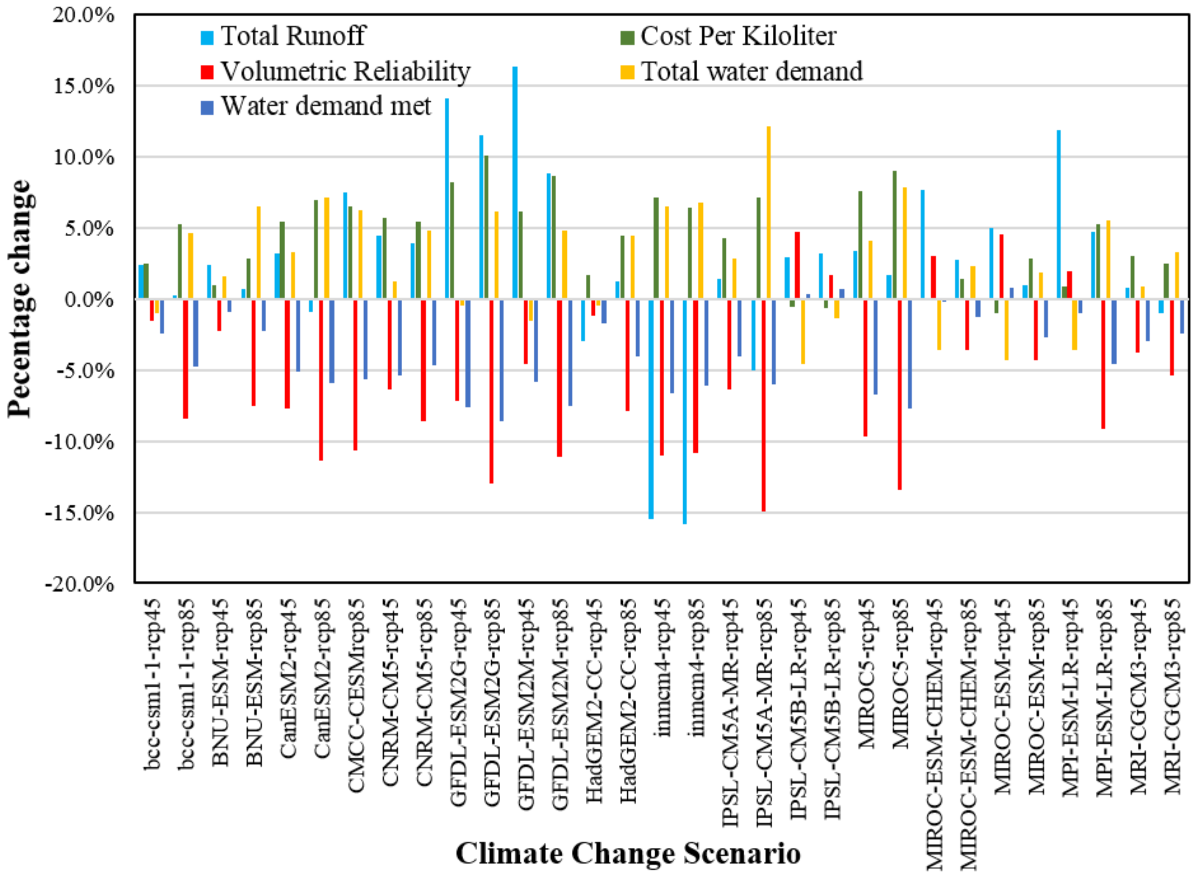

Figure 5-34: The impact of climate change on total runoff, cost per kilo litre, volumetric reliability, Total water demand and water demand met through SWH.

While the total runoff is expected to increase as a result of increases in rainfall from July to September (Figure 4-7, Figure 4-8), over these months, the SWH storages are typically full, so the additional runoff will not be stored – providing no benefit. However, evaporation is expected to increase all year round, and rainfall is likely to decrease over many of the other months of the year. The increase in evaporation also reduces the effective volume of storage through increasing the volume of evaporation losses. The result is an increased outdoor demand (pools and irrigation) and reduced runoff when it is 'needed'.

Unlike RWH, for SWH, the effect of climate change is a lot clearer – systems optimised using historical data will have a decreased ability to meet demand (volumetric reliability) while demand for water will increase. In addition, the cost of water will increase.

5.2.3 Stormwater management benefits

SWH is cited as a technique for attenuating peak flows and reducing runoff volume (e.g. (Hatt *et al.*, 2006; Fletcher *et al.*, 2008, 2013). Section 5.2.1 demonstrates that SWH has the potential to significantly reduce the total stormwater runoff by between 20% and 26%. However, as was demonstrated for RWH in Section 5.1.4, a reduction in runoff volume does not necessarily equate to the attenuation of peak flows. The maximum attenuation of peak flows within the catchment would coincide with the largest storage volume and the greatest consistent demand. Additionally, a decentralised approach would be required; otherwise, the attenuation of peak flows would not benefit the Liesbeek River Catchment, although it may benefit downstream catchments that are not being considered as part of this study. Therefore, Scenario 25 (SWH scheme supplying gardens, pools and toilets at subcatchment scale – 30 SWH schemes) was modelled in the Catchment Stormwater Model, as described in Section 4.4.8. Additionally, in order to minimise the effect that runoff from the mountain has on the scale of any reduction in peak flow, the results presented in this section (Section 5.2.3) only, unless otherwise stated, consider runoff from the urbanised component of the catchment, as described in Section 4.4.8.3.

Figure 5-35 compares the modelled flow with and without SWH in the catchment. It suggests that although SWH attenuates most storm event flows, as was found with RWH, not attenuate all of them. In some cases, SWH makes no noticeable impact on the runoff (peak flow and volume). This is due to storm events following after other storm events, when the SWH system's storage is already full and overflowing and there is no additional storage available to attenuate the subsequent storms runoff (other than temporary flood storage – discussed below). For all return periods, there remains a greater degree of uncertainty as to how much SWH will attenuate the peak flows compared with a stormwater pond (be it a retention or detention pond) as the attenuation provided is in part dependant on the demand for harvested stormwater. Where a stormwater detention pond might empty completely in 24 hours, a SWH storage pond is unlikely to substantially empty in the same period. However, as evidenced in Figure 5-36, SWH could be reasonably expected to attenuate the peak flow by between 10% and 30% for more than 50% of storm events. Unfortunately, there remains a degree of uncertainty as to the degree of attenuation that might be expected. It is difficult to predict in advance how much attenuation will be obtained unless it is assumed that the SWH storage is full and that the only storage offered is the temporary storage of water above the crest of the weir – termed 'live storage' or 'dynamic storage' – which could be estimated. Of the storm events with a return period of greater than 1 year, that took place between 2003-2012, it is clear that the SWH in Scenario 25 (SWH scheme supplying gardens, pools and toilets at subcatchment scale – 30 SWH schemes) has the potential to reduce the peak flow significantly for larger return periods. For example, the peak flow was reduced from a maximum of 40 m^3/s to a maximum of 32 m^3/s for the largest storm event (20-year recurrence interval). This is a significant reduction in peak flow.

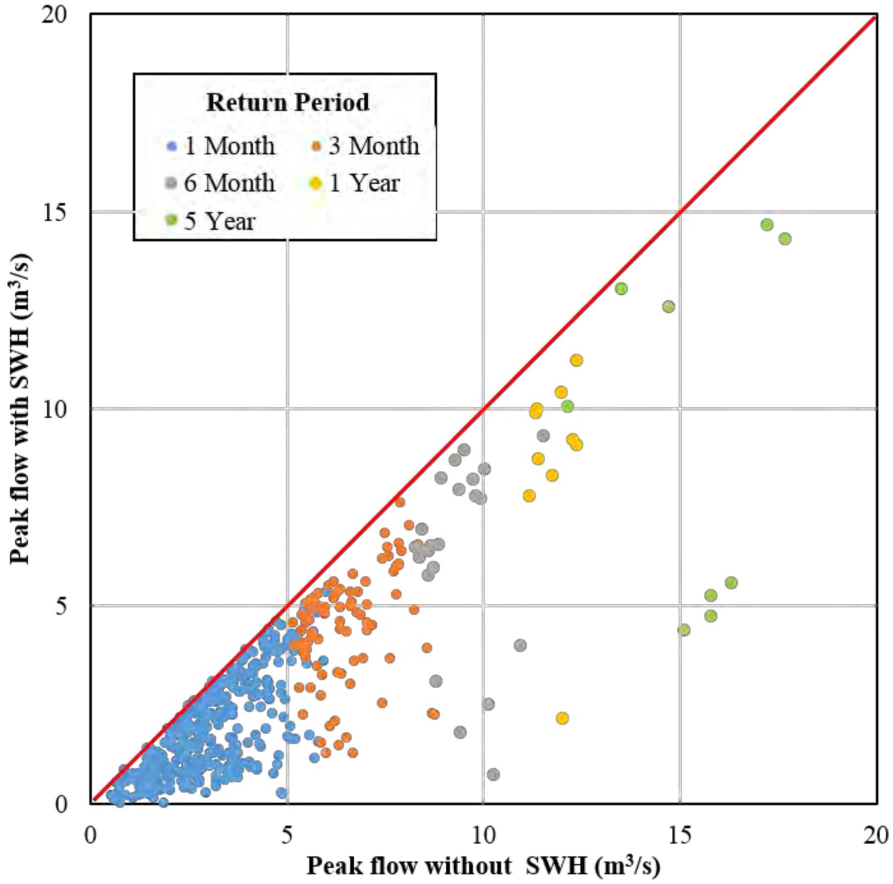

Figure 5-35: Comparison of modelled flows in the Liesbeek River with and without SWH, for events between 2003-2012

The reduction in peak flow is best understood by inspecting the Liesbeek River catchment's modelled hydrograph – an extract is presented in Figure 5-37 – which shows the effect SWH has on the river's flow. It is evident from this nine-day extract with five storm events, how SWH might – in the case of the Liesbeek River Catchment – attenuate peak flows. The extract illustrates the modelled results for the catchment without SWH and with SWH for two scenarios – Scenario 23 (outdoor demand) and Scenario 25 (outdoor and toilet demand). It is taken during the middle of winter and so there is negligible outdoor demand and thus the SWH

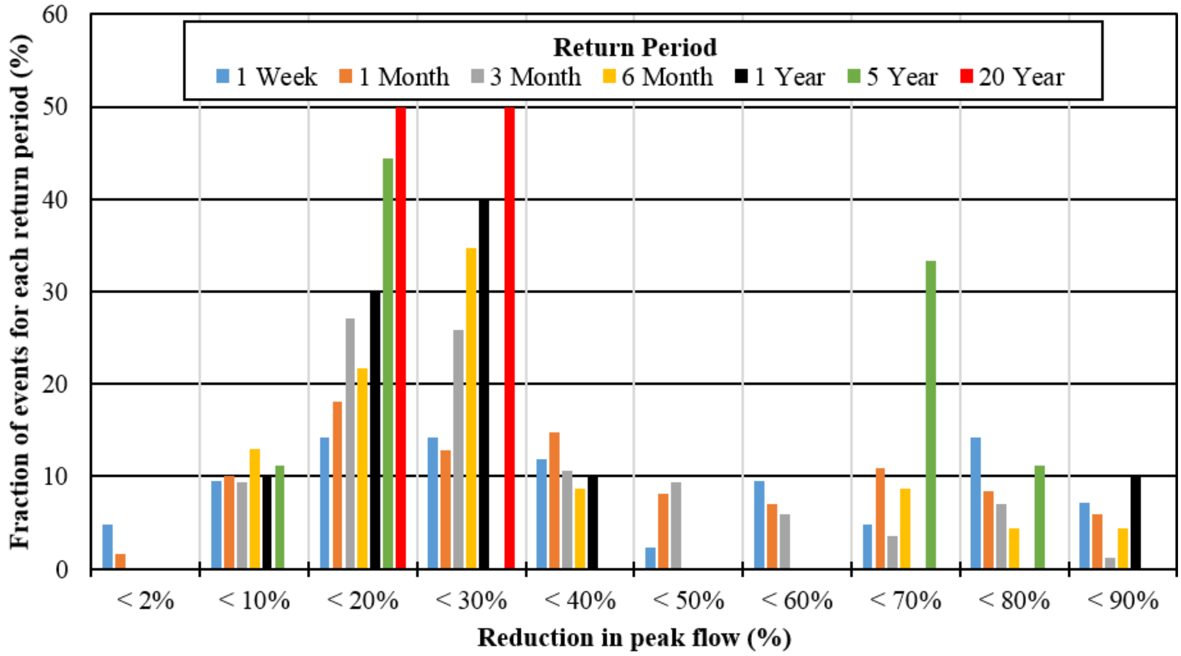

Figure 5-36: Distribution of the reduction of peak flow as a result of SWH in the Liesbeek River Catchment for different return periods, for all events between 2003-2012

storage units are full (to the top of the weir) for Scenario 23. However, due to additional toilet demand in Scenario 25, the SWH storages are partially emptied – as toilet use remains relatively constant year round – and thus in this Scenario 25 there is additional storage space available to attenuate the runoff. For the storm event labelled '1', there is a degree of attenuation for both scenarios; however, for Scenario 25 (includes toilet water demand), as a result of the additional available storage there is greater attenuation. Where the storages are effectively already full (Scenario 23), attenuation is still offered through temporary storage of water above the crest of the weir – termed 'live storage' or 'dynamic storage'. At points '2' and '3', it is evident that both scenarios are offering roughly the same level of attenuation of peak flow. The flow in the receding limbs – at points '2' and '3' – is greater than with no SWH; this is most evident in '3'. This is due to the temporary storage offered by the SWH ponds – above the weir crest – which has acted to attenuate the peak flow. The degree of attenuation and temporary storage will be partially dependent on the SWH pond's overflow outlet. In the case of this model, each pond had a 10m-wide weir with a spillway directly to the river. It was ensured that the depth in the pond never exceeded 0.5m above the weir's invert – in other words, the SWH ponds provide maximum additional temporary storage of 0.5m above the crest of the weir. The majority of the ponds used 0.25 – 0.5m above the crest of the weir to provide the additional storage to attenuate peak flows.

In order to further demonstrate the impact that such a reduction in peak flow might have on flooding (and flooding risks), the 2D model discussed in Section 4.3.6 was used to simulate the impact that SWH might have had on local flooding for the 12th of July 2009 storm event –

the only major flooding event for which rainfall records and calibration data were available (Section 4.3.6). The flood risk levels –using the CoCT's definition of flood risk (CSRM, 2009a) – were determined for the situations with and without SWH (see Figure 5-38). It is evident that without active management, SWH has the potential to eliminate any significant flood risks in storm events such as that which occurred on the 12th July 2009.

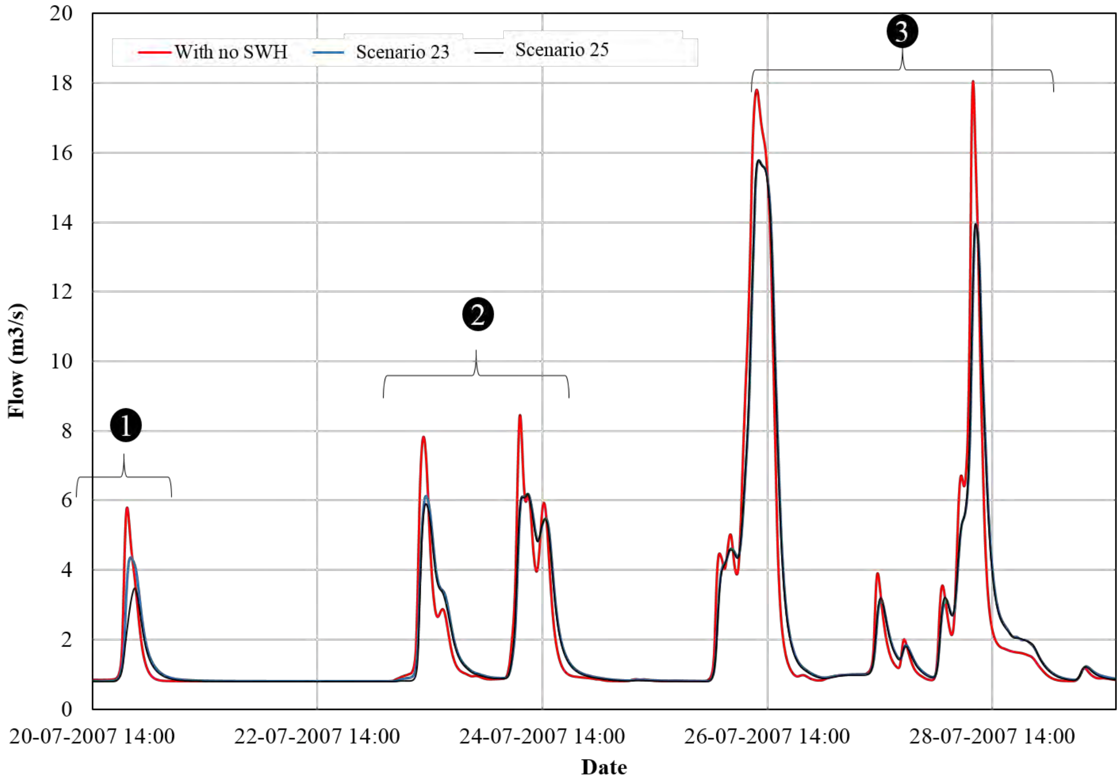

Figure 5-37: Hydrograph with SWH providing outdoor demand, SWH providing outdoor and toilet demand, and without SWH

A further opportunity exists for stormwater managers. Should they 'actively manage' their SWH systems using Real-Time Control (RTC) systems in such a manner that, prior to a predicted storm event, the storage is partially emptied, significant attenuation could be achieved without compromising the ability to meet water demand. This would require the development of a reasonably accurate, calibrated runoff model that could make use of predicted rainfall to predict the runoff for a particular storm. Based on the anticipated runoff, the stormwater manager could partially empty the SWH system's storage a day or more before (depending on the availability of rainfall predictions). This will result in an increase in the pre-event flow rate in the river, but a decrease in the peak flows, which could prevent flooding.

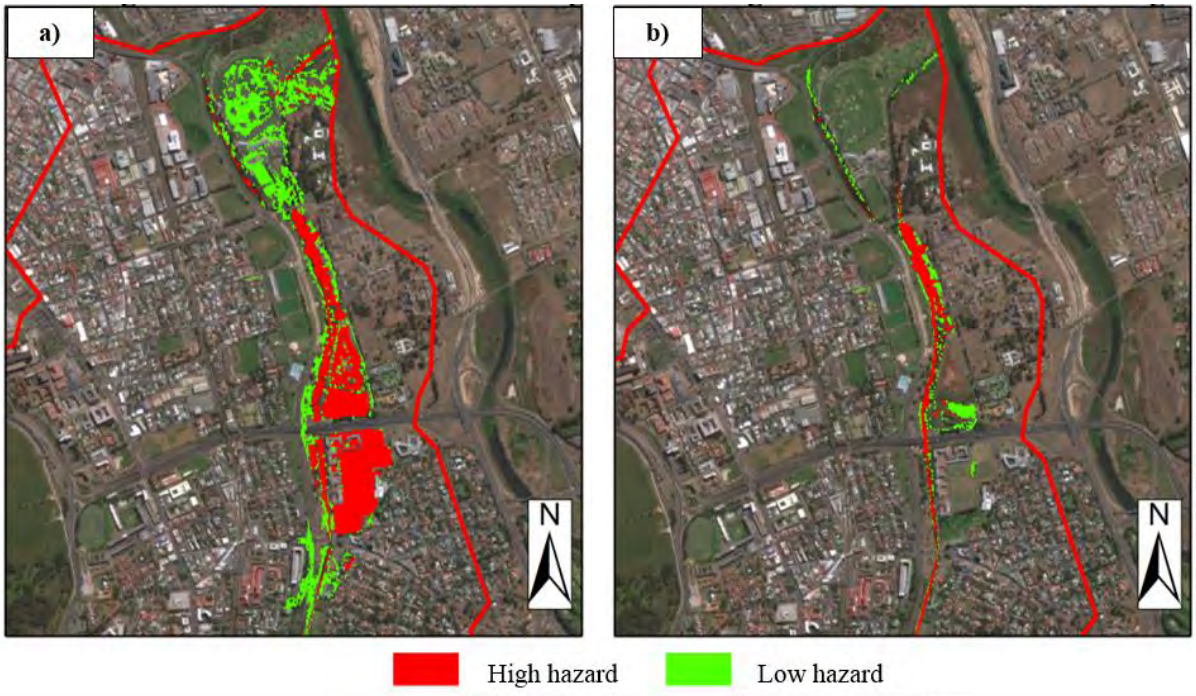

Figure 5-38: The impact of stormwater harvesting on flooding in the Liesbeek River Catchment (12th July 2009) – a) without SWH, b) with SWH

5.2.4 Consideration of ecosystem goods and services

In line with one of the objectives of this research, the value of the additional benefits have been considered. De Wit *et al.* (2009) undertook an investigation of the value of natural assets in the City of Cape Town. Through their own investigation and review of literature, they monetised the value of different 'natural assets' and ecosystem goods and services from wetlands and parks. These values were adjusted to 2013ZAR in line with all other values in this thesis and are presented in Table 5-5. While parks, wetlands and open spaces, such as those that might be created for an SWH system, have in this section been considered to provide a positive amenity value, De Wit *et al.* (2009) note that some can provide a negative amenity value. The analysis of values reported using the Hedonic method assessed property prices in relation to their proximity to a park or wetland. The other valuation methods are highlighted in Section 2.6.6.

Table 5-5: Value in 2013ZAR/m^2 of different 'natural assets' per year

'Asset'	Minimum	Average	Maximum	Method of estimation
Parks	0.46	0.74	1.03	Contingent valuation
Wetlands	0.37	0.63	0.89	Contingent valuation
Parks and wetlands	8.14	10.91	15.13	Hedonic pricing – increased property value
Wetlands	3.8	4.02	4.24	Replacement cost – water treatment and flow attenuation

From Table 5-5, it is clear that the maximum benefits (recreational use, added property value, water treatment, storm flow attenuation) could be considered at around 2013ZAR 20.40/yr.m^2. Thus considering the size of the systems (cumulatively at catchment scale), there could be significant value, estimated at 2013ZAR 2–7.2 million/yr. within the Liesbeek River Catchment.

Internationally, open space typically accounts for between 10% to 17% of a development (CSIR, 2005a). In the urbanised portion of Liesbeek River Catchment, 14% of land is currently undeveloped – well within international norms. SWH in the Liesbeek would, if designed to minimise cost (excluding land costs), require between 0.7% and 3.33% of the catchment (Table 5-6). If, land is set aside along or near the river, and facilities are designed in a multipurpose manner before a catchment is developed, the inclusion of SWH ponds should not be an insurmountable problem. There is a problem with the current configuration of the Liesbeek River Catchment, however, in that 4.27% of this land is at the mouth of the Liesbeek River, which is adequate for a centralised system (2013ZAR 2 million), but would not provide the same level of environmental benefits as a decentralised system (2013ZAR 7.2 million). Additionally, the majority of the remaining open space is either not situated in areas where it could be used for SWH – i.e. the edge of the catchment – or is used for other purposes such as school sports fields.

If the benefits of SWH were to be included in an analysis, it would also be fair to consider the value of the land on which such facilities are built. An analysis of the average value of undeveloped land was undertaken using the 2012 General Valuations role – Section 4.2.1. This resulted in an estimated value of 2012ZAR 3600/m^2, which was adjusted to 2013ZAR 3880/m^2 according to property inflation in the City of Cape Town. If this is annualised (using a discount rate of 3.1%) over 100 years, it equates to a value of 2013ZAR 126/m^2. This would equate to an annual cost of between 2013ZAR12-42 million/yr. It is evident that this cost significantly exceeds the benefits of SWH.

Table 5-6: Surface area of SWH storage as a percentage of total catchment area

Scenario	% of Liesbeek River Catchment used for SWH
Scenario 21	3.17
Scenario 22	2.5
Scenario 23	3.33
Scenario 24	2.50
Scenario 25	1.98
Scenario 26	0.75

Also based on De Wit *et al.* (2009), Table 5-7 provides the total cost of flood damage over 100 years, which has been annualised. Therefore, were SWH able to reduce all flooding – highly unlikely – it would equate to the reported annual benefits. While the values (costs and benefits) are cumulatively significant, in order to consider the viability of SWH, including benefits, they need to be reduced to a per-kilolitre value, as presented in Table 5-7. It is evident that the cost per-kilolitre of harvested stormwater will roughly double [Table 5-7, (5) vs. (6)] if the cost of land is included. However, the outcomes of the analysis would be significantly different if the urban area had been planned with SWH in mind. If public open space was utilised so it could perform the functions laid out in Table 5-5 and therefore not require additional urban space, these facilities could offer significant value to the community. The per-kilolitre cost would significantly decrease, and would be approximately equivalent to what the CoCT charges residents who use 6–10.5 kℓ/month – Appendix K. This would make SWH viable for the vast majority of households in the catchment. Furthermore, Section 5.2.3 indicated that SWH has the potential to significantly reduce flooding. If this were realised, for example in Scenario 25, the net cost [Table 5-7, (8)] would be further reduced.

Table 5-7: Value of additional costs and benefits per kilolitre

No.	Description	Scenario 26 - centralised (2013ZAR/kℓ)	Scenario 23 - decentralised (2013ZAR/kℓ)
1	Benefits (Table 5-5)	2.27	5.16
2	Land costs	14.01	31.85
3	Net benefits (1-2)	-11.74	-26.69
4	Reduced flood costs	0.00	3.74
5	Cost of SWH, excluding benefits and land costs	12.85	16.38
6	Cost, including benefits and land costs (5-1+2)	24.59	43.07
7	Cost, including only benefits (Table 5-5) (5-1)	10.58	11.22
8	Cost, including benefits and land costs (5-1+2+4)	10.58	7.48

5.2.5 Effect of economic changes

In order to assess the implications of economic variability on the viability of SWH, a sensitivity analysis was conducted using discount rates of 3.1% to 4.5% (see Section 4.4.5.2) for Scenarios 21 to 26. Using the *URSHM*, Scenarios 21 through 26 were modelled using set storage sizes, as calculated using OF A (Table 4-17) and a discount rate of 3.1%. The results presented in Figure 5-39 show the change in average cost per kilolitre throughout the catchment. The analyses show that an increase in discount rate will increase the cost per kilolitre. The difference equates to an approximately 16% increase in the cost per kilolitre (between a discount rate of 3.1% and 4.5%), for both SWH using a decentralised approach (Scenarios 21, 23, and 25) and using a centralised approach (Scenarios 22, 24 and 26).

Considering the uncertainty as to future prices of water, future prices of electricity, future availability of water etc., the use of a discount rate of 3.1% provides a reasonable indication (Section 4.4.5.2) of the potential of SWH in the Liesbeek River Catchment. The increase in cost per kilolitre (between a discount rate of 3.1% and 4.5%) is approximately triple (as a percentage) the increase in cost per kilolitre expected for RWH systems (Section 5.1.3), but the overall cost is roughly equivalent in Rand terms (± ZAR 2).

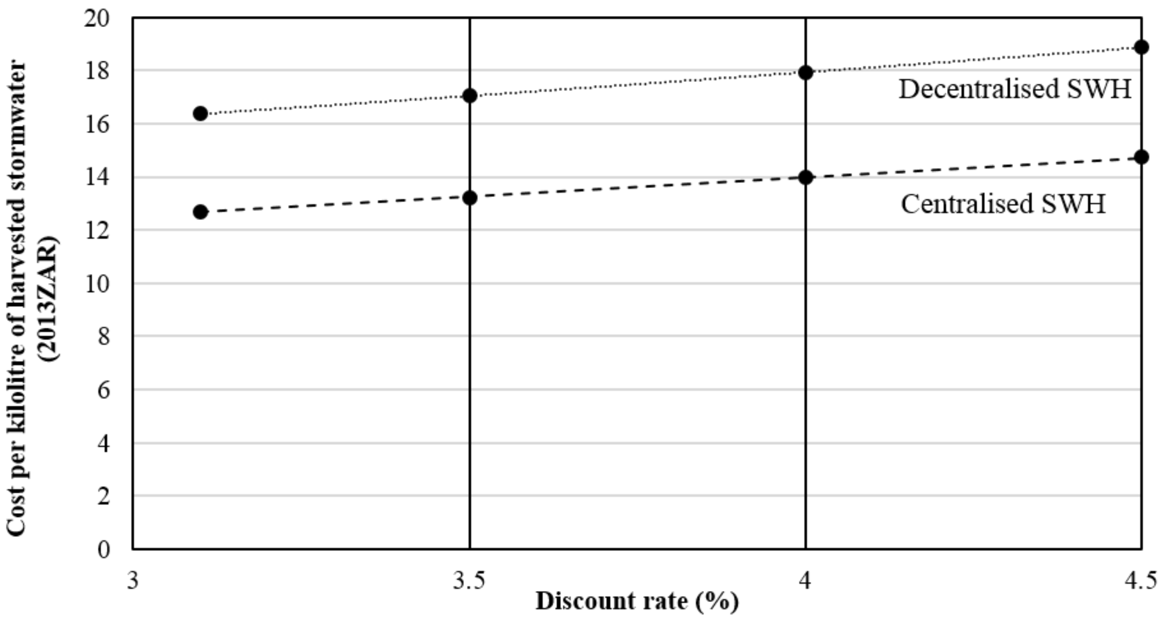

Figure 5-39: Sensitivity to changes in the discount rate – SWH

5.2.6 Impact of modelling methods

Sections 2.5 and 2.6 highlighted a number of important modelling considerations that may influence the results of an analysis of the viability of SWH. The most important were the selection of an appropriate time step; and the effect of spatially lumping the modelling of SWH systems to evaluate catchment-scale impacts. As is evident in Table 2-15, there is also limited guidance regarding the use of either the YBS or YAS rule when modelling SWH systems. These three aspects were investigated and are briefly discussed in this section.

5.2.6.1 Selecting an appropriate time step and storage algorithm

The selection of an appropriate time step has a number of important implications for this research, including the optimisation of an SWH system, the accurate modelling of a single SWH system in isolation and the accurate modelling of SWH in the catchment as a whole.

It is generally accepted (see Table 2-15) that the YAS algorithm is most appropriate for modelling RWH systems and provides conservative results. There is, however, limited guidance when modelling SWH systems. Roebuck (2007) noted that for a larger system (to service communal buildings) the use of the YAS approach was not fully vindicated. Mitchell *et al.* (2008b) noted that the storage capacity and time step were important considerations when selecting whether to use YAS or YBS. They suggested that for a time step <6 hrs and a storage volume of 16 kℓ/ha, the choice of YAS/YBS would have little impact. In this study on the Liesbeek River Catchment, the storage size ranged from 100-1000 kℓ/ha. Table 5-8 and Table 5-9 show the results, at a catchment scale, of modelling 30 different SWH systems in the Liesbeek River Catchment using the YAS and YBS algorithms. It is evident that the differences are negligible, with only the difference in percentage of dry periods using the daily time step, exceeding 1%.

Table 5-8: Comparison of modelled performance using the YAS and YBS operating rules at hourly time steps

Performance parameter	YAS	YBS	Difference (%)
Demand met (Mℓ/yr.)	1023	1024	-0.08
Total storage volume (m^3)	386100	386100	0.00
Volumetric reliability	0.52	0.52	-0.07
% dry periods	0.29	0.29	0.31
% collected	0.26	0.26	-0.02
Cost per kilolitre (2013ZAR/kℓ)	16	16	0.07

Table 5-9: Comparison of modelled performance using the YAS and YBS operating rules at daily time steps

Performance parameter	YAS	YBS	Difference (%)
Demand met (Mℓ/yr.)	1018	1028	-0.93
Total storage volume (m^3)	386100	386100	0.00
Volumetric reliability	0.52	0.53	-0.89
% dry periods	0.31	0.30	4.34
% collected	0.26	0.27	-0.60
Cost per kilolitre (2013ZAR/kℓ)	17	16	0.91

The difference between using the YAS or YBS methods was also compared using daily and hourly time steps. Table 5-10 shows the variation in the modelled performance at the Liesbeek River Catchment scale. Figure 5-40 and Figure 5-41 show the variation in results at a system scale. As expected, the YBS estimates a higher yield and consequently higher volumetric reliability, while the YAS provides a more conservative estimate (Table 5-10). The YBS in conjunction with the daily time step model provides a provides a better estimate (smaller difference when compared with the YAS with either YAS or YBS in conjunction with a hourly time step model which is assumed to be more accurate than the daily time step model) of the percentage of dry periods to be experienced. It is also evident that the range in error at the system scale is relatively small – except for the percentage of dry periods that has been explained above (Figure 5-40 and Figure 5-41). Either method would present reasonable results within the range of uncertainties expected with rainfall, runoff and demand modelling.

Table 5-10: Comparison of the modelled SWH performance using the YAS and YBS operating rules at the catchment scale using daily and hourly time steps

Performance parameter	YAS (Daily time step) vs. YBS (Hourly time step) (%)	YBS (Daily time step) vs. YBS (Hourly time step) (%)	YAS (Daily time step) vs. YAS (Hourly time step) (%)	YBS (Daily time step) vs. YAS (Hourly time step) (%)
Volumetric reliability	-0.47	0.42	-0.39	0.50
% dry periods	8.48	3.96	8.13	3.64
% collected	-0.22	0.38	-0.20	0.40
Cost per kilolitre (2013ZAR/kℓ)	0.49	-0.42	0.42	-0.49

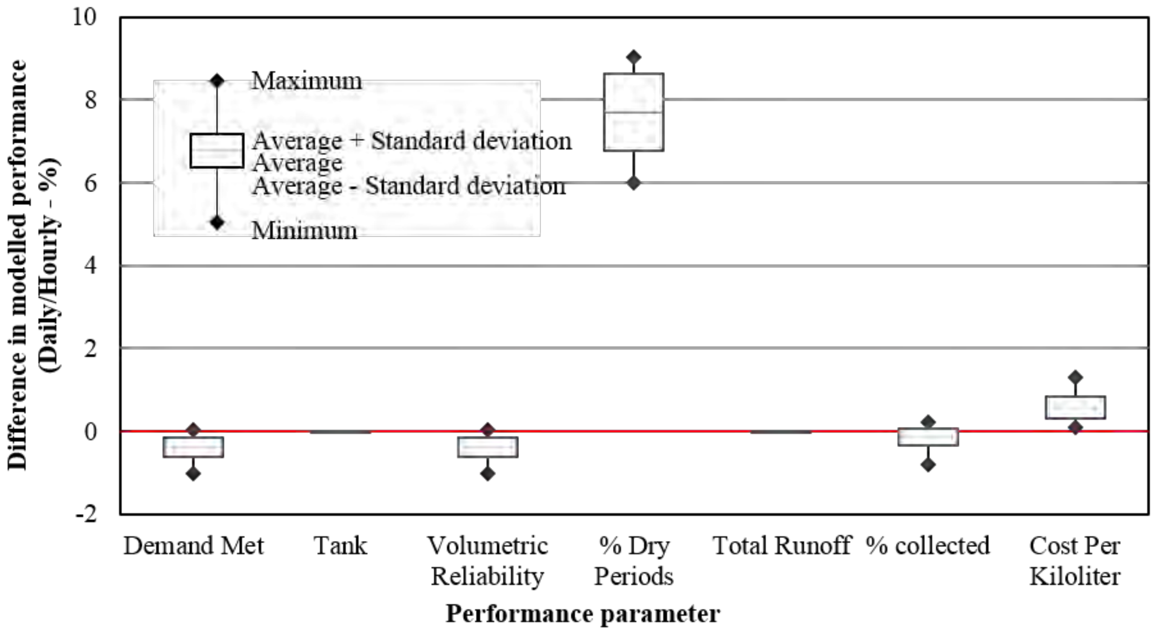

Figure 5-40: Comparison of the modelled performance using daily and hourly time step models using the YAS storage algorithm at the system scale (storage sizes optimised in the daily time step model)

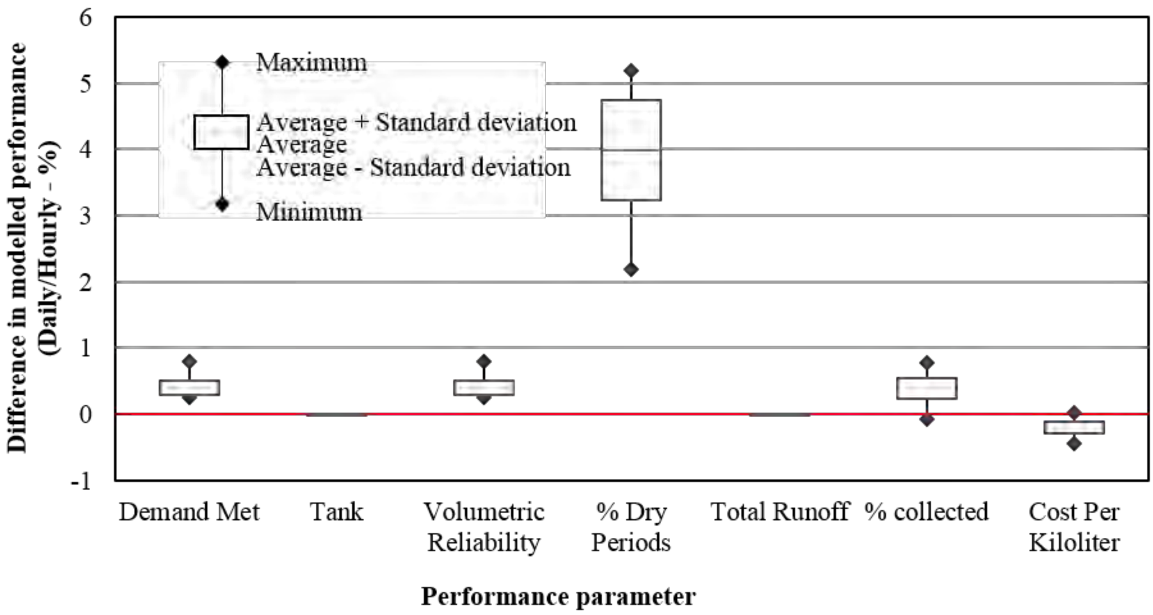

Figure 5-41: Comparison of the modelled performance using daily and hourly time step models using the YBS storage algorithm at the system scale (storage sizes optimised in the daily time step model)

5.2.6.2 Spatial lumping of SWH systems

Neumann & Maheepala (2013) suggested that '*the input variables of a number of stormwater harvesting systems spread across a catchment can be linearly combined (or summed) into a single system without introducing significant errors provided that the individual harvesting systems are well designed*'. They further suggested that the errors they found (2.4%-5% in demand met) were within the range of uncertainties associated with rainfall runoff and demand modelling.

In this study, the Liesbeek River Catchment was divided into 30 SWH subcatchments, and the results of simulating SWH in these catchments independently and in a lumped manner (summing all demand, storage size, runoff etc.) were compared using Scenarios 21, 23 and 25. Note that, in Table 5-11, the errors found in this study for demand met, the percentage for dry periods, and the percentage collected are larger than those reported in Neumann & Maheepala (2013). There are a number of possible reasons for this, including the significant variation in demand as well as climate variation across the catchment. However, the specific reason is of little consequence; rather, that an error of up to 9.7% is possible is an indication that caution is required, especially when it comes to economic analyses that rely on the volume of demand met to determine the viability of a system from an economic perspective.

Table 5-11: Difference in performance parameters when modelling SWH systems separately and spatially lumping all the SWH systems together

Scenario	Difference in Volumetric reliability (%)	Difference in Dry periods (%)	Difference in percentage of runoff collected (%)
Scenario 21	6.7	-4.6	5.3
Scenario 23	8.9	3.8	8.2
Scenario 25	9.7	2.9	8.3

5.2.6.3 Summary and discussion of the impact of modelling methods

It is evident that the spatial scale of analysis (whether systems are modelled independently or lumped) can impact its results. These results indicate that, for this study where the storage volume significantly exceeds the 16 kℓ/ha suggested by Mitchell *et al.* (2008b), it is of little significance whether the YAS or YBS algorithm is used for modelling the SWH systems' storage. It is apparent that, in this research, the use of either a daily or hourly time step is acceptable for sizing the SWH systems' storage volume and that the only performance parameter that will show any variation is the percentage of dry periods.

5.2.7 Viability of SWH: Summary and discussion

SWH has the potential to reduce the total current potable water demand of the Liesbeek River Catchment by up to 20%. This would be a significant reduction for the CoCT. In order for such reductions in water demand to be realised would, however, require that all residents and businesses make use of harvested stormwater for at least flushing toilets and outdoor irrigation. This would likely require changes in the regulations related to the supply of water in the CoCT. Therefore, while technically and economically SWH might be an option for reducing potable water demand, the social, institutional and political implications would still need to be investigated.

SWH, unlike RWH, has the potential to offer additional benefits (including water quality treatment, amenity value etc.) to the surrounding residents and community, which can partially offset the costs of operating the SWH system and make it equivalent in cost and potentially cheaper than the potable water currently supplied by the CoCT. This is especially true in the higher density catchments, where there is a relatively high demand (e.g. where there are blocks of flats / small properties). Additionally, through active management of the SWH storage units, there is the potential to significantly attenuate peak flows during both small and extreme events. While, in principle, a similar benefit to SWH might be realised through the active management of RWH systems, SWH has the advantage of being a quasi-centralised approach. At the scale for it to be economically viable, it would require the management of roughly 20 storage units (ponds) instead of over 6,000 RWH storage units (tanks). Therefore, practically, SWH is a better option.

It was also evident that lower-than-expected demand, as a result of lower-than-expected adoption, poor-quality data used in design, or the installation of water-efficient devices, could negatively affect the economic viability of SWH. This highlights the need for access to credible end-use water demand data for estimating water demand for such schemes, as well as in-depth social studies (beyond the scope of this study) that assess the communities' willingness to adopt alternative water supplies.

Climate change is typically a concern for water resource planners. Section 4.2.3.5 highlighted that evaporation is expected to increase, while precipitation is expected to decrease. The analysis using the adjusted runoff – based on the expected changes in evaporation and precipitation from the 31 different climate change scenarios – indicated that it is very likely that SWH systems will have a decreased volumetric reliability and the cost of harvested stormwater is likely to increase.

SWH offers a means of reducing municipal water demand, decreasing total runoff volumes, offering amenity benefits and, if actively managed, a means of attenuating peak flows. In certain areas, it offers a means of financially and economically providing water that is less expensive than the currently supplied potable water. Currently, therefore, SWH is a viable option that should be investigated under the following conditions:

- Harvested stormwater is used for as many end uses as possible – primarily toilet flushing and irrigation.
- SWH is more viable at higher population densities, which equate to a higher and more constant water demand (toilet flushing throughout the year).
- Additional benefits may be realised through actively managing the volume in storage, in order to attenuate peak flows, through detaining stormwater runoff.

5.3 Viability of simultaneously harvesting rainwater and stormwater

Up until this point, the viability of RWH and SWH harvesting have been considered in isolation, due to the '*broadly similar benefits in reducing pollution loads, downstream stormwater flows and demand for mains water*' (DECNSW, 2006). However, it is evident that there are, as noted in Australia, '*distinct differences in costs, stakeholders, and maintenance and health risks between these approaches*' (DECNSW, 2006).

In Section 4.5.2, it was noted that a review of the literature indicated that harvested stormwater is most appropriate for outdoor uses and could be considered for toilet flushing. In Section 4.5.1, RWH was considered appropriate for all non-potable end uses. So far, the analysis would indicate that SWH is a better alternative than RWH, but for the sake of completeness, it is important to consider the viability of a system where RWH and SWH are integrated. This section does so by considering the four Scenarios (27-30) in Section 4.5.3.

For ease of reference Table 4-22 which details the different scenarios used to assess the viability of simultaneously harvesting rainwater and stormwater has been repeated below.

Table 4-22: Scenarios 27 through 30 (rainwater and stormwater harvesting)

Scenario	Rainwater Harvesting		Stormwater Harvesting	
	End-use water demand description	Scenario from Table 4-20	End-use water demand description	Scenario from Table 4-21
Scenario 27	Supplying toilet flushing, washing machine, shower / bath only	Scenario 8	Gardens and pools (at subcatchment scale)	Scenario 23
Scenario 28	Supplying toilet flushing, washing machine, shower / bath only	Scenario 8	Gardens and pools (catchment scale)	Scenario 24
Scenario 29	Supplying washing machine, shower / bath only	Scenario 7	Gardens, pools and toilets (at subcatchment scale)	Scenario 25
Scenario 30	Supplying washing machine, shower / bath only	Scenario 7	Gardens, pools and toilets (catchment scale)	Scenario 26

5.3.1 Scenario analysis

This section primarily focuses on the potential of RWH and SWH to reduce the demand for potable water and the cost of doing so. A detailed discussion as to the impacts of different end-use scenarios for RWH and SWH has been provided in Sections 5.1 and 5.2.

Table 5-12 presents the results of the analysis regarding the potential reductions in water demand for Scenarios 27 to 30 – see Table 4-22. Table 5-12 indicates that both RWH and SWH have the potential to significantly reduce the catchment's water demand and that, together, it is possible to reduce demand significantly more than using one or the other in isolation. As explained in Section 5.2, it is possible to ensure that the demand met by Scenario 28 (SWH at a catchment scale) equals that of 27 (SWH at a subcatchment scale) and that the demand met by Scenario 30 (SWH at a catchment scale) equals 29 (SWH at a subcatchment scale). The reason that they do not match is that the optimisation of SWH at a catchment scale proves to be cheaper with less storage than the sum of all storages when harvesting at a subcatchment scale.

While Table 5-12 presents a positive picture of the potential to harvest rainwater and stormwater concurrently, Table 5-13 highlights the likely problem – cost. In all scenarios, SWH is significantly cheaper than RWH and cheaper than the average price per kilolitre (R21.10) paid for municipal water in the catchment. In essence, if made to choose between RWH or SWH, the rational choice would be to choose SWH. For example, under Scenario 30, many consumers would already be incentivised to make use of harvested stormwater, but not rainwater. For the CoCT to incentivise the uptake of RWH in conjunction with SWH, it would be necessary to either offer rebates or increase the current water tariffs – as discussed in Section 5.1.1.

Table 5-12: Water demand met through RWH and SWH concurrently

Scenario	27	28	29	30
Water demand met by RWH (Mℓ/yr.)	631	631	500	500
Water demand met by SWH (Mℓ/yr.)	786	683	1046	953
Total water demand met by RWH and SWH (Mℓ/yr.)	1417	1314	1546	1453
Percentage of catchment's total water demand (%)	33	31	36	34

Table 5-13: Cost of RWH and SWH when harvested concurrently

Scenario	27	28	29	30
Average cost per kilolitre – RWH (2013ZAR)	R 50.70	R 50.70	R 62.30	R 62.30
Average cost per kilolitre – SWH (2013ZAR)	R 26.80	R 22.20	R 17.90	R 14.20
Average cost per kilolitre – RWH and SWH (2013ZAR)	R 37.44	R 35.89	R 32.26	R 30.75

The assumption that the cost of water is an overriding limitation to the uptake of RWH is only reasonable as long as it is possible to source water from alternative sources at a lower price. In the case of severe droughts or water supply system failures, the economic benefits of having water available would be significant.

5.3.2 Stormwater management benefits

The stormwater management benefits (e.g. peak flow attenuation) of RWH and SWH in conjunction show little variation from those presented in Section 5.2.3. The main differences can only be identified with a careful inspection of the hydrograph. On doing so, it is evident that the trends identified in Section 5.1.4 (RWH) and Section 5.2.3 (SWH) remain unchanged. RWH and SWH both show potential – albeit to different degrees – for attenuating the peak flow of the first storm event followed by reduced or no attenuation for the following storm events, until a sufficiently long dry period has led to the RWH or SWH storage being sufficiently emptied. This additional benefit, as noted in Sections 5.1.4 and 5.2.3, is not consistently present, but is more often present in SWH systems than RWH systems.

The conclusions of Section 5.2.3 remain in place: for maximum stormwater management benefits to be realised, it will be necessary for the storage units of the SWH system to be actively managed to ensure that there is available capacity to attenuate peak flows.

5.3.3 Viability of RWH and SWH in conjunction: Summary and discussion

Encouraging the implementation of RWH and SWH in conjunction is potentially an unwise decision. As would be expected, RWH and SWH are both most economical under maximum demand. Reducing the demand for harvested rainwater due to the use of 'cheaper' harvested stormwater will only make RWH less viable – and vice versa – as is clear in Section 5.3.1.

An important consideration would be the practicalities of having a three-pipe supply system (potable, rainwater and stormwater) as this would no doubt increase the risk of cross connections, thus posing potential public health risks. A situation can be envisaged where SWH provides water for irrigation, while RWH and potable supplies provide water indoors. Alternatively SWH could act as the primary back-up supply for RWH – assuming the harvested stormwater was of an acceptable quality. However, the decrease in constant demand for harvested stormwater will affect the benefits in terms of stormwater peak flow attenuation – unless the ponds are actively managed. The analysis of the stormwater management benefits of Scenarios 25, 27 and 29 showed no discernible difference, and as such, the implementation of RWH alongside SWH provides no additional stormwater management benefit – as was the case with RWH in isolation.

While encouraging RWH in conjunction with SWH would increase the total volume of demand met, it will come at an economic cost where either the cost per kilolitre of harvested rainwater or stormwater increases. It would also increase the financial and economic risks for the implementing agent – the CoCT, in this case. This research indicates that a long-term plan incorporating both RWH and SWH is not a viable option for the Liesbeek River Catchment and needs to be carefully considered elsewhere.

5.4 Summary and discussion of results

Each subchapter in this section contains a summary section. This section aims to provide an overview of the results and the discussion thereof.

This study set out to investigate the viability of RWH and SWH in the Liesbeek River Catchment. The results indicate, as expected, that both RWH and SWH are technically viable means of reducing potable water demand and total runoff volumes. However, RWH was found to have no real additional benefits and was found to not be an economically viable option for the majority of residential households, except for the few who use large volumes of water. While not proven, it is likely that many of these households could reduce their demand in other ways that would be financially and economically better for the household. Most notably, this research found that, although RWH may increase the initial storage in the catchment, it does not offer a reliable means of attenuating peak flows.

SWH was, however, found to be a financially and economically viable option under certain conditions. These included that there was adequate demand throughout the year, which typically implied a diversity of end uses, including toilet flushing and irrigation. It was also found that SWH, if actively managed, may be used to more reliably attenuate peak flows. Additionally, it was highlighted that SWH has a range of additional benefits (e.g. amenity value), which in an economic analysis may offset the cost of operating the system.

When RWH and SWH were considered in parallel, the results were found to be similar as to when they were considered in isolation. The major difference was that the cost of RWH increased substantially due to the reduced demand that was being met by the SWH system. As a result, RWH is, under current conditions, not a viable alternative water supply option – whether in isolation or in parallel with SWH.

5.4.1 Implementing RWH/SWH in the Liesbeek River Catchment

This study indicates that SWH is a more viable option for the Liesbeek River Catchment. The CoCT is, however, unlikely to ever be able to acquire the required land to develop SWH at the subcatchment scale due to the developed nature of the catchment. After all, it is one of the oldest settled catchments in the RSA. While it is still possible to develop SWH at the catchment scale, this would require significant investment by the CoCT, both financially and institutionally, and may take many years to develop. Since RWH systems have a life cycle of roughly 15 to 25 years, it would not be unreasonable to encourage their adoption where they are currently viable (technically, financially and economically) with a view to switching over to SWH at the end of the system's life cycle. This would require long-term planning from both individuals and the CoCT, including the development of institutional and regulatory frameworks.

Currently, there are a number of separate initiatives looking at developing the lower reaches of the Liesbeek River Catchment. These include, *inter alia*, Two Rivers Urban Park (TRUP) (CTS, 2012) and the Liesbeek River Life Plan (FOL, 2015). These plans should seriously consider the possibility of developing a centralised SWH scheme at the confluence of the Liesbeek River with a view to supplying residential and commercial properties in the Liesbeek River Catchment with a secondary source of water for non-potable uses – such as irrigation and the flushing of toilets. Such a project would require a long-term vision for the Liesbeek River Catchment, something both projects are attempting to, at some level, develop.

5.4.2 Implementing RWH/SWH in the RSA: What lessons can be learnt?

The major lesson that should be taken from this study – with respect to implementing RWH/SWH in the RSA and likely elsewhere – is that, for the optimal solution to be realised, it needs to be considered early in the development process and is not something that can necessarily be retrofitted when the need arises. Currently, it is evident that RWH is, in general, not viable in the RSA except where a property has a relatively high demand for water. However, SWH might well be a viable alternative water resource, dependent on the scale at which it is implemented, the end use for which it is used and the population density that drives the water demand. This does not mean that development needs to always develop stormwater harvesting, but a long-term view needs to be adopted, one that recognises that in the future there may be a need to harvest stormwater. Development should carefully consider the following:

- Whether SWH is a viable option for meeting non-potable water demand – this includes recognising the potential benefits that SWH might offer.
- The layout of future developments needs to carefully consider the location of public open space, ensuring that an adequate area is located in the lower reaches of the watershed(s)

being developed – and not at the top of the watershed, as is the case in the Liesbeek River Catchment. This will allow for the future development of SWH schemes.

- Properties should be developed in such a manner that the plumbing system is designed to accommodate a dual reticulation system, were one to be installed.
- Water authorities should develop the necessary regulations and guidelines to regulate the use of harvested stormwater.

If the above is considered early on, SWH could now or in the future provide urban areas in the RSA with a supplementary source of non-potable water and reduce the demand for potable water. This, in turn, may assist in ensuring that all South Africans have access to sufficient water, in line with the RSA's Constitution. Failure to consider the above may result in SWH being an uneconomical (cost of land acquisition) or impractical (not possible to move development) option. This would result in the need to consider other options that are less desirable.

6. Conclusions and recommendations

This chapter provides an overview of this thesis. Section 6.1 summarises the motivation for this research, and the literature review, highlighting the aspects most relevant to this research. Section 6.2 gives a brief overview of the method, with a focus on the Urban Rainwater Stormwater Harvesting Model (*URSHM*). Section 6.3 discusses the results, drawing out the most relevant conclusions and their pertinence. Section 6.3 examines the contributions this thesis has made to knowledge, which are categorised as general contributions, and those emanating from the results of the study itself. The chapter concludes in Section 6.5, with recommendations for further research.

6.1 Rainwater and Stormwater harvesting: State of the art

The introduction, Chapter 1, provided the motivation for and background to this research. It highlighted the fact that the Republic of South Africa (RSA) faces a range of challenges with regard to the management of water, not the least of which is water scarcity, which could be exacerbated by climate change. It further showed that the RSA is not alone in facing these challenges and, in response to these and other challenges, there has been a paradigm shift internationally to manage water more holistically. This new paradigm in water management recognises the value of water in all its competing uses, which has resulted, internationally, in a growing interest in rainwater harvesting (RWH) and stormwater harvesting (SWH).

While there has been a significant amount of research internationally focused on RWH at a site scale, where the regional-scale impacts have been considered, it has been done in a simplistic manner, which has subsequently been shown to be unreliable. Furthermore, within the RSA, there has been relatively little notable research into the impacts, whether positive or negative, of urban domestic RWH.

SWH, on the other hand, is part of a rapidly developing field internationally, and while the RSA has experience with SWH in the form of the Atlantis Water Resource Management Scheme, it is one example only that started as an interim solution while a more conventional pipeline was developed (DWAF, 2010). The literature review found that SWH is not typically considered or included in water management planning in the RSA.

The literature review, Chapter 2, highlighted that one of the major barriers to the widespread implementation of SWH is the paucity of research into the reliability and affordability of treatment technologies (Hatt *et al.*, 2004b; Philp *et al.*, 2008). This is, rightly, considered a challenge to the long-term success of SWH, as individuals may be unaware of the risks associated with SWH, or how to mitigate them.

Internationally, there is a lack of studies considering the impacts of RWH and SWH in combination. While RWH and SWH have broadly similar benefits, there are distinct differences as well. If roof runoff is managed at the site scale, it might result in a reduction of stormwater runoff, consequently compromising the viability of SWH. This has not, to date, been assessed.

Chapter 2 also reviewed the different methods for modelling RWH and SWH. The different factors and data requirements for each approach were discussed. The literature review highlighted that access to data is important for studies of this nature. As a result, it was not surprising that many of the most advanced studies are from ‘developed’ countries, such as Australia, where data are more readily available. The RSA is a developing country where useful data are often not available, so this potentially poses a problem. Data availability allows for more complex models, which in principle should provide more accurate results. However, currently, there are no ‘results’, partly because of the limited data. This is potentially problematic, as the literature review suggests that uncertainty as to the costs and benefits of alternative approaches to water management, such as RWH and SWH, are potentially barriers to their wider acceptance and adoption – both institutionally and socially. There is, therefore, a need for a study such as this one, that considers the benefits and costs of RWH and SWH, which could be used to motivate for or against the adoption of RWH and/or SWH in the RSA. A study of this nature could then be built on through in-depth social and economic studies that investigate other aspects related to the viability of such systems.

6.2 Overview of the method

In order to address the identified research question it was decided to make use of a case study. The selection of an appropriate catchment is discussed in Chapter 3. Chapter 4 presents the methods developed and applied in this thesis on the case study catchment. The most important aspects of Chapter 4 are: the disaggregation of water demand data (Section 4.2.3.5) and the development of the Urban Rainwater Stormwater Harvesting Model (*URSHM*) (Section 4.4). These methods and the tools developed provide the basis for future studies of a similar nature to be undertaken.

The disaggregation of water demand data provides a method for reasonably assessing the per-capita and household demand for properties with and without water demand data. The *URSHM* offers a means of assessing the viability of RWH and SWH in areas where these are proposed as interventions, and for which no data as to the size of proposed – or installed – RWH or SWH systems exist. The *URSHM* makes use of accepted methods for hydraulically modelling RWH and SWH systems, in combination with a whole life cycle costing approach, to assess the economic viability of a RWH/SWH system. The results of the hydraulic and economic calculations are used in four objective functions that rationally size a RWH/SWH system, dependent on the selected objective function.

Chapter 4 also describes the methods used to overcome the challenges of modelling the impacts of climate change, including estimating evaporation. It concludes with an overview of the 30 scenarios that were analysed in order to assess the viability of RWH and SWH in the RSA. The analysis of these 30 scenarios form the basis of the results and conclusions with respect to the viability of RWH and SWH in the RSA.

6.3 Viability of rainwater and stormwater harvesting

6.3.1 The viability of rainwater harvesting

The analysis of the viability of RWH for residential use indicated that, in general, RWH offers a means of reducing municipal water demand; however it is only economically viable for the minority of property owners who have high water demand, most commonly the more affluent households with larger properties in wealthier areas. RWH is only a viable option economically when harvested rainwater is used for as many end uses as possible, and the largest possible catchment area (as much of the roof area as possible) is connected to the RWH storage tank.

The analyses further indicated that climate change may have an extreme impact on RWH depending on which climate change model is used – in some cases, positive, and in others, negative. Of the 31 downscaled climate models considered, the general trend however, was for a minimal change in system performance and, hence, viability.

The investigation of the potential stormwater management benefits of RWH indicated that RWH was an unreliable means of attenuating peak flows, even for small storm events. This is not to say that RWH never attenuates peak flow, but rather, that it is dependent on a number of factors. As such, RWH cannot be relied upon to attenuate peak flows. This potential benefit is often cited in stormwater management guidelines (e.g. Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013).

6.3.2 The viability of stormwater harvesting

The analysis of the viability of SWH for residential use indicated that, in general, SWH offers a means of reducing municipal water demand in the Liesbeek River Catchment by up to 20% whilst potentially offering other benefits such as water quality treatment, amenity value and attenuation. Additionally, through active management of the SWH storage units, there is the potential to significantly enhance the SWH system's ability to attenuate peak flows during both small and extreme events. The reduction in potable water demand would be significant for the CoCT, but would require that all residents and businesses make use of harvested stormwater for, at least, flushing toilets and outdoor irrigation. In order for this to become a reality, it would likely require changes in the regulations related to the supply of water in the CoCT. In the meantime, the non-financial benefits should be recognised and may act to partially offset the costs of operating the SWH system.

Climate change is increasingly a concern for water resource planners. While some climate change scenarios indicated significant decreases in runoff volumes, others showed limited change. However, due to the significant volumes of runoff, the impact on the proposed SWH schemes is relatively insignificant. Overall, it seems reasonable to expect a slight decrease in volumetric reliability.

Overall this investigation found that SWH may, under certain conditions, be a viable alternative to potable water in the Liesbeek River Catchment. SWH was found to be most viable where: the harvested stormwater is used for as many end uses as possible – notably toilet flushing and irrigation; population densities are higher, which equate to a higher and more constant water demand (toilet flushing throughout the year); and the additional benefits are actively developed. However, while technically and economically SWH might be an option in the Liesbeek River Catchment, the social, institutional and political implications would need to be further investigated.

6.3.3 The viability of rainwater and stormwater harvesting

The analysis of the viability of implementing of RWH and SWH in conjunction indicates that this is not advisable. RWH and SWH are both, as would be expected, most economical under the maximum demand. Reducing demand for harvested rainwater due to the use of ‘cheaper’ harvested stormwater will only make RWH less viable – and vice versa. While harvesting rainwater and stormwater concurrently will increase the total volume of water demand met thus reducing the demand for potable water, it will come at an economic cost where either the cost per kilolitre of harvested rainwater or stormwater increases. RWH is already not economically viable, and increasing the cost of SWH makes it a less viable option. It also increases the financial and economic risks for the implementing agent – the CoCT in this case. The implementation of RWH and SWH in conjunction is only useful in the case where there is no other source of water and the cost of the harvested water is less of concern.

6.4 Contributions to knowledge

This thesis has contributed to knowledge in several ways. These are discussed in relation to general contributions and the RSA separately.

6.4.1 General contributions

During this research, several challenges were identified and overcome. In doing so, a number of procedures were developed and insights gained that could assist others to undertake similar studies. These include:

- The procedure for estimating water demand, including: the estimation of indoor water demand, data patching methods and outdoor demand calibration. These procedures require only billing data and a single shape file, which may be easily captured from orthophotos.
- The development of the *URSHM* and its objective functions for rationally sizing a RWH/SWH system. While this is currently in the form of a set of Excel workbooks, it could, in time, form the basis of a distributable tool that could streamline such studies.
- A critique of current approaches to assessing the impact, especially of attenuation, that RWH and SWH might have at the catchment scale.

- Confirmation that the use of linear extrapolation of an ‘average household's RWH at the system scale to estimate the catchment scale effects is unreliable and overestimates water demand savings by roughly 10% (dependent on end use), whilst underestimating overflow by between 10% to 25%.
- A method for including the modelling of RWH in a continuous stormwater model was proposed and used that did not treat RWH as a constant depression storage or constant volume of storage.
- It was also shown that, while RWH should not be considered a stormwater management tool, it does, at times, significantly impact on runoff volumes and flow rates – just not reliably. This finding is important, because future modelling, and especially the calibration of continuous stormwater models for urban catchments, could be made significantly more challenging if RWH were to be widely adopted and there were no baseline data prior to its adoption.

6.4.2 The viability of RWH and SWH in South Africa

This study provides the first comprehensive analysis of the viability of RWH and SWH – considering a diversity of potential fit-for-purpose end uses and the impact of climate change, including system life cycle costing – at the catchment scale in the RSA. While other studies have looked at elements of RWH and SWH in the RSA, this study has used and developed methods that avoid the use of linear extrapolation to assess the benefit of RWH and SWH in the RSA. Specifically this study has found that, in the Liesbeek River Catchment:

- For the majority of households, RWH is not a viable option.
- RWH is not a reliable means of attenuating peak flows, even for small storm events. Therefore, it should not be considered as a stormwater management option.
- SWH is a viable option where there is adequate demand, but would require high levels of adoption.
- SWH is a reasonable means of attenuating peak flows for small recurrence interval storm events.

While this study presents the first indepth study of RWH and SWH in the RSA, its results may not be transferable across the whole of the RSA. As such further such studies may be required before it is possible to infer the potential viability of RWH and SWH in the RSA as a whole.

6.5 Recommendations for further research

This research has focused on the financial, economic, technical and practical viability of RWH and SWH. It has highlighted several areas that require further research:

- There is a need to investigate, in the RSA, the social acceptability of RWH and SWH as alternative water resources. While such studies have been undertaken elsewhere (e.g. Australia), it is important to investigate these issues locally and in-depth. Experience within the RSA with respect to treated effluent re-use has shown that, even if there is acceptance from leaders in a community, there may not be general acceptance from the community.
- The analysis of the 2011 RSA Census highlighted that, in the Liesbeek River Catchment, there was a high proportion of properties being rented, especially around the University of Cape Town's campus. There is a need to investigate whether tenants, who in the RSA do not always pay separately for water, would be willing use harvested rainwater and/or stormwater instead of potable municipal water.
- There is a need to gain a better understanding of water demand in the RSA. This would require the monitoring of water demand for a range of sectors and for specific end uses across a diversity of climatic, social, spatial and temporal contexts. This would also assist in developing stochastic water demand generation models that could be calibrated for the RSA and assist in many other studies.
- There is a need to investigate the water demand and end-uses of a range of commercial facilities. There is very poor, generalised and unreliable guidance as to how water is used in commercial properties. For example, a restaurant, grocery store and bank will all use water in different proportions for different end-uses. To be able to reasonably understand the potential for RWH/SWH as an alternative water supply source for these end-uses, it will be important to have a more in-depth, and reasonable understanding of how water is used in different commercial facilities.
- This study, as a result of the conditions in the selected catchment, focused on the open storage of stormwater. Future studies should consider MAR recharge and the associated costs.
- While this study has presented results within the RSA context, there would be value in repeating the study in a range of settlements across the RSA. This would account for the differences in outdoor demand and climatic variations experienced in the RSA.

This research has also highlighted many issues that need to be addressed to enable research in the future:

- It is necessary to install basic monitoring and data logging (rainfall, flow and quality) equipment across urban areas in the RSA. This would help to address the urgent need for basic calibration data. While the Liesbeek River catchment had two South African Weather Service rain gauges and one operational flow gauge, this was barely adequate for this study. There are many, if not the vast majority of, catchments that have neither rain gauges nor river flow monitoring stations. The collection of this data is crucial, especially if future modelling is to be undertaken over longer periods than 10 years.

- This study also highlighted that water management / conservation tools such as RWH might significantly impact the hydrology of a catchment in an uncertain manner. Without some basic baseline flow gauges for continuous monitoring, it will make the calibration of models in the future increasingly more complicated / difficult.
- It was evident that stormwater models in the CoCT, and likely elsewhere, remain event models. As indicated in the literature, such models are now outdated. Cities across the RSA should aim to ensure that all future stormwater models are developed and calibrated as continuous models.
- While the model is currently in the form of a spreadsheet, it could in time be developed into a standalone application. This would improve the ease with which future studies are undertaken.

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Appendix A : South African context

South Africa (RSA) is a water-stressed, urbanising, developing country facing a range of challenges with respect to water management, *inter alia,* resource shortages, environmental degradation, fragmented institutional structures and basic services backlogs (Kok & Collinson, 2006; DEA, 2010; UNEP, 2010; RSA, 2011a, 2011b; Fisher-Jeffes *et al.*, 2012; DWA, 2013). It also faces the challenge of dealing with the legacy of Apartheid – the policy followed prior to 1994 of '*separate'* development for different ethnic groups – including large disparities in income, unemployment and service delivery backlogs (UNEP, 2010; RSA, 2011a). This section provides an overview of the water resource challenges that the RSA faces. As this thesis makes use of a case study located in the City of Cape Town, this section provides some additional background on the water crisis specifically facing the City of Cape Town.

A1 Water resources – RSA

In 2005, the RSA was estimated to be the 29th driest country out of 193 countries, based on the total actual renewable water resources per person per year (Muller *et al.*, 2009). The situation is aggravated in that freshwater resources in the RSA are unevenly distributed and disproportionally available relative to demand (UNDP *et al.*, 2000; Blignaut & Heerden, 2009; UNEP, 2010; Carden, 2013). Approximately 60% of runoff occurs from only 20% of the country's total surface area. For half the country, the runoff is approximately 5% of rainfall, well below the international average of 35% (Schulze, 2005b). The low runoff results from many factors, most notably the level of aridity in the RSA (Schulze, 2005b). Meanwhile, surface water resources are already almost fully exploited across the country, with over 320 dams capturing 66% of the total runoff and inter-basin transfers forming a critical part of South Africa's water management infrastructure (DWAF, 2004; Addams *et al.*, 2009; DWA, 2013).

The first National Water Resources Strategy (DWAF, 2004) indicated that, by 2050, the RSA will have exceeded the limits of its economically usable, land-based water resources. Addams *et al.* (2009), however, predict that, by 2030, South Africa will already be facing significant water resource shortages, with an average supply shortfall of 17%. The situation is notably worse in certain catchments such as the Upper Vaal (31% deficit), the Olifants (39% deficit) and the Berg (28% deficit).

Currently, about 60% of the RSA's population lives in urban centres (RSA, 2011a, 2011b). Urbanisation is not only expected to continue in South Africa (Kok & Collinson, 2006; RSA, 2011a, 2011b), it is likely that almost all of the RSA's population growth will take place in urban areas (RSA, 2011b). For example, while the RSA's current population growth is estimated at 1.34% per annum (StatsSA, 2013b), urban areas are growing at approximately 5% per annum (DWA, 2013). The disparity between population growth and growth in urban areas is an indication of the levels of urbanisation.

Table A-1 highlights that agriculture and urban / municipal consumers are currently and will continue to be the sectors using the greatest amount of water. While agricultural demand will likely remain relatively stable, the bulk of the increased water demand is expected to come from urban water users (Basson *et al.*, 1997; Addams *et al.*, 2009). This is already being seen, as '*over the last ten years water consumption in the domestic sector has increased from 22% to 27% of the total resource* (DWA, 2013)'. Addams *et al.* (2009) further note that by 2030, 50% of urban water demand will likely be from the wealthiest quintile of the population, a sector that could specifically be targeted to reduce their demand for municipal water.

Table A-1: Water demand by sector in South Africa

Water demand sector	Estimated sectoral water demand	
	2004 (DWAF, 2004)	2010 (UNEP, 2010)
Agriculture	>60%	62.7%
Industry	-	6.1%
Urban Use* / Municipalities**	23%	31.2%
Other	15%	-

* DWAF (2004) ** UNEP (2010)

Ashton (2000); Scholes (2001); Turton (2008); Addams *et al.* (2009); Muller *et al.* (2009); RSA (2011b); DWA (2013) amongst others all highlight the social and economic consequences that water shortages could have on South Africa. The National Water Resources Strategy (DWAF, 2004) stated in 2004 that South Africa has enough water 'for the foreseeable future', but almost 10 years later, the 'foreseeable future' seems to have changed. Water scarcity now appears to be a real and major threat to South Africa's economic and social future. On the other hand, Muller *et al.* (2009) note that Singapore, a country with limited water availability, has achieved substantial economic growth while countries with abundant water supplies such as Bangladesh and the Democratic Republic of Congo (DRC) have not seen the same economic growth. Therefore, physical water availability, if well managed, need not hinder economic growth.

The second National Water Resource Strategy (NWRS2) (DWA, 2013) is clear that water supply authorities across the RSA need to consider desalination (including the desalination of sea water, treated effluent, and acid mine drainage) to ensure the adequate availability of water. DWA (2013) does, however, recognise the potential cost implications of desalination and notes there is a need to consider a diversity of resources. Meantime the NWRS, fails to consider rainwater and stormwater harvesting as a potential resource in the RSA's residential areas.

A2 Water resources – Cape Town

'*Water is a scarce resource in South Africa and also in Cape Town* (CoCT, 2007)'. The situation in Cape Town reflects that of many of South Africa's urban areas. The failure to meet the growing water demand in the City of Cape Town (CoCT) is recognised as '*a limiting constraint to the social upliftment and economic prosperity of the city* (CoCT, 2007)'. The

City's water conservation and demand management strategy has been moderately successful, but currently this only aims to delay the need to invest in a new water resource scheme until 2029 (CoCT, 2011b). Meantime many factors, including climate change and changes in the environmental reserve requirements, may negatively affect the City's water resources. Whether by 2029 or a later date, the CoCT will likely require new water resources.

Kleynhans *et al.* (2011) considered what future surface water options were available for supplying the Western Cape – of which Cape Town is a major component. The options considered all took water from rivers and diverted it to storage. All the options considered were feasible; however, any *'scheme that diverts or impounds water will impact on the downstream river flows and ecology, sometimes at a significant level all the way to the estuary* (Kleynhans *et al.*, 2011)'. The CoCT's 2011-2016 Water Services Development Plan (WSDP) (CoCT, 2011b), the Western Cape Water Supply System Reconciliation Strategy Study (Ninham Shand & UWP Consulting, 2007) and the Western Cape Sustainable Water Management Plan (PGWC & DWA, 2012) all identify a number of alternatives that may assist in ensuring water demand never exceeds available supply. These include, *inter alia*, the development of the Table Mountain Group Aquifer, the Cape Flats Aquifer, the Lourens River Diversion Scheme, the Eerste River Diversion Scheme, water re-use, water conservation and water demand management (WC/WDM), desalination, and the raising of existing dam walls. While WC/WDM assists in reducing water demand, which in itself has an environmental benefit, the environmental impacts were not fully considered for any of the other alternatives, nor whether the intended water use was appropriate for the anticipated quality of water abstraction (e.g. whether potable water should be used for irrigation at all). Instead, the proposed alternatives are all focused on ensuring that centralised bulk infrastructure is able to meet demand. Little consideration was given to decentralised infrastructure.

Appendix B: Urbanisation and urban water management

Marsalek *et al.* (2006) note that '*urbanisation affects many resources and components of the environment in urban areas and beyond*'. Water is one such resource. Urbanisation is, therefore, an important issue for water managers in urban areas. Since 1992, when the concept of Integrated Water Resource Management (IWRM) was adopted as part of Agenda 21 (UN, 1992a), many approaches to water management – at all scales – have been developed that promote more holistic management of the water cycle. The following section outlines the impact of urbanisation on water resources, current approaches for managing / mitigating these impacts, and how rainwater and stormwater harvesting could form part of a holistic management response.

B1 Urbanisation and the water cycle

Urbanisation results in the natural water cycle being altered (Vörösmarty & Sahagian, 2000; AMEC *et al.*, 2001; Shuster *et al.*, 2005; Philp *et al.*, 2008; DWAF, 2010; Lee *et al.*, 2010; Wong *et al.*, 2012). Marsalek *et al.* (2006) broadly summarise the changes in water flows as an increase in surface imperviousness, changes in runoff conveyance networks and increased water demand. Urbanisation also leads to changes in the physical, chemical and microbiological quality of water in the water cycle (Marsalek *et al.*, 2006; Fletcher *et al.*, 2008; Aryal *et al.*, 2010; Armitage *et al.*, 2013). The main changes to the water cycle are illustrated in Figure B-1.

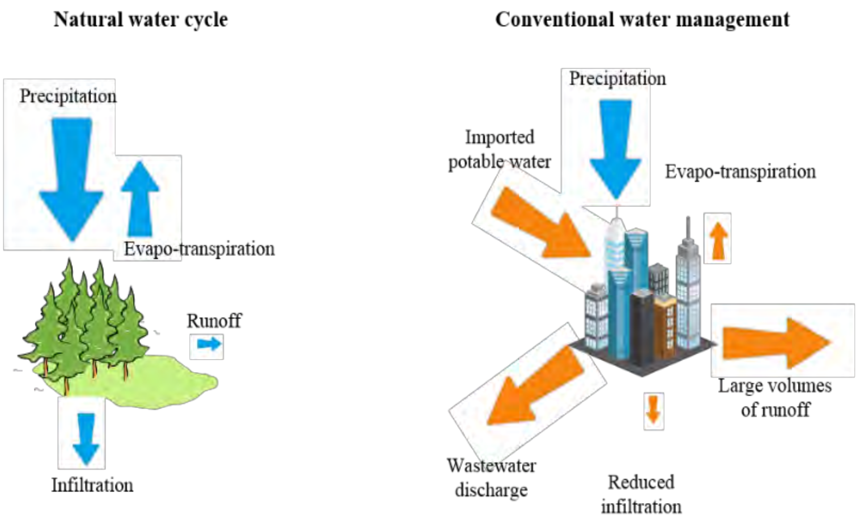


Figure B-1: The Urban Water Cycle, showing changes to the natural water cycle with traditional urban development (Hoban & Wong, 2006)

B1.1 An increase in surface imperviousness

Urbanisation typically results in an increase in the impervious surface area, which has significant impacts on a watershed's hydrology (Leopold, 1968; Walsh, 2000; CSIR, 2005b; Shuster *et al.*, 2005). Leopold (1968) noted that the volume of runoff is primarily determined by the soil's infiltration characteristics. The increase in the impervious area associated with urbanisation results in greater volumes of runoff and higher peak flows. Furthermore, the increase in impervious surfaces generally results in a decrease in infiltration that in return decreases groundwater recharge (AMEC *et al.*, 2001). On the other hand it has been shown that, in some cities, leakages from the water reticulation system may actually be increasing local groundwater recharge (Lerner, 2002).

B1.2 Changes in runoff conveyance networks

Urbanisation has resulted in significant changes to how runoff is conveyed in most urban areas (Marsalek *et al.*, 2006). Typically, natural channels have been replaced with hydraulically highly efficient concreted channels. While the increase in impervious areas results in increased runoff volumes, Fletcher *et al.* (2008) highlighted that 80% to 90% of the increase in peak flows can be explained by the nature of the conveyance network. Alternative approaches to stormwater management, such as SuDS, encourage the retardation and infiltration of stormwater runoff through a form of 'naturalisation' of stormwater infrastructure in an attempt to reduce peak flows (Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013).

B1.3 Increased water demand and improved sanitation

The increasing population together with the inevitable demand for better services that are a consequence of urbanisation leads to a rapidly increasing demand for water (Marsalek *et al.*, 2006). The degree to which water demand increases is the result of numerous factors that are discussed further in Section 2.4.

An increase in water demand is typically associated with improved sanitation provision – to remove or handle wastewater. The provision of 'wet' sanitation services requires the introduction of wastewater treatment works which are intended to protect receiving water bodies from the impacts of raw sewage. However, due to poor system maintenance and operation (e.g. failing to manage stormwater ingress), WWTW are often not as effective as intended and instead introduce raw or partially treated sewage into receiving water bodies in a concentrated (point source) manner. The combination of raw and partially treated sewage leads to degraded water quality (Leopold, 1968). The impacts that WWTWs are having on receiving water bodies – whether from poor quality effluent or the direct release of effluent into rivers – are prevalent in South African cities. A recent CoCT study to determine what additional resources are needed to manage pollution in stormwater and rivers in Cape Town identified that WWTW were significant point sources of pollution affecting the receiving water bodies' water quality. The report recommended that they needed to be fixed / upgraded in order to improve water quality (PDNA, 2011). This recommendation will need to be implemented to ensure that the quality of harvested stormwater is acceptable, without requiring advanced treatment (e.g. reverse osmosis). Additionally, as discussed in Section 2.6.9, in order to avoid a 'disgust' factor – revulsion or deep-seated negative response – affecting the social acceptance of harvested stormwater as a resource, it is necessary to protect SWH systems from pollution, especially sewage.

B1.4 Decreasing stormwater quality

Urbanisation also leads to decreasing stormwater quality (Duncan, 1995; Makepeace *et al.*, 1995; AMEC *et al.*, 2001; Marsalek *et al.*, 2006; Lee *et al.*, 2010). Table B-1 summarises the pollutants typically conveyed by stormwater and their effect on water quality. Buys & Aldous (2009) noted that stormwater runoff is a major contributor to deteriorating water quality in the urban water systems of cities in RSA. Conventional drainage systems are generally focused on managing local flooding and largely ignore the need to preserve or improve water quality (AMEC *et al.*, 2001; Woods-Ballard *et al.*, 2007; Burns *et al.*, 2010; Armitage *et al.*, 2013).

B2 Integration of urban water management – a paradigm shift

Section B.1 highlights that urbanisation has resulted in wide-scale changes to the water cycle. These changes to the water cycle have significant environmental impacts. As a result, it is widely accepted that a new, holistic approach to urban water management is required (Mitchell *et al.*, 2001; Marsalek *et al.*, 2006; Brown *et al.*, 2008; Wong & Brown, 2008; Jacobsen *et al.*, 2012; WWAP, 2012). In 1992, in response to the realisation that the scarcity '*and misuse of fresh water poses a serious and growing threat to sustainable development and protection of the environment* (UN, 1992b)', the delegates at the International Conference on Water and the Environment adopted the following four 'Dublin Principles' for water management:

i) Fresh water is a finite and vulnerable resource, essential to sustain life, development and the environment

ii) Water development and management should be based on a participatory approach, involving users, planners and policy-makers at all levels

iii) Women play a central part in the provision, management and safeguarding of water

iv) Water has an economic value in all its competing uses and should be recognized as an economic good

Table B-1: Summary of urban stormwater pollutants (AMEC *et al.*, 2001)

Constituents	Effects
Sediment – Suspended Solids, Dissolved Solids	• Stream turbidity • Habitat changes • Recreation/aesthetic loss • Contaminant transport • Filling of lakes and reservoirs
Nutrients – Nitrate, Nitrite, Ammonia, Organic Nitrogen, Phosphate, Total Phosphorus	• Algae blooms • Eutrophication • Ammonia and nitrate toxicity • Recreation/aesthetic loss
Microbes – Total and Faecal Coliforms, Faecal Streptococci Viruses, *E.Coli*, Enterococci	• Ear/Intestinal infections • Shellfish bed closure • Recreation/aesthetic loss
Organic Matter – Vegetation, Sewage, Other Oxygen-demanding Materials	• Dissolved oxygen depletion • Odours • Fish kills
Toxic Pollutants – Heavy Metals (cadmium, copper, lead, zinc), Organics, Hydrocarbons, Pesticides/Herbicides	• Human & aquatic toxicity • Bioaccumulation in the food chain
Thermal Pollution	• Dissolved oxygen depletion • Habitat changes
Trash and debris	• Recreation/aesthetic loss

The conference commended these principles to the United Nations Conference on Environment and Development held in Rio de Janeiro later the same year, which in turn proposed Integrated Water Resources Management (IWRM) as a means to '*satisfy the freshwater needs of all countries for their sustainable development* (UN, 1992a)'. IWRM essentially aims to ensure water is managed in line with the Dublin Principles (UN, 1992b). Ten years later, at the 2002 World Summit on Sustainable Development (WSSD), delegates concluded that, over the years, it had been shown that the IWRM approach would be critical to achieving many of the Millennium Development Goals (MDG), and that, by 2005, IWRM should form part of all national and regional planning (Faures & D'Amore, 2006). Closely related to IWRM is Integrated Urban Water Management (IUWM), which can be viewed as a subset of IWRM (Maheepala *et al.*, 2010). The difference between IUWM and IWRM is the spatial scale and focus. IWRM focuses on water allocation problems at the river basin level. The river basins could include both urban and agricultural areas. IUWM, on the other hand, focuses on water supply, wastewater and stormwater services in urban areas only (Maheepala *et al.*, 2010).

Internationally, it is becoming increasingly accepted that a new approach to urban water management is needed – especially in developing countries (Jacobsen *et al.*, 2012). IUWM's primary aim is to ensure the multi-functionality of urban water services, to optimise the outcomes achieved by the system as a whole (Mitchell, 2006). However, due to the realisation that, to achieve the goals of '*transforming cities to more sustainable urban water cities … will require a major socio-technical overhaul of conventional approaches* (Wong & Brown, 2008)', a plethora of approaches to and terms associated with urban water management have emerged. Fletcher *et al.* (2014) note that: '*Terminology evolves locally and thus has an important role in establishing awareness and credibility of new approaches and contains nuanced understandings of the principles that are applied locally to address specific problems.*' A selection is highlighted in Table B-2. These approaches are similar, and all closely reflect the Dublin / IWRM / IUWM principles. In essence, these approaches are largely synonymous, with minor technical differences to do with either scale or a particular focus. A comprehensive overview of the evolution and application of many of these terms is provided by Fletcher *et al.* (2014).

Table B-2: Current terminology reflecting alternative approaches to urban water management

Term	Definition
Green Infrastructure (GI)	'Green infrastructure is an approach that communities can choose to maintain healthy waters, provide multiple environmental benefits and support sustainable communities. Unlike single-purpose grey stormwater infrastructure, which uses pipes to dispose of rainwater, green infrastructure uses vegetation and soil to manage rainwater where it falls (USEPA, 2012a)'.
Low Impact Development (LID)	'LID is an approach to land development (or re-development) that works with nature to manage stormwater as close to its source as possible. LID employs principles such as preserving and recreating natural landscape features, minimizing effective imperviousness to create functional and appealing site drainage that treat stormwater as a resource rather than a waste product (USEPA, 2012b)'.
Sustainable Urban Drainage Systems (SUDS/SuDS)	'Sustainable Drainage Systems offer an alternative approach to conventional drainage practices by attempting to manage surface water drainage systems holistically in line with the ideals of sustainable development (Armitage *et al.*, 2013)'.
Water Sensitive Urban Design (WSUD)	'Water Sensitive Urban Design is an approach to urban planning and design that integrates land and water planning and management into urban design. WSUD is based on the premise that urban development and redevelopment must address the sustainability of water (Engineers Australia, 2006)'.
Low Impact Urban Design & Development (LIUDD)	'Low impact and water-sensitive approaches to urban development have been evolving in New Zealand since the late 1990´s… Key elements include working with nature, avoiding or minimising impervious surfaces, minimising earthworks in construction, utilising vegetation to assist in trapping sediment and pollutants (WSUD & LIUDD Working Group, 2012)'.
Total Water Cycle Management (TWCM)	'TWCM recognises that all elements of the water cycle are interdependent—a decision made in one part of the water cycle impacts other parts of the cycle. All the elements of the water cycle should be considered — separately and in combination. TWCM also requires integration of infrastructure planning with land use planning (Water by Design, 2010)'.

B3 Water Sensitive cities

Brown *et al.* (2009) presented a framework (Figure B-2) for '*underpinning the development of urban water transitions policy and city-scale benchmarking at the macro scale*'. The aim of the framework was to assist urban water managers to enable a transition to more sustainable water management and ultimately to the realisation of what was termed 'Water Sensitive Cities' (WSC). To realise WSC, urban water managers would need to intentionally and progressively implement water-sensitive principles through the use of WSUD. Simply put: '*Water Sensitive Urban Design is the process and Water Sensitive Cities are the* [desired] *outcome* (Wong *et al.*, 2012)'.

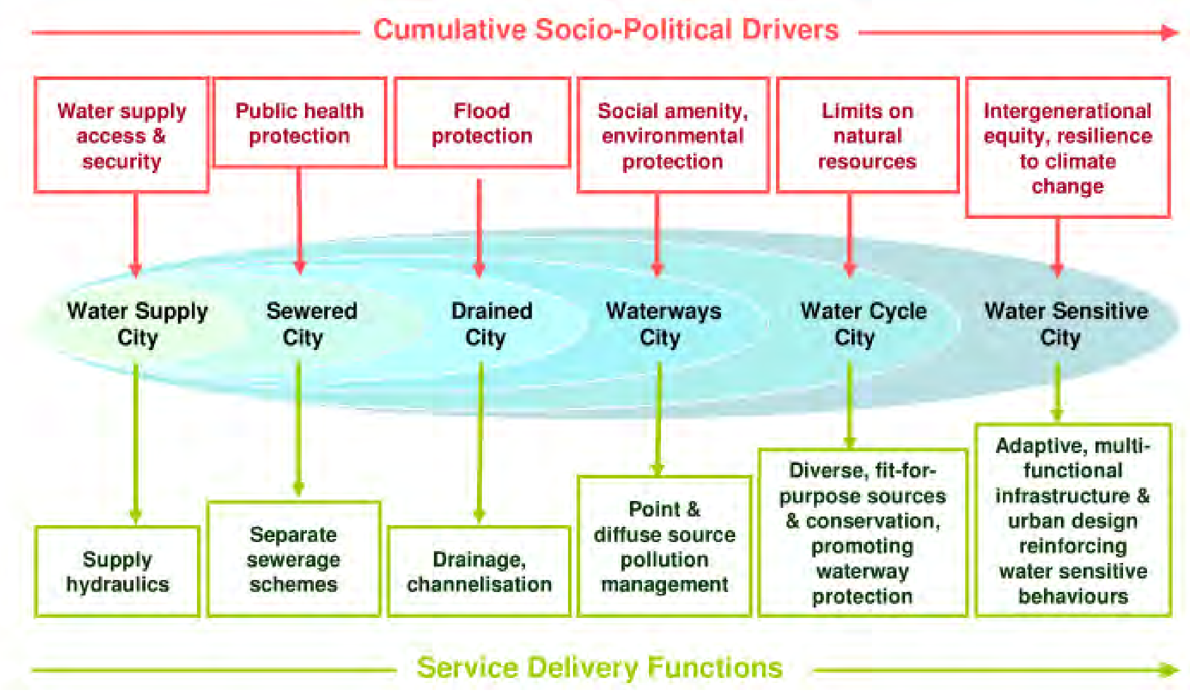


Figure B-2: Urban Water Management Transitions Framework (Brown *et al.*, 2009)

Brown *et al.* (2009) present the typology of six different states that cities may transition through in becoming more sustainable. They further present the associated socio-political drivers and service delivery functions leading to each transition. The framework presents a useful tool for benchmarking a city's progress (either forwards or backwards) at a macro scale. The framework can also be used as a heuristic device to support the development of urban water policy to enable further (positive) transition, the goal being to achieve a WSC. While Brown *et al.* (2009) detail the characteristics of a WSC, Wong & Brown (2008) propose 'three pillars' that characterise a WSC. These are:

i) **Cities as Catchments**: access to a diversity of water sources underpinned by a diversity of centralised and decentralised infrastructure;

ii) **Cities Providing Ecosystem Services**: provision of ecosystem services for the built and natural environment; and

iii) **Cities Comprising Water Sensitive Communities**: socio-political capital for sustainability and water sensitive decision making and behaviours'.

While no city can yet claim to be a WSC (Wong & Brown, 2008), there are examples of cities across the world that are implementing elements of WSUD – whether they call it WSUD or not (Ward *et al.*, 2012).

B4 Water Sensitivity – what does it actually mean?

Wong & Brown (2008) proposed that a WSC would have the following characteristics: a city with access to a diversity of water sources (both centralised and decentralised); a city providing ecosystem services for the built and natural environment; and a city with socio-political capital for sustainability and water sensitive decision making and behaviours. Wong & Ashley (2006) suggest that *''Water Sensitive' defines a new paradigm in integrated urban water cycle management that integrates the various disciplines of engineering and environmental sciences associated with the provision of water services'.* Armitage *et al.* (2014) further proposed that water sensitivity in the RSA required an appreciation that: the RSA is a water-scarce country, access to water is a basic human right, water is an 'economic good', water management should be based on a participatory approach and water is a finite and vulnerable resource. They also emphasised that water sensitivity required that water is used in a '*fit for purpose*' manner, which is in line with the principles of IUWM that consider water supply, drainage and sanitation as components of an integrated physical system known as the urban water cycle (Carden, 2013).

In essence, water sensitivity takes IUWM further by recognising that achieving 'sustainable urban water cities' would require a major socio-technical overhaul of conventional approaches (Wong & Brown, 2008). To achieve this overhaul, the concept of WSUD was developed. WSUD brings 'sensitivity to water' into urban design thereby ensuring that water is given due prominence within the urban design processes. 'Water Sensitivity' defines*: 'a new paradigm in integrated urban water cycle management that integrates the various disciplines of engineering and environmental sciences associated with the provision of water services including the protection of aquatic environments in urban areas* (Wong & Ashley, 2006).' In essence WSUD integrates the social and physical sciences in an attempt to ensure that community values and aspirations of urban spaces are realised.

B5 Water Sensitive settlements

In the RSA, the concept of Water Sensitive Settlements (WSS) (broadly based on the Australian WSC approach) has been proposed (Armitage *et al.*, 2014). This proposal was the outcome of a RSA Water Research Commission (WRC) project (Project Number: K5/2071) entitled: '*Water Sensitive Urban Design (WSUD) or Low Impact Design (LID) for improving water resource protection/conservation and reuse in urban landscapes*'. LID is a term that originates from the USA, but has recently been superseded by the more inclusive term 'Green Infrastructure'. WSUD originates from Australia, and the philosophy has been evolving since the early 1990s. Recently, WSUD has also been included in many policy documents at all levels of government in South Africa, most notably the RSA's Climate Change Response Strategy (RSA, 2011a). As a result, WSUD was selected as the approach (or term for the approach) that would be used in the RSA. The WRC is, in a current project, considering the use of the term 'Water Sensitive Design (WSD)', to allow for a broader focus on the development of not only urban and peri-urban communities, but also those in rural environments (WRC, 2014).

The Water Research Commission (WRC) project (Project Number: K5/2071) included the development of a theoretical framework. After an investigation of the challenges and opportunities for WSUD in the RSA (see Fisher-Jeffes *et al.*, 2012), the conceptual framework developed by Brown *et al.* (2009) for 'benchmarking' evolution towards a Water-sensitive City (WSC) was used as a starting point for the development of a framework for the RSA. The WSC vision was considered relevant to the RSA and may assist in addressing some of the challenges facing the country's water sector, but it needed to be contextualised for the unique development challenges the RSA faces. This included, *inter alia*, expanding the definition of 'city' to include a broad range of settlement types, recognition of the legacy of apartheid in RSA and the development of a context-specific framework (framework for WSS) – herein referred to as the 'Framework'. A key outcome during the development of the Framework was that the concept of 'water sensitive', and the challenges and opportunities for 'water sensitivity', are context specific and that the tools and designs developed for/in the RSA and other developing countries will likely vary from those implemented in developed countries.

The framework has four complementary components as described in the following sections.

B5.1 Research component

There is a need for ongoing research as well as capacity building to develop RSA-relevant guidelines for the realisation of WSS. The notion of the '4T' approach (tools, transfer, tactics and trials) was thus conceptualised as a useful, cyclical strategy to support the promotion and adoption of WSUD in RSA. It suggests there is a need for: the ongoing development of tools (manuals, guidelines etc.); the transfer of knowledge to appropriate officials; the application of tactics for encouraging implementation of WSUD in the RSA (such as getting new policies written); and the testing of WSUD technologies through trials (pilot studies, small-scale developments etc.).

B5.2 Vision component

The Brown *et al.* (2009) framework suggests six transition states for urban water management – with their associated socio-political drivers and delivery functions – that are used to underpin the development of policy and to benchmark a city's progress (either forwards or backwards) at a macro scale. Most formally developed areas in the RSA's cities could be described as 'drained cities'. As a consequence of being envisaged mostly for cities in the developed world, the framework does not take into account the impact of many factors unique to the RSA (Fisher-Jeffes *et al.*, 2012). It was, therefore, adapted for the RSA context, as shown in Figure B-3. If the RSA wishes to transition towards WSS in line with current international best practices, the legacy of apartheid – that resulted in significant backlogs in infrastructure (e.g. poorly serviced informal settlements), which the government is attempting to address – will need to be recognised. Any attempt to transition to WSS's will need to consider both formal and informal areas.

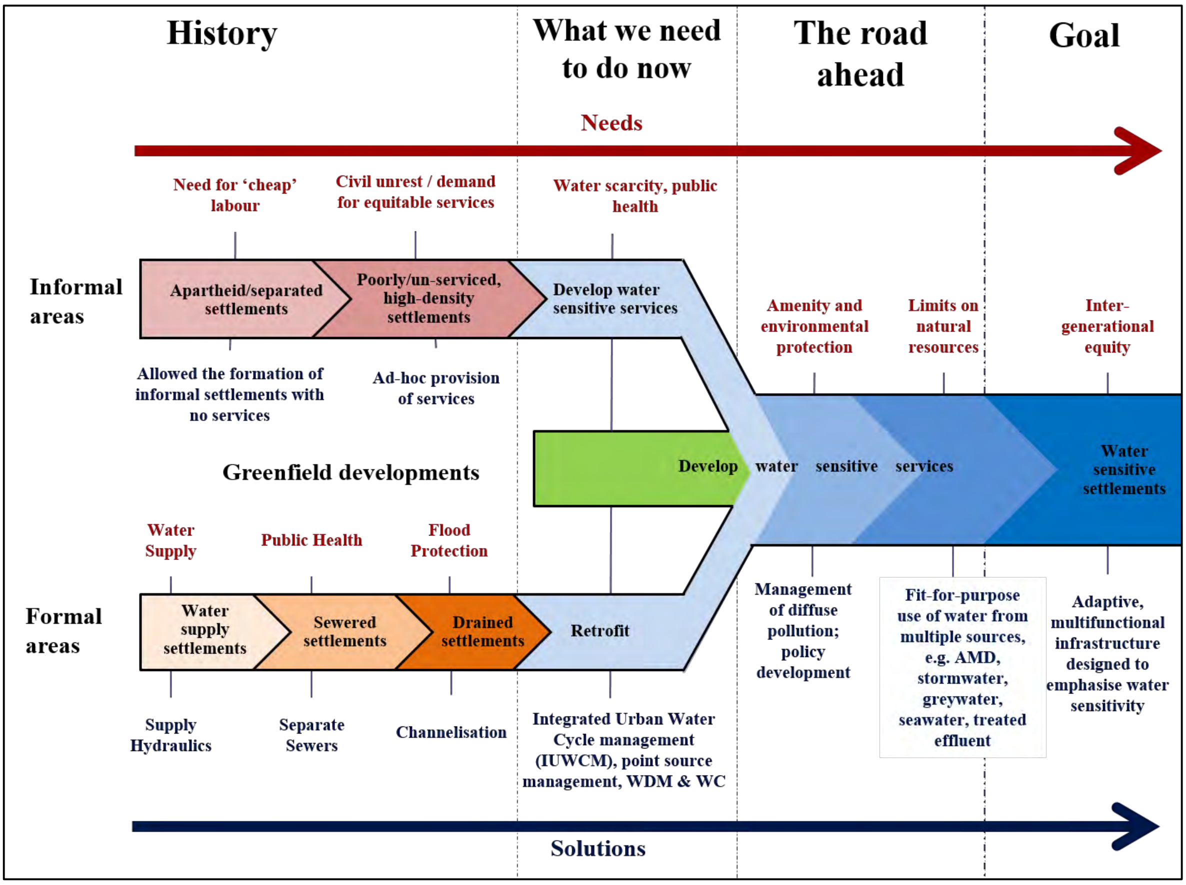


Figure B-3: The RSA's transition to Water-sensitive Settlements: *'Two histories, one future'* (Armitage *et al.*, 2014)

B5.3 Implementation component

The various aspects required to transition to WSS, are as follows: policy development, the establishment of appropriate institutional structures, community support, construction of appropriate infrastructure and ongoing and planned operation and maintenance. The most important consideration in the RSA is how to transition in the context of limited resources – both human and financial. It would be unreasonable to expect a municipality with limited funding and capacity to retrofit all of its urban water systems. Using the analogy of Maslow's Hierarchy of Needs (Maslow, 1943), municipalities need to ensure that they are at least meeting the physical water needs of their residents whilst attempting to provide services that help transition to the ultimate goal, Water Sensitive Settlements. A municipality cannot be expected to focus on establishing ecosystem sustainability and intergenerational equity unless it can simultaneously provide adequate and safe water to its citizens. Where it is not possible to incorporate the principles of water sensitivity (for example, the emergency provision of water services), municipalities should at least target their initiatives with the underlying philosophy of: 'Do what you can with what you have'. Municipalities can, therefore, begin by strengthening local legislation and regulations to encourage this transition.

B5.4 Narrative component

Narratives '*... simplify and offer a stable vision and interpretation of reality and are able to rally diverse people around particular story lines* (Molle, 2008)'. The Framework proposes a narrative for WSS in the RSA. The narrative expresses why a WSS is needed, how it can be implemented and what the desired outcome should be in the RSA. The WSS narrative for the RSA was developed to tie together the other components of the framework, so at the very least, all stakeholders could understand and engage with the idea of a WSS. The narrative is:

Why?

'...mitigating water scarcity, improving water quality, thereby protecting ecosystems, through the development of water sensitive urban areas (for all) that are sustainable, resilient and adaptable to change, while simultaneously being a place where people want to live…'

Result!

How?

Appendix C : Goonrey's Framework

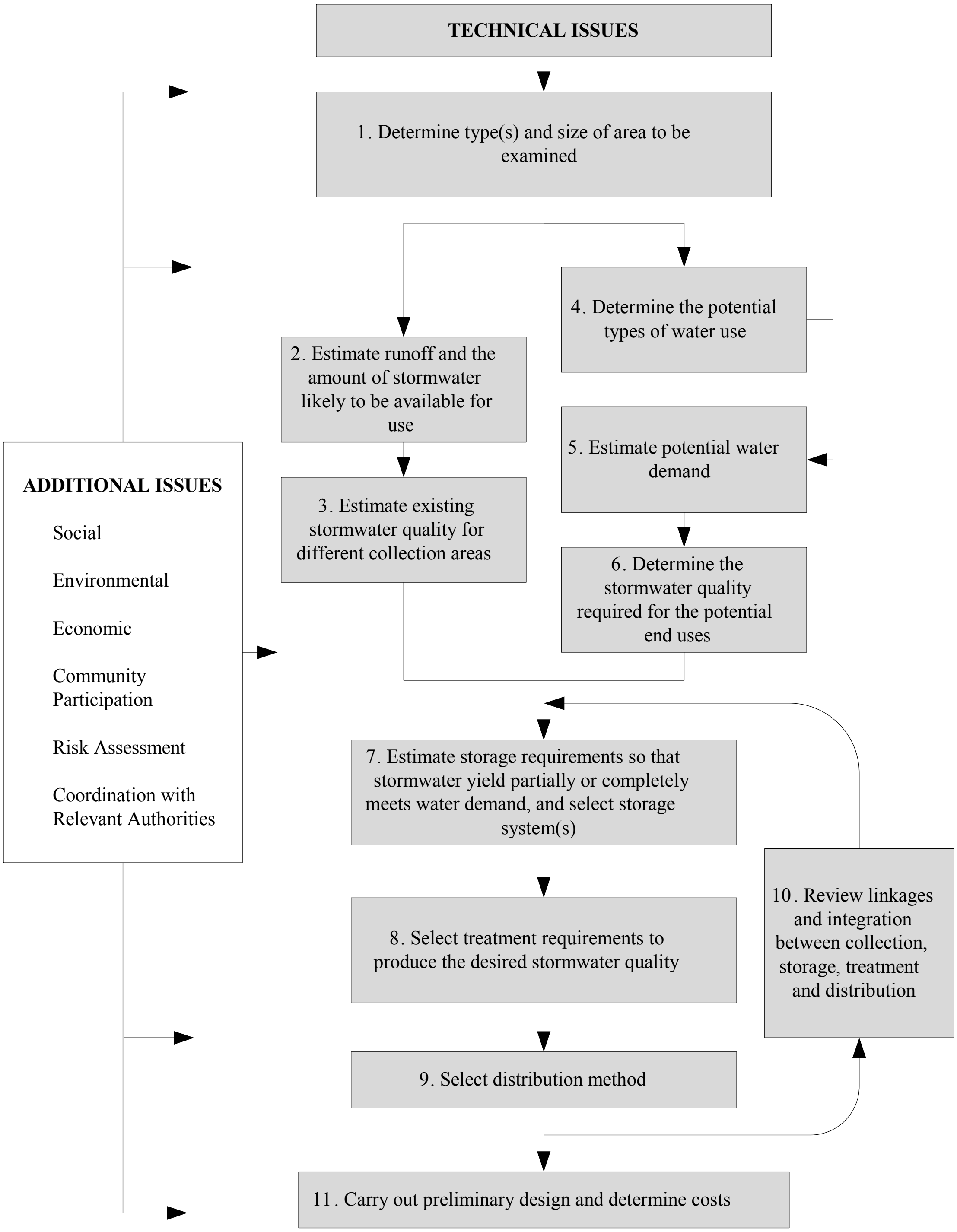


Figure C-1: Goonrey's Framework for assessing the potential for stormwater harvesting

Appendix D : Liesbeek River Catchment (2002-2014)

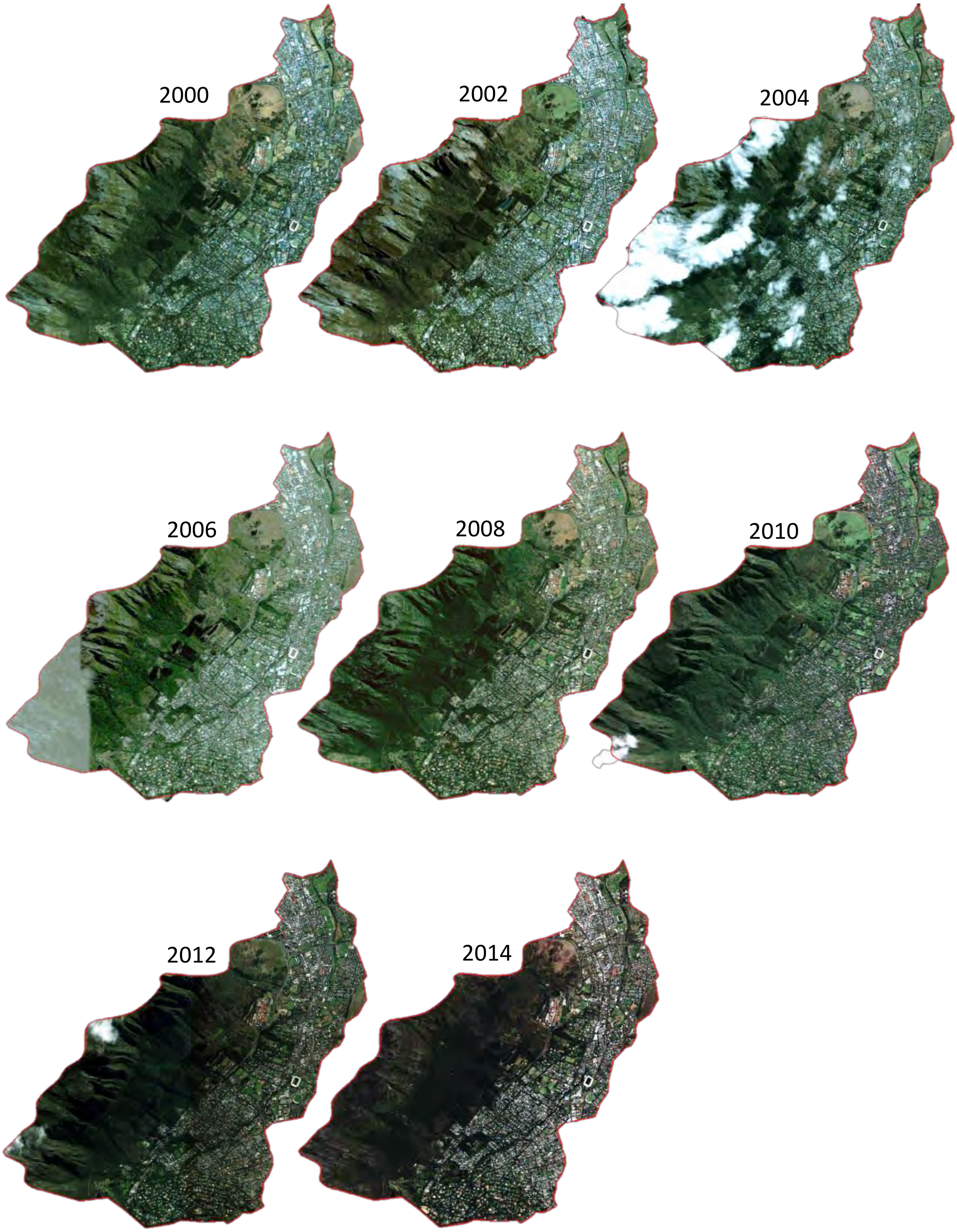


Figure D-1: Changes in the Liesbeek River Catchment 2000 - 2014

Appendix E: Rainfall disaggregation parameters

E.1 Historic Rainfall statistics

Table E-1: Historic rainfall statistics at Kirstenbosch

Event Duration		1 hour	3 hours	6 hours	12 hours	24 hours
January	mean	0.04	0.11	0.22	0.44	0.88
	variance	0.10	0.54	1.53	3.34	7.68
	autocovariance	0.04	0.14	0.13	0.32	0.22
	proportion dry	0.96	0.93	0.90	0.84	0.78
February	mean	0.04	0.13	0.26	0.51	1.02
	variance	0.14	0.75	1.85	3.77	8.67
	autocovariance	0.06	0.21	0.29	0.40	0.34
	proportion dry	0.97	0.95	0.92	0.87	0.81
March	mean	0.05	0.15	0.30	0.60	1.20
	variance	0.24	1.19	3.07	8.58	22.09
	autocovariance	0.12	0.58	1.21	1.48	0.99
	proportion dry	0.96	0.93	0.90	0.85	0.81
April	mean	0.19	0.57	1.14	2.29	4.58
	variance	1.79	12.89	35.35	85.85	234.71
	autocovariance	1.25	4.07	7.75	25.83	83.49
	proportion dry	0.92	0.88	0.85	0.78	0.70
May	mean	0.29	0.88	1.76	3.50	6.97
	variance	1.76	10.67	30.15	84.24	201.45
	autocovariance	1.02	4.66	10.35	17.28	37.02
	proportion dry	0.85	0.78	0.69	0.61	0.47
June	mean	0.28	0.83	1.66	3.32	6.65
	variance	1.77	10.46	29.82	81.23	212.19
	autocovariance	0.99	5.32	11.33	14.81	11.51
	proportion dry	0.86	0.77	0.70	0.62	0.50
July	mean	0.23	0.69	1.38	2.77	5.54
	variance	1.34	8.28	23.10	57.51	179.94
	autocovariance	0.80	4.10	9.46	21.71	17.85
	proportion dry	0.89	0.83	0.77	0.71	0.63
August	mean	0.27	0.81	1.62	3.24	6.48
	variance	1.39	8.47	24.49	55.97	142.79
	autocovariance	0.80	3.15	6.79	15.40	5.52

Event Duration		1 hour	3 hours	6 hours	12 hours	24 hours
	proportion dry	0.86	0.78	0.71	0.66	0.57
September	mean	0.19	0.57	1.14	2.28	4.56
	variance	0.76	4.69	13.15	31.38	75.87
	autocovariance	0.43	1.96	3.33	4.75	-0.35
	proportion dry	0.89	0.81	0.74	0.65	0.54
October	mean	0.09	0.26	0.52	1.04	2.09
	variance	0.27	1.48	4.48	11.69	24.93
	autocovariance	0.16	0.69	1.38	2.28	3.86
	proportion dry	0.93	0.88	0.81	0.73	0.66
November	mean	0.11	0.34	0.67	1.34	2.69
	variance	0.52	2.74	6.07	17.49	41.10
	autocovariance	0.28	0.75	2.33	3.92	3.40
	proportion dry	0.93	0.88	0.83	0.75	0.67
December	mean	0.04	0.13	0.26	0.52	1.05
	variance	0.16	0.86	1.92	4.79	12.36
	autocovariance	0.08	0.22	0.53	1.37	1.66
	proportion dry	0.96	0.93	0.90	0.85	0.80

E.2 R Script for estimating Bartlett-Lewis Input parameters for Hyetos

```
“# Bartlett-Lewis model equations
meanMBLRPM<-function(a,l,v,k,f,mx,h=1)
{
  x<-(h*l*mx*v*(1+k/f))/(a-1)
  return(x)
}
varMBLRPM<-function(a,l,v,k,f,mx,h=1) {
  A<-(2*l*(1+k/f)*(mx^2)*(v^a))/((f^2)*((f^2)-1)*(a-1)*(a-2)*(a-3))
  B<-(2*(f^2)-2+k*f)*(f^2)*((a-3)*h*(v^(2-a))-(v^(3-a))+((v+h)^(3-a)))
  C<-k*(f*(a-3)*h*(v^(2-a))-(v^(3-a))+((v+f*h)^(3-a)))
  D<-A*(B-C)
  return(D)
}
```

```
covarMBLRPM<-function(a,l,v,k,f,mx,h=1,lag=1) {
  A<-(l*(1+k/f)*(mx^2)*(v^a))/((f^2)*((f^2)-1)*(a-1)*(a-2)*(a-3))
  B<-(2*(f^2)-2+k*f)*(f^2)*(((v+(lag+1)*h)^(3-a))-2*((v+lag*h)^(3-a))+((v+(lag-1)*h)^(3-a)))
  C<-k*(((v+(lag+1)*h*f)^(3-a))-(2*((v+h*lag*f)^(3-a)))+((v+(lag-1)*h*f)^(3-a)))
  D<-A*(B-C)
  return(D)
}

pdrMBLRPM<-function(a,l,v,k,f,h=1) {
  mt<-((1+(f*(k+f))-(0.25*f*(k+f)*(k+4*f))+((f/72)*(k+f)*(4*(k^2)+27*k*f+72*(f^2))))*v)/(f*(a-1))
  G00<-((1-k-f+1.5*k*f+(f^2)+0.5*(k^2))*v)/(f*(a-1))
  A<-(f+(k*(v/(v+(k+f)*h))^(a-1)))/(f+k)
  D<-exp(l*(-h-mt+G00*A))
  return(D)
}

# Historical statistics
mean1 =  ;var1 = ;cov1lag1 = ;pdr1 =
mean3 = ;var3 = ;cov3lag1 = ;pdr3 =
mean6 = ;var6 = ;cov6lag1 = ;pdr6 =
mean12 = ;var12 = ;cov12lag1 = ;pdr12 =
mean24 = ;var24 = ;cov24lag1 = ;pdr24 =

# Objective function to be minimized

objfun <- function(x) {

a<-x[1];l<-x[2];v<-x[3];k<-x[4];f<-x[5];mx<-x[6]
w1=1;w2=1;w3=1;w4=1
```

```
S1 <- w1*((meanMBLRPM(a,l,v,k,f,mx,h=1)/mean1)-1)^(2)+w2*((varMBLRPM(a,l,v,k,f,mx,h=1)/var1)-1)^(2)+w3*((covarMBLRPM(a,l,v,k,f,mx,h=1,lag=1)/cov1lag1)-1)^(2)+w4*((pdrMBLRPM(a,l,v,k,f,h=1)/pdr1)-1)^(2)

S6 <- w1*((meanMBLRPM(a,l,v,k,f,mx,h=6)/mean6)-1)^(2)+w2*((varMBLRPM(a,l,v,k,f,mx,h=6)/var6)-1)^(2)+w3*((covarMBLRPM(a,l,v,k,f,mx,h=6,lag=1)/cov6lag1)-1)^(2)+w4*((pdrMBLRPM(a,l,v,k,f,h=6)/pdr6)-1)^(2)

S12 <- w1*((meanMBLRPM(a,l,v,k,f,mx,h=12)/mean12)-1)^(2)+w2*((varMBLRPM(a,l,v,k,f,mx,h=12)/var12)-1)^(2)+w3*((covarMBLRPM(a,l,v,k,f,mx,h=12,lag=1)/cov12lag1)-1)^(2)+w4*((pdrMBLRPM(a,l,v,k,f,h=12)/pdr12)-1)^(2)

S24 <- w1*((meanMBLRPM(a,l,v,k,f,mx,h=24)/mean24)-1)^(2)+w2*((varMBLRPM(a,l,v,k,f,mx,h=24)/var24)-1)^(2)+w3*((covarMBLRPM(a,l,v,k,f,mx,h=24,lag=1)/cov24lag1)-1)^(2)+w4*((pdrMBLRPM(a,l,v,k,f,h=24)/pdr24)-1)^(2)

S<-S1+S6+S12+S24

if(is.infinite(S)) {S<-10^8}

if(is.na(S)) {S<-10^8}

return(S)

}

# Parameter bounds

xmin <- c(1.0001,0.001,0.001,0.001,0.001,0.001)

xmax <- c(15,0.1,20,20,1,50)

xlow <- c(1.0001,0.001,0.001,0.001,0.001,0.001)

xup <- c(15,0.1,20,20,1,50)

# Model calibration

par <- eas(n=6,m=100,xmin,xmax,xlow,xup,fn=objfun,maxeval=15000,ftol=1.e-07,ratio=0.99,pmut=0.9,beta=5,maxclimbs=5)

print(par)”
```

E.3 Hyetos Input parameters and sample results

Table E-2: Hyetos input parameters for disaggregating Kirstenbosch rainfall station

	lamda λ d-1	kapa κ = β/η (-)	phi φ = γ/η (-)	alpha α (-)	ni ν d	mi_X μX mm d-1	Θi_X μX mm d-2
January	0.568	2.178	0.906	12.016	0.833	45.251	45.251
February	0.261	1.190	0.140	12.260	0.115	39.906	39.906
March	0.145	3.721	0.113	14.967	0.148	23.412	23.412
April	0.505	6.510	0.077	1.958	0.002	47.969	47.969
May	0.486	0.190	0.088	10.646	0.601	73.554	73.554
June	0.455	8.444	0.146	15.000	0.218	16.146	16.146
July	0.383	2.228	0.114	8.720	0.144	38.933	38.933
August	0.602	5.818	0.116	14.822	0.133	21.495	21.495
September	0.714	6.488	0.121	4.142	0.023	15.741	15.741
October	0.800	0.001	0.523	3.329	0.143	41.238	41.238
November	0.349	0.724	0.110	13.927	0.226	58.242	58.242
December	0.318	0.250	0.238	3.916	0.149	32.318	32.318

E.4 MudRAIN Input parameters

Table E-3: Marginal statistics (November)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
Power	1	0	1	0	1	1	1	1	1	1	1	0
Repititions	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000
Distance	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
0 Adjust	0.2	0.2	1	1	0.8	0.9	0.8	0.9	0.9	0.3	0.3	0.5
Probability	0.4	0.3	0.6	0.6	0.275	0.7	0.4	0.6	0.4	0.6	0.5	0.5

Appendix F: Liesbeek River - water quality

Table F-1: Water quality analysis in the Liesbeek River catchment

	Target Guideline	**Target value**	**NR23 (1988 – 2003)**	**NR22 (1988 – 2013)**	**NR12 (1975 – 2011)**	**NR08 (1975 – 2013)**	**NR13 (1975 – 2003)**
TSS	SAWQG – Domestic (DWAF, 1996c)	0	100%	100%	100%	100%	100%
	Based on 10% over 90 percentile at NR23 (PDNA, 2011)	20	9%	11%	12%	18%	46%
	SAWQG – Agricultural: irrigation (DWAF, 1996b)	50	3%	3%	5%	6%	15%
TP	Based on 10% over 90 percentile at NR23 (PDNA, 2011)	0.21	8%	7%	6%	9%	18%
Nitrate and Nitrite	SAWQG – Domestic (DWAF, 1996c)	6	1%	1%	0%	0%	1%
Ammonia	SAWQG – Domestic (DWAF, 1996c)	2	1%	1%	1%	1%	1%
	Ecological Reserve (PDNA, 2011)	0.1	43%	40%	55%	89%	73%
Total Inorganic Nitrogen	Ecological Reserve (PDNA, 2011)	4	1%	89%	1%	1%	98%
Conductivity	Based on 15% over 90 percentile NR23 (PDNA, 2011)	25	8%	10%	5%	49%	74%
Dissolved Oxygen	Ecological Reserve (PDNA, 2011)	4	1%	0%	0%	12%	26%
E.coli	SAWQG - Recreational Use (DWAF, 1996d)	130	56%	79%	100%	87%	58%
Faecal coliforms	SAWQG - Recreational Use (DWAF, 1996d)	130	62%	84%	99%	94%	72%
Orthophosphates	Ecological Reserve (PDNA, 2011)	0.125	4%	3%	3%	6%	5%
PH	Based on 10% over 90 percentile at NR23 (PDNA, 2011)	6.5-8.5	11%	7%	7%	9%	12%
Temperature	Based on +4C over 90 percentile at NR23 (PDNA, 2011)	25	1%	1%	1%	2%	1%

Appendix G: 1D Modelling Parameters

Table G-1: Overview of calibrated subcatchment parameters

Parameter	10 Percentile	Average	90 Percentile
Area (ha)	2.08	15.32	28.15
Width (m)	65.28	201.18	331.54
Flow Length (m)	213.87	627.62	1168.51
Slope (%)	5.99	13.71	24.25
Imperv (%)	13.00	41.85	66.44
N Imperv	0.02	0.02	0.03
N Perv	0.26	0.35	0.47
Dstore Imperv (mm)	2.70	3.27	3.70
Dstore Perv (mm)	5.40	6.58	9.41
Zero Imperv (%)	19.14	19.14	19.14
Percent Routed (%)	24.68	27.16	41.13
Suction Head (mm)	104.50	154.32	210.06
Conductivity (mm/hr)	1.45	11.73	29.18
Initial Deficit (frac.)	0.08	0.13	0.19

Table G-2: Overview of Conduit input

Parameter	Min	Average	Max
Roughness (manning's coefficient)	0.01	0.014391	0.033
Geometry(m)	0.2	0.764758	3.2

Appendix H: 1D calibration and verification results

Table H-1: 1-D Calibrated model runoff and routing errors

#	Scenario	Runoff Continuity Error	Routing Continuity Error
1	Calibrated Model	< 0.04 %	< 0.4 %
2	Whole Catchment	< 0.04 %	< 0.4 %
3	Whole Catchment with RWH (Upper)	< 0.04 %	< 0.4 %
4	Whole Catchment with RWH (Lower	< 0.04 %	< 0.4 %
5	Urban Catchment	< 0.04 %	< 0.4 %
6	Urban Catchment with RWH (Upper)	< 0.04 %	< 0.4 %
7	Urban Catchment with RWH (Lower)	< 0.04 %	< 0.4 %
8	SWH (Scenario 21,23,25)	< 0.04 %	< 0.2 %
9	Combined RWH and SWH (Scenario 27 and 29)	< 0.035 %	< 0.18 %

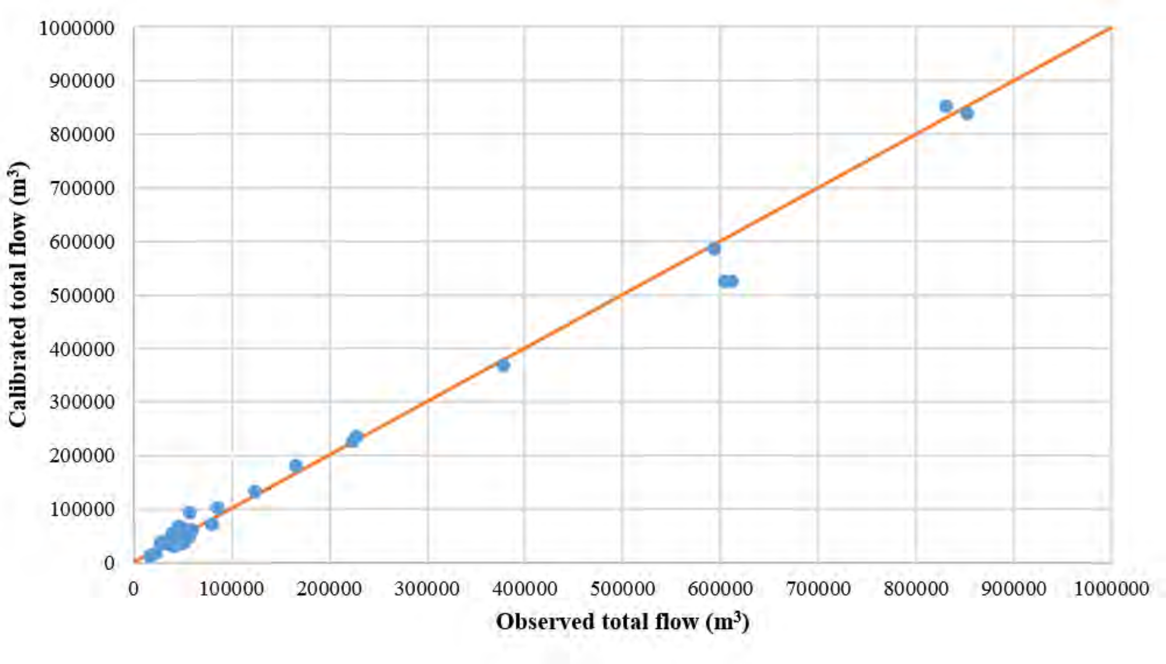


Figure H-1: Calibration of total flow volume

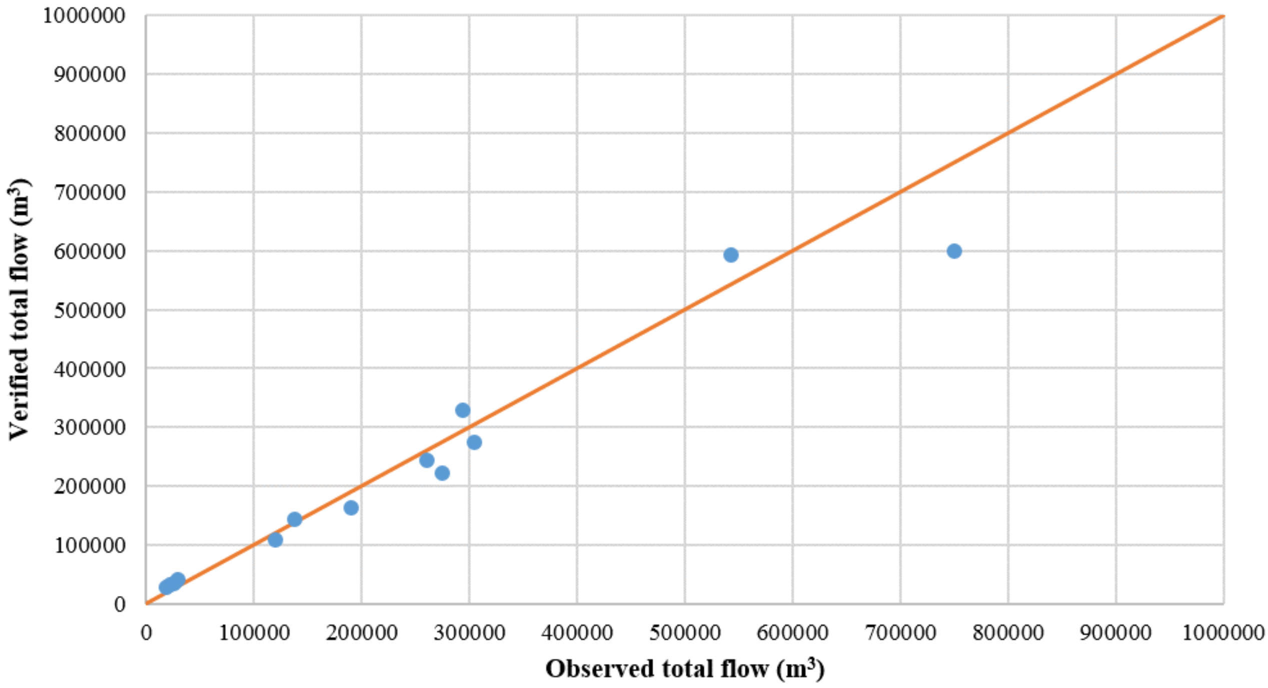


Figure H-2: Calibration of total flow volume

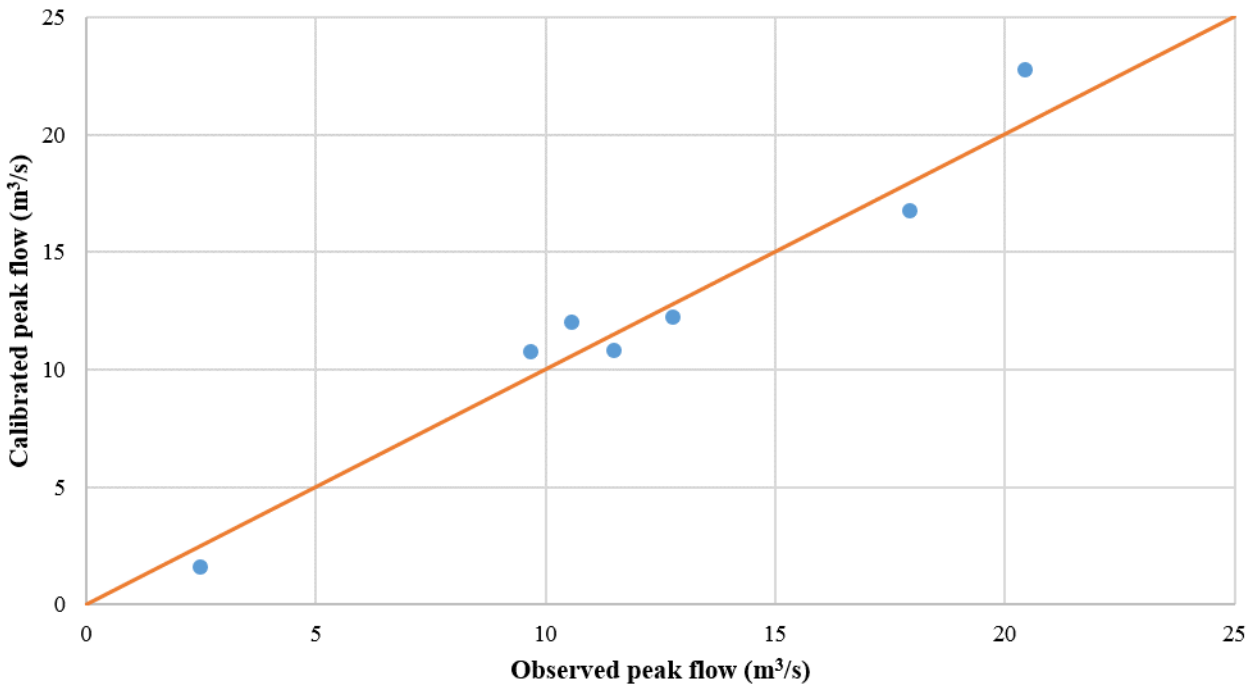


Figure H-4: Calibrated and observed peak flows for selected events

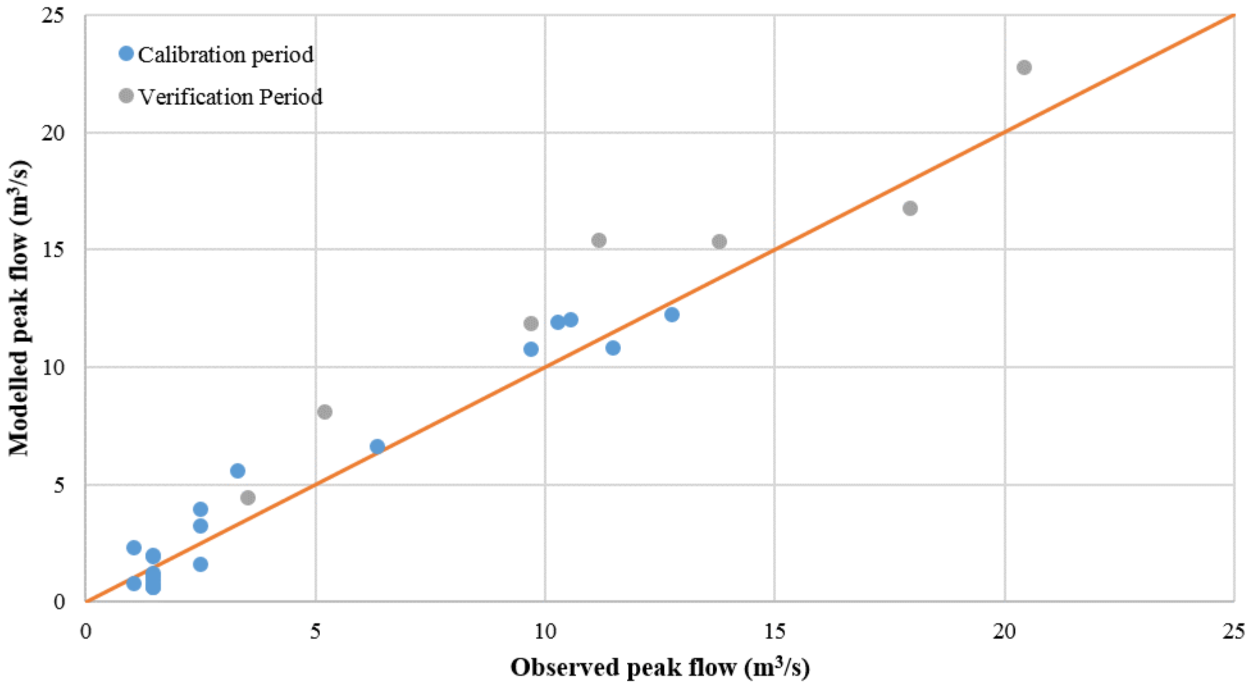


Figure H-3: Peak flows for all events during calibration and verification period

Appendix I: 2D modelling parameter

Table I-1: Basic 2D modeling parameters

Parameter	Input
Bounding layer	The 2D model above the 1D river conduit was modelled using Directional cells with a 5m resolution and a roughness of 0.015. The floodplain was modelled, based on landuse, using hexagonal cells with a resolution of between 5-8m and a roughness of between 0.011-0.04.
Mesh size	5-8 m.
2D Nodes	2D Nodes were generated by PCSWMM.
Elevation layer	The 2D nodes sampled elevation from the DEM developed for this research. The DEM was adjusted to ‘fill in the river’ which is represented in the model by a 1D conduit.
Obstruction layer	The obstruction layer was created by extracting all the roof areas on each property.
Centerline layer	The rivers center line was used.
Downstream layer	The boundary conditions assumed normal flow at the boundary of the site.
DEM layer	The 2D nodes sampled elevation from the DEM developed for this research. The DEM was adjusted to ‘fill in the river’ which is represented in the model by a 1D conduit.
Linking between 1D and 2D	The 1D model was linked to the 2D model using bottom orifices.

Appendix J: 2D Flooding calibration record

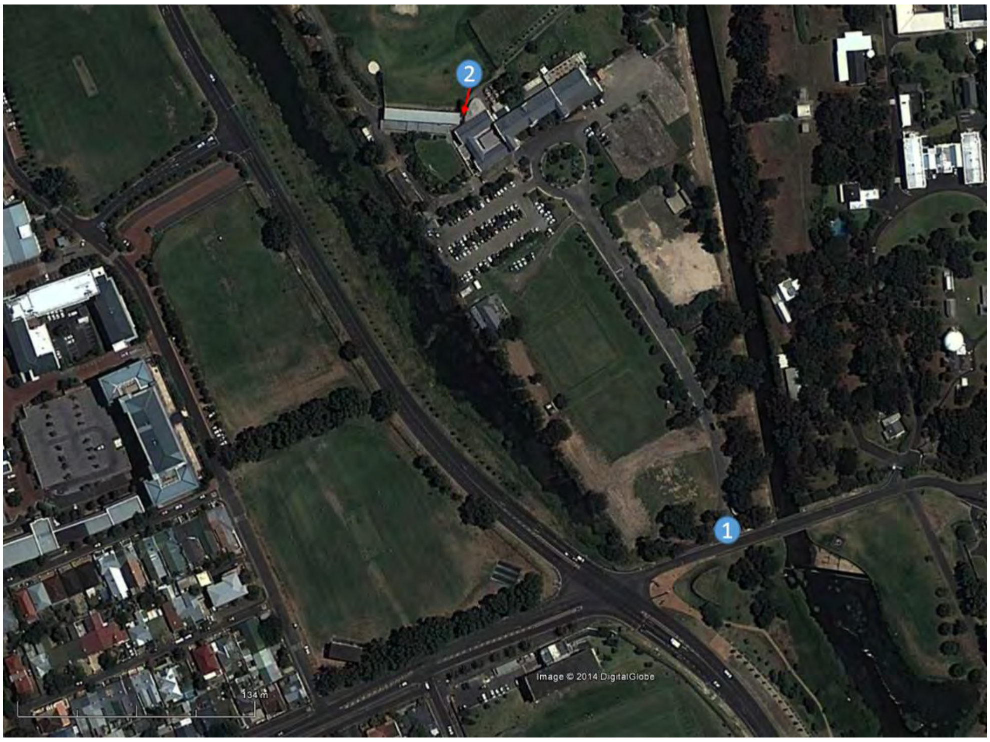


Figure J-1: Location of 2D flooding control points

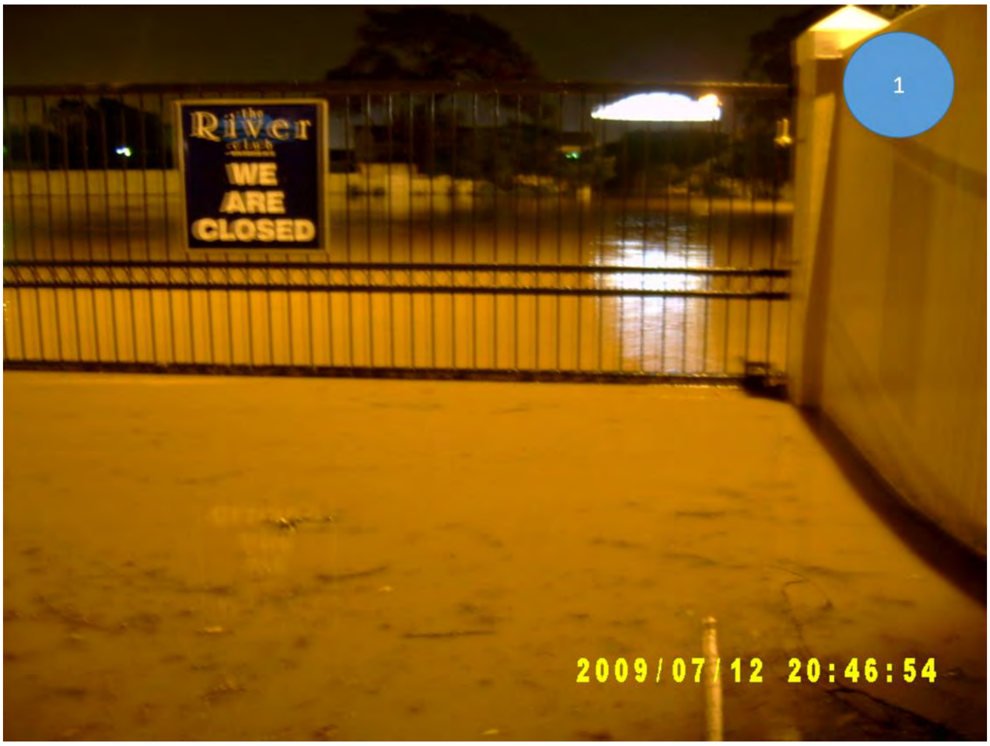


Figure J-2: 2D flooding control point 1

Figure J-3: 2D flooding control point 2

Table J-1: 2-D Calibrated model runoff and routing errors

	Scenario	Runoff Continuity Error	Routing Continuity Error
1	2D Flooding	< 0.02 %	< 3 %
2	2D Flooding with SWH	< 0.02 %	<3 %

Appendix K: Municipal water and electricity tariffs

Table K-1: City of Cape Town water tariffs 2013

Demand (kℓl)	**Total per kℓ**	**Water**	**Sewage**
From	**Incl. VAT**	**Incl. VAT**	**Incl. VAT**
0 – 6	R0.00	R0.00	R0.00
6 - 10.5	R14.41	R8.66	R8.21
10.5 - 20	R24.06	R13.24	R15.46
20 - 35	R31.44	R19.61	R16.90
35 - 50	R36.64	R24.22	R17.74
> 50	R44.37	R31.95	R17.74

Table K-1: City of Cape Town electricity tariffs 2013

Use	**Tariff Category**	**Tariff (including VAT)**
RWH	ZAR/KWH	1.425
	Connection fee (ZAR/day)	23.56
SWH	ZAR/KWH	1.2713

Appendix L: Ethics Clearance Forms

EBE Faculty: Assessment of Ethics in Research Projects (Rev2)

Any person planning to undertake research in the Faculty of Engineering and the Built Environment at the University of Cape Town is required to complete this form before collecting or analysing data. When completed it should be submitted to the supervisor (where applicable) and from there to the Head of Department. If any of the questions below have been answered YES, and the applicant is NOT a fourth year student, the Head should forward this form for approval by the Faculty EIR committee: submit to Ms Zulpha Geyer (Zulpha.Geyer@uct.ac.za; Chem Eng Building, Ph 021 650 4791).
NB: A copy of this signed form must be included with the thesis/dissertation/report when it is submitted for examination

This form must only be completed once the most recent revision EBE EIR Handbook has been read.

Name of Principal Researcher/Student: **Lloyd Fisher-Jeffes** Department: **Civil Engineering**

Preferred email address of the applicant: Lloyd.fisherjeffes@gmail.com

If a Student: Degree: **PhD** Supervisor: **A/Prof N Armitage**

If a Research Contract indicate source of funding/sponsorship:

Research Project Title:
Is Stormwater a viable resource in the Salt River catchment

Overview of ethics issues in your research project:

Question 1: Is there a possibility that your research could cause harm to a third party (i.e. a person not involved in your project)?	YES **NO**
Question 2: Is your research making use of human subjects as sources of data? If your answer is YES, please complete Addendum 2.	**YES** NO
Question 3: Does your research involve the participation of or provision of services to communities? If your answer is YES, please complete Addendum 3.	YES **NO**
Question 4: If your research is sponsored, is there any potential for conflicts of interest? If your answer is YES, please complete Addendum 4.	YES **NO**

If you have answered YES to any of the above questions, please append a copy of your research proposal, as well as any interview schedules or questionnaires (Addendum 1) and please complete further addenda as appropriate. Ensure that you refer to the EIR Handbook to assist you in completing the documentation requirements for this form.

I hereby undertake to carry out my research in such a way that

- there is no apparent legal objection to the nature or the method of research; and
- the research will not compromise staff or students or the other responsibilities of the University;
- the stated objective will be achieved, and the findings will have a high degree of validity;
- limitations and alternative interpretations will be considered;
- the findings could be subject to peer review and publicly available; and
- I will comply with the conventions of copyright and avoid any practice that would constitute plagiarism.

Signed by:

	Full name and signature	Date
Principal Researcher/Student: **Lloyd Fisher-Jeffes**		23/07/2012

This application is approved by:

Supervisor (if applicable): **A/Prof N Armitage**		24/7/2012
HOD (or delegated nominee): *Final authority for all assessments with NO to all questions and for all undergraduate research.*		3/10/2012
Chair: Faculty EIR Committee: For applicants other than undergraduate students who have answered YES to any of the above questions.		16/10/2012

LETTER OF UNDERTAKING

with respect to the use of

MONTLHY WATER CONSUMPTION DATA PROVIDED BY THE CITY OF CAPE TOWN

I Lloyd Fisher-Jeffes undertake, for the information of water consumption of the City of Cape Town, as provided to me:

1. To use the data provided for research purposes only (PhD. Water Research Commission projects);
2. not to distribute to any other party, or publish the information;
3. to provide the City's Water and Sanitation Department access to any paper/report/thesis prior to publication;
4. to acknowledge the City's Water and Sanitation Department in any publication;
5. to clearly state that the City/ City's Water and Sanitation Department cannot be held responsible for any processing of data, conclusions reached, or consequences arising from the use of the information;
6. not to disclose the identity, address or consumption of a specific consumer; and
7. to provide a copy to the City of any final publications related to the data (electronic version preferred).

Date: 16/10/2013

Signed:_ Signed by candidate

South African Weather Service

ISO 9001:2008 certified

DISCLOSURE STATEMENT

The provision of the data is subject to the User providing the South African Weather Service (SAWS) with a detailed and complete disclosure, in writing and in line with the requirements of clauses 1.1 to 2.4 (below), of the purpose for which the specified data is to be used. The statement is to be attached to this document as Schedule 1.

1 Should the User intend using the specified data for commercial gain then the disclosure should include the following:

1.1 the commercial nature of the project/funded research project in connection with which the User intends to use the specified data;

1.2 the names and fields of expertise of any participants in the project/funded research project for which the specified data is intended; and

1.3 the projected commercial gains to the User as a result of the intended use of the specified data for the project/funded research project.

2 Should the User intend using the specified data for the purposes of conducting research, then the disclosure should include the following:

2.1 the title of the research paper or project for which the specified data is to be used;

2.2 the details of the institution and supervisory body or person(s) under the auspices of which the research is to be undertaken;

2.3 an undertaking to supply SAWS with a copy of the final results of the research in printed and/or electronic format; and

2.4 the assurance that no commercial gain will be received from the outcome from the research.

If the specified data is used in research with disclosure being provided in accordance with paragraph 2 and the User is given the opportunity to receive financial benefit from the research following the publication of the results, then additional disclosure in terms of paragraph 1 is required.

The condition of this disclosure statement is applicable to the purpose and data requirements of the transaction recorded in Schedule 1. This statement is effective from 24 May 2012.

Bolepi House, 442 Rigel Avenue South, Erasmusrand, 0181 Private Bag X097, Pretoria, 0001 Tel: + 27 (0) 12 367 6000
Weatherline: 082 162 or 083 123 0500 www.weathersa.co.za sms line: [illegible]

CORP-LETT-EXT-004

Appendix M: List of climate change models

Table M-1a: Climate change models evaluated in this thesis

No.	Model	Representative Concentration Pathway	Modelling Institution
1	bcc-csm1-1	45	Beijing Climate Center, China Meteorological Administration
2		85	
3	BNU-ESM	45	College of Global Change and Earth System Science, Beijing Normal University
4		85	
5	CanESM2	45	Canadian Centre for Climate Modelling and Analysis
6		85	
7	CMCC-CES	85	Centro Euro-Mediterraneo per I Cambiamenti Climatici
8	CNRM-CM5	45	
9		85	
10	GFDL-ESM2G	45	Geophysical Fluid Dynamics Laboratory
11		85	
12	GFDL-ESM2M	45	
13		85	
14	HadGEM2-CC	45	National Institute of Meteorological Research/Korea Meteorological Administration
15		85	
16	inmcm4	45	Institute for Numerical Mathematics
17		85	
18	IPSL-CM5A-MR	45	Institut Pierre-Simon Laplace
19		85	
20	IPSL-CM5B-LR	45	
21		85	
22	MIROC5	45	Atmosphere and Ocean Research Institute (The University of Tokyo), National Institute for Environmental Studies, and Japan Agency for Marine-Earth Science and Technology
23		85	
24	MIROC-ESM-CHEM	45	Japan Agency for Marine-Earth Science and Technology, Atmosphere and Ocean Research Institute (The University of Tokyo), and National Institute for Environmental Studies
25		85	
26	MIROC-ESM	45	
27		85	

Table M-1b: Climate change models evaluated in this thesis (Continued)

No.	Model	Representative Concentration Pathway	Modelling Institution
28	MPI-ESM-LR	45	Max Planck Institute for Meteorology (MPI-M)
29		85	
30	MRI-CGCM3	45	Meteorological Research Institute
31		85	

* Note 1: The ERA-interim (part of the ECMWF re-analysis project) for the period 1979-2012 was used as equivalent to the historical data record.

Appendix N : Climate change – modelled changes in climate at Kirstenbosch and Observatory weather stations

Table N-1a: Average modelled evaporation at Kirstenbosch weather station (mm/day)

	Average Evaporation 1979-2012 (mm/day)	Average modelled Evaporation 2050-2099 (mm/day)														
RCP		**45**	**85**	**45**	**85**	**45**	**85**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**
Month		**bcc-csm1-1-rcp45**	**bcc-csm1-1-rcp85**	**BNU-ESM-rcp45**	**BNU-ESM-rcp85**	**CanESM2-rcp45**	**CanESM2-rcp85**	**CMCC-CESM rcp85**	**CNRM-CM5-rcp45**	**CNRM-CM5-rcp85**	**GFDL-ESM2G-rcp45**	**GFDL-ESM2G-rcp85**	**GFDL-ESM2M-rcp45**	**GFDL-ESM2M-rcp85**	**HadGEM2-CC-rcp45**	**HadGEM2-CC-rcp85**
Jan	4.88	5.02	5.18	5.07	5.26	5.14	5.34	5.24	5.16	5.28	4.99	5.12	4.94	5.06	4.96	5.55
Feb	4.49	4.70	4.85	4.68	4.93	4.74	4.89	4.88	4.82	4.98	4.62	4.80	4.62	4.70	4.61	5.13
Mar	3.69	3.91	4.06	3.94	4.12	3.98	4.10	4.13	4.01	4.07	3.84	3.99	3.87	3.99	3.86	4.32
Apr	2.65	2.90	3.02	2.92	3.05	2.90	3.03	3.01	2.90	3.02	2.76	2.91	2.85	2.90	2.85	3.20
May	1.78	1.91	1.99	1.96	2.07	1.92	2.01	1.96	1.93	1.98	1.76	1.85	1.84	1.88	1.95	2.09
Jun	1.42	1.50	1.57	1.54	1.60	1.51	1.56	1.56	1.49	1.55	1.41	1.45	1.43	1.48	1.57	1.65
Jul	1.53	1.58	1.64	1.63	1.70	1.59	1.65	1.67	1.60	1.64	1.51	1.55	1.55	1.57	1.65	1.73
Aug	1.98	2.08	2.13	2.10	2.17	2.06	2.13	2.16	2.04	2.09	1.94	2.02	1.98	2.05	2.12	2.18
Sept	2.66	2.75	2.84	2.82	2.94	2.78	2.85	2.90	2.71	2.79	2.65	2.70	2.67	2.76	2.85	2.91
Oct	3.56	3.52	3.66	3.64	3.80	3.68	3.79	3.79	3.52	3.68	3.49	3.64	3.49	3.64	3.73	3.88
Nov	4.30	4.37	4.54	4.43	4.69	4.61	4.76	4.69	4.45	4.57	4.33	4.56	4.38	4.53	4.54	4.93
Dec	4.90	5.00	5.21	5.05	5.29	5.21	5.38	5.25	5.04	5.14	4.88	5.07	4.88	5.05	4.94	5.50

Table N-1b: Average modelled evaporation at Kirstenbosch weather station (mm/day) (Continued)

	Average Evaporation 1979-2012 (mm/day)	**Average modelled Evaporation 2050-2099 (mm/day)**														
RCP		**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**
Month		**inmcm4-rcp85**	**IPSL-CM5A-MR-rcp45**	**IPSL-CM5A-MR-rcp85**	**IPSL-CM5B-LR-rcp45**	**IPSL-CM5B-LR-rcp85**	**MIROC5-rcp45**	**MIROC5-rcp85**	**MIROC-ESM-CHEM-rcp45**	**MIROC-ESM-CHEM-rcp85**	**MIROC-ESM-rcp45**	**MIROC-ESM-rcp85**	**MPI-ESM-LR-rcp45**	**MPI-ESM-LR-rcp85**	**MRI-CGCM3-rcp45**	**MRI-CGCM3-rcp85**
Jan	4.88	5.20	5.12	5.49	5.49	5.02	5.16	5.30	4.98	5.21	4.97	5.25	5.00	5.23	5.03	5.17
Feb	4.49	4.78	4.77	5.12	5.12	4.77	4.77	4.87	4.61	4.80	4.57	4.80	4.70	4.90	4.70	4.84
Mar	3.69	3.95	4.02	4.25	4.25	4.05	3.92	4.06	3.87	4.07	3.85	4.10	3.92	4.02	3.92	4.05
Apr	2.65	2.91	2.92	3.13	3.13	3.00	2.87	2.97	2.88	3.01	2.86	3.03	2.80	2.95	2.90	3.00
May	1.78	2.03	1.97	2.10	2.10	2.00	1.92	2.00	1.97	2.07	1.96	2.05	1.86	1.95	1.95	1.99
Jun	1.42	1.60	1.54	1.63	1.63	1.55	1.53	1.56	1.57	1.63	1.56	1.62	1.50	1.57	1.53	1.55
Jul	1.53	1.66	1.63	1.70	1.70	1.64	1.57	1.62	1.61	1.69	1.62	1.69	1.62	1.67	1.61	1.66
Aug	1.98	2.13	2.13	2.21	2.21	2.12	2.01	2.07	2.04	2.12	2.08	2.11	2.07	2.14	2.06	2.14
Sept	2.66	2.91	2.83	2.99	2.99	2.82	2.71	2.80	2.70	2.81	2.72	2.79	2.72	2.89	2.76	2.87
Oct	3.56	3.80	3.64	3.88	3.88	3.56	3.66	3.72	3.53	3.73	3.56	3.67	3.58	3.82	3.51	3.63
Nov	4.30	4.66	4.50	4.79	4.79	4.36	4.65	4.78	4.41	4.71	4.46	4.66	4.44	4.69	4.38	4.51
Dec	4.90	5.15	5.12	5.41	5.41	4.93	5.10	5.28	4.99	5.21	4.97	5.20	4.92	5.20	4.90	5.12

Table N-2a: Average modelled rainfall at Kirstenbosch weather station (mm/month)

		Average modelled Rainfall 2050-2099 (mm/month)														
RCP	**Average Rainfall 1979-2012 (mm/day)**	**45**	**85**	**45**	**85**	**45**	**85**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**
Month		**bcc-csm1-1-rcp45**	**bcc-csm1-1-rcp85**	**BNU-ESM-rcp45**	**BNU-ESM-rcp85**	**CanESM2-rcp45**	**CanESM2-rcp85**	**CMCC-CESMrcp85**	**CNRM-CM5-rcp45**	**CNRM-CM5-rcp85**	**GFDL-ESM2G-rcp45**	**GFDL-ESM2G-rcp85**	**GFDL-ESM2M-rcp45**	**GFDL-ESM2M-rcp85**	**HadGEM2-CC-rcp45**	**HadGEM2-CC-rcp85**
Jan	34.25	38.03	32.23	32.32	31.44	25.13	19.31	19.37	20.02	20.68	16.03	17.99	20.38	22.28	30.02	25.31
Feb	18.44	20.86	15.83	23.28	21.83	19.22	16.70	14.49	18.15	18.93	14.57	10.63	17.13	12.77	29.35	16.13
Mar	34.37	25.31	19.53	24.32	24.45	22.82	22.59	22.38	26.33	25.86	18.97	17.77	22.17	16.83	29.75	27.13
Apr	68.11	60.74	52.74	52.45	46.40	72.47	58.23	71.49	67.13	56.43	72.30	50.19	74.14	51.89	82.33	76.43
May	183.65	167.62	164.00	167.31	146.91	159.00	148.71	193.37	145.43	159.44	195.47	195.56	182.93	177.86	169.01	161.91
Jun	193.69	204.78	207.20	213.02	216.48	211.32	212.69	241.64	194.46	198.73	244.56	237.28	245.04	241.21	213.68	201.10
Jul	198.18	223.13	217.91	227.59	234.30	231.62	210.08	245.96	227.81	232.38	242.31	252.19	240.59	248.55	189.66	206.56
Aug	212.04	207.34	208.34	230.04	225.69	223.15	216.88	216.77	229.17	227.66	255.08	260.89	273.10	250.12	207.29	215.94
Sept	173.67	190.82	192.03	166.80	179.31	180.32	205.45	184.38	219.35	213.02	232.19	228.69	232.17	207.53	166.92	184.72
Oct	92.03	155.25	151.65	123.42	136.32	140.35	139.05	136.53	165.47	143.26	164.44	148.10	168.74	154.90	127.61	140.58
Nov	67.70	81.60	76.55	82.32	58.36	62.46	65.23	64.49	81.57	75.85	64.44	48.44	62.25	54.89	62.13	62.37
Dec	34.93	43.95	29.94	45.26	43.45	30.76	29.05	24.63	34.35	32.85	28.95	26.04	38.39	27.20	39.10	33.86

Table N-2b: Average modelled rainfall at Kirstenbosch weather station (mm/month) (Continued)

	Average Rainfall 1979-2012 (mm/day)	Average modelled Rainfall 2050-2099 (mm/month)															
RCP		**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**
Month		**inmcm4-rcp45**	**inmcm4-rcp85**	**IPSL-CM5A-MR-rcp45**	**IPSL-CM5A-MR-rcp85**	**IPSL-CM5B-LR-rcp45**	**IPSL-CM5B-LR-rcp85**	**MIROC5-rcp45**	**MIROC5-rcp85**	**MIROC-ESM-CHEM-rcp45**	**MIROC-ESM-CHEM-rcp85**	**MIROC-ESM-rcp45**	**MIROC-ESM-rcp85**	**MPI-ESM-LR-rcp45**	**MPI-ESM-LR-rcp85**	**MRI-CGCM3-rcp45**	**MRI-CGCM3-rcp85**
Jan	34.25	16.06	15.93	24.34	17.87	42.99	38.56	16.31	14.72	32.87	29.95	29.70	30.33	27.42	17.74	22.70	26.08
Feb	18.44	14.21	15.73	12.77	11.23	29.34	20.26	13.87	17.04	29.32	24.30	27.19	19.31	20.67	13.88	21.39	20.06
Mar	34.37	27.34	21.82	23.91	18.38	24.45	23.48	30.06	20.23	30.72	20.11	34.38	25.54	36.18	28.25	22.21	21.81
Apr	68.11	55.42	54.17	56.87	43.64	53.61	52.50	60.78	51.98	67.35	58.73	75.21	64.05	80.15	68.86	54.34	53.88
May	183.65	123.64	109.15	139.24	134.37	157.04	156.05	151.37	155.88	167.40	153.22	171.86	146.24	188.64	171.95	146.38	145.63
Jun	193.69	161.79	155.72	195.00	193.02	200.04	202.00	197.14	184.20	187.15	178.20	190.21	185.40	214.27	211.97	197.48	188.51
Jul	198.18	176.42	177.62	222.11	217.72	203.17	219.31	238.57	225.79	224.93	225.45	219.07	223.42	235.08	246.36	229.82	236.39
Aug	212.04	178.33	183.97	221.47	200.20	178.82	211.06	237.36	254.64	235.33	214.16	202.17	208.45	230.80	217.31	216.41	196.78
Sept	173.67	142.53	137.02	186.81	176.15	200.35	186.08	222.94	226.84	210.36	199.11	190.13	194.34	199.03	186.75	174.53	170.49
Oct	92.03	111.46	95.13	152.00	124.39	188.89	179.77	145.51	146.68	160.96	156.86	146.86	152.17	153.72	121.18	147.01	147.21
Nov	67.70	47.99	47.84	76.41	62.50	116.70	101.24	49.58	45.04	93.11	87.98	101.17	91.46	91.51	70.89	78.36	93.14
Dec	34.93	26.38	28.71	43.10	32.87	62.09	68.37	35.70	28.91	51.66	47.02	51.67	38.50	52.73	29.19	42.87	36.30

Table N-3a: Average modelled evaporation at Observatory weather station (mm/day)

	Average Evaporation 1979-2012 (mm/day)	Average modelled evaporation 2050-2099 (mm/day)														
RCP		45	85	45	85	45	85	85	45	85	45	85	45	85	45	85
Month		bcc-csm1-1-rcp45	bcc-csm1-1-rcp85	BNU-ESM-rcp45	BNU-ESM-rcp85	CanESM2-rcp45	CanESM2-rcp85	CMCC-CESMrcp85	CNRM-CM5-rcp45	CNRM-CM5-rcp85	GFDL-ESM2G-rcp45	GFDL-ESM2G-rcp85	GFDL-ESM2M-rcp45	GFDL-ESM2M-rcp85	HadGEM2-CC-rcp45	HadGEM2-CC-rcp85
Jan	4.99	5.11	5.27	5.15	5.36	5.23	5.43	5.33	5.29	5.39	5.09	5.23	5.03	5.16	5.04	5.35
Feb	4.59	4.79	4.94	4.78	5.03	4.84	4.99	4.97	4.93	5.09	4.74	4.91	4.73	4.81	4.71	4.95
Mar	3.78	3.97	4.13	4.02	4.21	4.06	4.16	4.19	4.11	4.16	3.92	4.09	3.95	4.07	3.92	4.17
Apr	2.70	2.94	3.05	2.96	3.09	2.93	3.06	3.04	2.96	3.07	2.81	2.96	2.89	2.95	2.87	3.08
May	1.81	1.94	2.01	1.99	2.09	1.95	2.04	1.98	1.96	2.01	1.80	1.88	1.87	1.92	1.97	2.05
Jun	1.47	1.53	1.60	1.57	1.63	1.55	1.59	1.59	1.53	1.59	1.45	1.49	1.46	1.51	1.59	1.64
Jul	1.57	1.62	1.68	1.67	1.74	1.63	1.69	1.72	1.65	1.68	1.55	1.59	1.58	1.60	1.68	1.71
Aug	2.03	2.13	2.18	2.15	2.23	2.12	2.18	2.21	2.10	2.14	2.00	2.06	2.03	2.10	2.15	2.17
Sept	2.71	2.81	2.90	2.88	3.01	2.85	2.91	2.95	2.77	2.85	2.70	2.76	2.74	2.81	2.88	2.90
Oct	3.61	3.58	3.72	3.70	3.86	3.72	3.85	3.83	3.58	3.73	3.54	3.69	3.55	3.70	3.77	3.86
Nov	4.37	4.41	4.58	4.48	4.73	4.64	4.79	4.74	4.52	4.63	4.39	4.60	4.43	4.59	4.60	4.79
Dec	4.98	5.07	5.29	5.13	5.37	5.30	5.46	5.33	5.14	5.24	4.99	5.16	4.97	5.15	5.02	5.30

Table N-3b: Average modelled evaporation at Observatory weather station (mm/day) (Continued)

	Average Evaporation 1979-2012 (mm/day)	**Average modelled evaporation 2050-2099 (mm/day)**															
RCP		**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**
Month		**inmcm4-rcp45**	**inmcm4-rcp85**	**IPSL-CM5A-MR-rcp45**	**IPSL-CM5A-MR-rcp85**	**IPSL-CM5B-LR-rcp45**	**IPSL-CM5B-LR-rcp85**	**MIROC5-rcp45**	**MIROC5-rcp85**	**MIROC-ESM-CHEM-rcp45**	**MIROC-ESM-CHEM-rcp85**	**MIROC-ESM-rcp45**	**MIROC-ESM-rcp85**	**MPI-ESM-LR-rcp45**	**MPI-ESM-LR-rcp85**	**MRI-CGCM3-rcp45**	**MRI-CGCM3-rcp85**
Jan	4.99	5.17	5.17	5.23	5.58	4.96	5.10	5.28	5.41	5.08	5.31	5.07	5.25	5.11	5.33	5.14	5.27
Feb	4.59	4.78	4.78	4.87	5.23	4.66	4.85	4.88	4.97	4.71	4.90	4.67	4.87	4.80	5.00	4.79	4.93
Mar	3.78	3.90	3.90	4.09	4.34	4.01	4.13	4.00	4.14	3.96	4.15	3.92	4.06	4.01	4.10	4.00	4.13
Apr	2.70	2.84	2.84	2.97	3.16	2.98	3.05	2.91	3.01	2.92	3.06	2.90	3.00	2.85	2.99	2.94	3.04
May	1.81	1.99	1.99	2.01	2.13	1.97	2.03	1.95	2.03	2.00	2.09	1.98	2.09	1.90	1.98	1.98	2.01
Jun	1.47	1.59	1.59	1.58	1.67	1.54	1.58	1.56	1.59	1.60	1.67	1.60	1.67	1.54	1.61	1.56	1.58
Jul	1.57	1.66	1.66	1.67	1.75	1.65	1.68	1.61	1.66	1.65	1.74	1.66	1.74	1.67	1.71	1.65	1.70
Aug	2.03	2.14	2.14	2.18	2.28	2.14	2.17	2.06	2.12	2.10	2.19	2.14	2.21	2.12	2.19	2.11	2.18
Sept	2.71	2.89	2.89	2.88	3.05	2.82	2.88	2.76	2.85	2.77	2.89	2.80	2.92	2.79	2.94	2.83	2.91
Oct	3.61	3.73	3.73	3.69	3.93	3.52	3.61	3.70	3.75	3.61	3.78	3.63	3.78	3.65	3.87	3.57	3.69
Nov	4.37	4.56	4.56	4.54	4.83	4.33	4.41	4.72	4.84	4.47	4.76	4.50	4.65	4.52	4.76	4.46	4.58
Dec	4.98	5.17	5.17	5.18	5.46	4.87	5.02	5.18	5.37	5.07	5.30	5.03	5.26	5.02	5.29	4.98	5.21

Table N-4a: Average modelled rainfall at Observatory weather station (mm/month)

	Average Evaporation 1979-2012 (mm/day)	Average modelled Rainfall 2050-2099 (mm/month)														
RCP		**45**	**85**	**45**	**85**	**45**	**85**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**
Month		**bcc-csm1-1-rcp45**	**bcc-csm1-1-rcp85**	**BNU-ESM-rcp45**	**BNU-ESM-rcp85**	**CanESM2-rcp45**	**CanESM2-rcp85**	**CMCC-CESM rcp85**	**CNRM-CM5-rcp45**	**CNRM-CM5-rcp85**	**GFDL-ESM2G-rcp45**	**GFDL-ESM2G-rcp85**	**GFDL-ESM2M-rcp45**	**GFDL-ESM2M-rcp85**	**HadGEM2-CC-rcp45**	**HadGEM2-CC-rcp85**
Jan	18.55	18.06	17.18	16.82	17.26	13.76	9.67	11.74	9.72	11.39	7.44	10.51	12.74	11.55	13.84	12.90
Feb	12.02	11.30	7.41	12.18	12.38	11.80	8.03	7.20	9.13	7.04	7.38	5.88	7.20	6.94	13.48	7.72
Mar	15.88	11.93	8.72	12.88	12.35	9.75	10.17	9.05	10.64	10.92	8.27	7.24	8.39	8.04	15.00	12.94
Apr	35.39	25.10	23.06	22.53	20.53	28.51	22.82	32.22	30.10	25.14	32.26	21.90	33.11	21.96	32.72	35.45
May	78.14	68.68	67.74	74.65	65.79	69.81	62.43	78.49	63.22	68.87	82.64	82.49	78.93	79.76	66.15	68.93
Jun	88.40	84.77	89.37	85.73	84.64	88.83	90.11	98.31	87.54	88.57	103.05	102.99	103.38	96.16	84.27	85.37
Jul	86.45	97.62	94.02	94.93	98.68	102.7	92.42	109.19	98.86	104.52	103.01	103.74	102.19	107.03	81.86	91.82
Aug	96.73	87.78	96.94	101.76	101.11	102.6	97.56	96.57	100.68	97.41	109.28	117.56	123.45	107.59	87.69	101.68
Sept	78.69	86.69	80.55	74.39	79.53	82.02	91.33	77.76	95.67	91.64	97.74	97.90	99.63	90.97	73.92	88.69
Oct	44.95	70.48	64.88	57.22	65.52	61.19	60.10	61.67	67.93	61.34	68.57	59.22	74.07	62.38	57.39	65.49
Nov	38.13	37.63	35.60	39.33	28.91	25.76	29.25	30.64	36.94	36.83	32.83	22.14	33.05	26.16	30.60	29.71
Dec	18.06	20.58	17.11	22.59	20.49	14.27	15.26	12.79	16.00	18.47	15.30	14.23	19.86	14.24	19.77	17.63

Table N-4b: Average modelled rainfall at Observatory weather station (mm/month) (Continued)

	Average Evaporation 1979-2012 (mm/day)	Average modelled Rainfall 2050-2099 (mm/month)															
RCP		**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**
Month		**inmcm4-rcp45**	**inmcm4-rcp85**	**IPSL-CM5A-MR-rcp45**	**IPSL-CM5A-MR-rcp85**	**IPSL-CM5B-LR-rcp45**	**IPSL-CM5B-LR-rcp85**	**MIROC5-rcp45**	**MIROC5-rcp85**	**MIROC-ESM-CHEM-rcp45**	**MIROC-ESM-CHEM-rcp85**	**MIROC-ESM-rcp45**	**MIROC-ESM-rcp85**	**MPI-ESM-LR-rcp45**	**MPI-ESM-LR-rcp85**	**MRI-CGCM3-rcp45**	**MRI-CGCM3-rcp85**
Jan	18.55	6.38	6.75	13.58	9.71	21.07	18.90	7.91	7.48	16.98	16.59	16.38	15.40	14.49	9.19	13.57	14.08
Feb	12.02	6.53	8.15	8.61	5.92	14.40	11.17	7.16	8.26	14.56	11.60	12.53	8.92	9.52	6.15	10.01	10.46
Mar	15.88	9.87	9.65	11.00	8.20	10.69	10.92	12.19	9.34	15.76	9.66	18.58	13.64	17.39	14.05	10.23	11.74
Apr	35.39	25.71	24.72	24.00	19.26	22.78	23.89	26.01	24.39	28.95	28.36	36.58	28.91	37.57	31.32	26.24	25.85
May	78.14	51.76	51.77	62.78	58.71	67.03	64.46	66.35	67.54	68.35	65.37	70.54	58.15	80.78	70.69	60.57	60.71
Jun	88.40	76.24	74.56	87.44	88.00	87.31	82.51	83.53	78.08	83.98	74.23	82.04	75.87	89.92	92.87	84.08	82.69
Jul	86.45	79.00	82.13	101.09	101.04	88.77	90.99	101.97	96.73	102.86	103.2	98.14	102.95	105.28	108.74	103.23	103.96
Aug	96.73	85.14	87.14	101.63	93.28	83.98	93.67	104.47	110.57	99.52	98.91	89.72	96.05	99.17	96.64	94.96	82.09
Sept	78.69	64.49	64.71	84.17	79.07	88.96	83.10	96.04	98.13	96.99	91.79	83.33	90.50	90.92	82.98	76.88	73.93
Oct	44.95	50.14	46.79	68.83	56.06	84.39	74.43	63.35	61.30	69.17	68.76	69.63	63.70	69.58	56.11	63.64	65.63
Nov	38.13	20.67	22.55	34.09	27.83	48.42	47.47	20.73	20.36	41.15	41.33	45.04	39.56	44.54	34.55	37.15	43.53
Dec	18.06	13.07	15.46	18.16	14.76	27.09	33.87	16.05	11.99	26.21	24.56	27.12	18.86	23.92	14.54	21.86	17.28

Table N-5a: Percentage change in evaporation at Kirstenbosch weather station between ERA-interim (1979-2012) and climate change model (2050-2100)

Month	Percentage change in evaporation at Kirstenbosch weather station between ERA-interim (1979-2012) and climate change model (2050-2100)																
	45	**85**	**45**	**85**	**45**	**85**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**	**45**	**85**
	bcc-csm1-1-rcp45	**bcc-csm1-1-rcp85**	**BNU-ESM-rcp45**	**BNU-ESM-rcp85**	**CanESM2-rcp45**	**CanESM2-rcp85**	**CMCC-CESMrcp85**	**CNRM-CM5-rcp45**	**CNRM-CM5-rcp85**	**GFDL-ESM2G-rcp45**	**GFDL-ESM2G-rcp85**	**GFDL-ESM2M-rcp45**	**GFDL-ESM2M-rcp85**	**HadGEM2-CC-rcp45**	**HadGEM2-CC-rcp85**	**inmcm4-rcp45**	**inmcm4-rcp85**
Jan	2.89	6.25	3.82	7.82	5.32	9.43	7.46	5.72	8.15	2.28	5.02	1.16	3.63	1.61	13.84	3.84	6.64
Feb	4.82	8.03	4.30	9.82	5.73	9.08	8.91	7.54	10.97	3.10	6.95	2.97	4.85	2.85	14.30	4.36	6.58
Mar	5.87	9.91	6.79	11.62	7.74	11.00	11.80	8.53	10.17	4.15	8.19	4.89	7.98	4.49	17.10	3.77	6.88
Apr	9.25	13.74	9.89	14.89	9.39	14.13	13.19	9.25	13.73	4.12	9.54	7.24	9.41	7.22	20.37	5.77	9.46
May	7.19	11.33	10.16	15.85	7.55	12.64	9.70	7.99	10.99	-1.15	3.77	3.10	5.62	9.48	16.98	9.97	13.69
Jun	5.51	10.25	8.16	12.48	6.44	9.57	9.67	4.84	9.25	-0.58	2.08	0.76	3.84	10.47	16.18	9.40	12.82
Jul	3.62	7.35	6.39	11.25	4.30	7.79	9.03	4.84	7.48	-1.49	1.24	1.26	2.58	8.06	13.25	5.62	8.31
Aug	5.12	7.92	6.36	9.77	4.31	7.54	9.32	3.26	5.61	-1.79	1.93	0.26	3.51	7.07	10.17	5.19	7.92
Sept	3.44	6.90	6.17	10.71	4.78	7.28	9.05	1.93	5.10	-0.08	1.80	0.36	3.81	7.19	9.45	6.91	9.45
Oct	-0.99	3.00	2.35	6.94	3.51	6.70	6.66	-1.06	3.49	-1.85	2.32	-1.94	2.34	4.94	9.15	3.26	6.88
Nov	1.48	5.37	2.96	8.84	7.03	10.56	8.94	3.47	6.07	0.65	5.99	1.73	5.14	5.42	14.60	4.30	8.23
Dec	2.06	6.45	3.06	8.14	6.47	9.80	7.17	2.88	4.94	-0.32	3.64	-0.32	3.20	0.87	12.29	3.75	5.19

Table N-5b: Percentage change in evaporation at Kirstenbosch weather station between ERA-interim (1979-2012) and climate change model (2050-2100) (Continued)

Month	Percentage change in evaporation at Kirstenbosch weather station between ERA-interim (1979-2012) and climate change model (2050-2100)																	
	45	85	45	85	45	85	45	85	45	85	45	85	45	85				
	IPSL-CM5A-MR-rcp45	IPSL-CM5A-MR-rcp85	IPSL-CM5B-LR-rcp45	IPSL-CM5B-LR-rcp85	MIROC5-rcp45	MIROC5-rcp85	MIROC-ESM-CHEM-rcp45	MIROC-ESM-CHEM-rcp85	MIROC-ESM-rcp45	MIROC-ESM-rcp85	MPI-ESM-LR-rcp45	MPI-ESM-LR-rcp85	MRI-CGCM3-rcp45	MRI-CGCM3-rcp85	Minimum change (negative means a decrease)	Maximum change	Standard Deviation	Average change
Jan	5.00	12.48	12.48	2.90	5.75	8.54	2.12	6.78	1.87	7.58	2.48	7.15	3.12	5.93	1.16	13.84	3.28	5.78
Feb	6.25	14.25	14.25	6.32	6.28	8.64	2.78	6.98	1.82	7.07	4.79	9.19	4.83	7.92	1.82	14.30	3.32	6.99
Mar	8.92	15.03	15.03	9.79	6.28	10.09	4.74	10.28	4.21	10.97	6.18	9.03	6.31	9.64	3.77	17.10	3.35	8.62
Apr	10.17	17.72	17.72	12.99	8.17	11.93	8.46	13.50	7.63	13.96	5.65	11.20	9.40	12.89	4.12	20.37	3.76	11.03
May	10.24	17.61	17.61	12.39	7.46	11.87	10.44	15.89	9.63	14.67	4.29	9.55	9.44	11.45	-1.15	17.61	4.38	10.24
Jun	8.36	14.50	14.50	8.87	7.44	9.99	10.33	14.85	9.96	13.79	5.90	10.43	7.35	8.93	-0.58	16.18	4.05	8.92
Jul	6.33	11.01	11.01	7.38	2.48	5.79	5.26	10.83	6.11	10.39	6.21	9.51	5.53	8.44	-1.49	13.25	3.37	6.68
Aug	7.53	11.95	11.95	7.14	1.59	4.82	3.31	7.39	5.14	6.61	4.53	8.15	4.24	8.38	-1.79	11.95	3.18	6.01
Sept	6.45	12.72	12.72	6.15	1.89	5.31	1.54	5.94	2.59	4.93	2.53	8.76	4.05	7.95	-0.08	12.72	3.38	5.73
Oct	2.25	9.16	9.16	0.02	2.84	4.59	-0.63	4.77	0.10	3.25	0.55	7.33	-1.20	2.17	-1.94	9.16	3.34	3.23
Nov	4.45	11.37	11.37	1.17	8.11	11.01	2.52	9.43	3.71	8.37	3.17	8.87	1.86	4.74	0.65	14.60	3.59	6.16
Dec	4.56	10.42	10.42	0.79	4.10	7.94	1.88	6.46	1.45	6.17	0.45	6.17	0.06	4.55	-0.32	12.29	3.40	4.67

Table N-6a: Percentage change in rainfall at Kirstenbosch weather station between ERA-interim (1979-2012) and climate change model (2050-2100)

Month	Percentage change in rainfall at Kirstenbosch weather station between ERA-interim (1979-2012) and climate change model (2050-2100)																
	45	85	45	85	45	85	85	45	85	45	85	45	85	45	85	45	85
	bcc-csm1-1-rcp45	bcc-csm1-1-rcp85	BNU-ESM-rcp45	BNU-ESM-rcp85	CanESM2-rcp45	CanESM2-rcp85	CMCC-CESMrcp85	CNRM-CM5-rcp45	CNRM-CM5-rcp85	GFDL-ESM2G-rcp45	GFDL-ESM2G-rcp85	GFDL-ESM2M-rcp45	GFDL-ESM2M-rcp85	HadGEM2-CC-rcp45	HadGEM2-CC-rcp85	inmcm4-rcp45	inmcm4-rcp85
Jan	11.05	-5.88	-5.62	-8.20	-26.62	-43.61	-43.43	-41.54	-39.61	-53.20	-47.46	-40.50	-34.94	-12.35	-26.11	-53.11	-53.48
Feb	13.08	-14.16	26.21	18.35	4.18	-9.45	-21.42	-1.58	2.64	-21.01	-42.34	-7.11	-30.76	59.11	-12.56	-22.94	-14.71
Mar	-26.37	-43.17	-29.25	-28.85	-33.60	-34.29	-34.89	-23.41	-24.75	-44.82	-48.30	-35.51	-51.05	-13.44	-21.07	-20.46	-36.51
Apr	-10.81	-22.56	-22.99	-31.88	6.41	-14.50	4.97	-1.43	-17.14	6.16	-26.30	8.86	-23.80	20.88	12.22	-18.63	-20.45
May	-8.73	-10.70	-8.90	-20.01	-13.42	-19.02	5.30	-20.81	-13.18	6.44	6.49	-0.39	-3.15	-7.97	-11.84	-32.68	-40.57
Jun	5.73	6.98	9.98	11.77	9.10	9.81	24.76	0.40	2.60	26.26	22.50	26.51	24.54	10.32	3.83	-16.47	-19.60
Jul	12.59	9.96	14.84	18.23	16.87	6.01	24.11	14.95	17.26	22.27	27.26	21.40	25.42	-4.30	4.23	-10.98	-10.37
Aug	-2.21	-1.74	8.49	6.44	5.24	2.28	2.23	8.08	7.37	20.30	23.04	28.80	17.96	-2.24	1.84	-15.90	-13.24
Sept	9.88	10.57	-3.95	3.25	3.83	18.30	6.17	26.31	22.66	33.70	31.68	33.68	19.50	-3.88	6.37	-17.93	-21.10
Oct	68.69	64.78	34.11	48.12	52.49	51.09	48.35	79.79	55.66	78.67	60.92	83.35	68.31	38.65	52.75	21.10	3.36
Nov	20.53	13.07	21.59	-13.80	-7.74	-3.65	-4.75	20.48	12.03	-4.82	-28.46	-8.05	-18.92	-8.23	-7.88	-29.12	-29.33
Dec	25.81	-14.29	29.56	24.37	-11.96	-16.84	-29.50	-1.67	-5.97	-17.13	-25.46	9.89	-22.15	11.92	-3.07	-24.49	-17.83

Table N-6b: Percentage change in rainfall at Kirstenbosch weather station between ERA-interim (1979-2012) and climate change model (2050-2100) (Continued)

Month	Percentage change in rainfall at Kirstenbosch weather station between ERA-interim (1979-2012) and climate change model (2050-2100)																	
	45	85	45	85	45	85	45	85	45	85	45	85	45	85				
	IPSL-CM5A-MR-rcp45	IPSL-CM5A-MR-rcp85	IPSL-CM5B-LR-rcp45	IPSL-CM5B-LR-rcp85	MIROC5-rcp45	MIROC5-rcp85	MIROC-ESM-CHEM-rcp45	MIROC-ESM-CHEM-rcp85	MIROC-ESM-rcp45	MIROC-ESM-rcp85	MPI-ESM-LR-rcp45	MPI-ESM-LR-rcp85	MRI-CGCM3-rcp45	MRI-CGCM3-rcp85	Minimum change ('-' means a decrease)	Maximum change	Standard Deviation	Average change
Jan	-28.94	-47.82	25.53	12.61	-52.37	-57.03	-4.03	-12.56	-13.27	-11.44	-19.93	-48.19	-33.71	-23.84	-57.03	25.53	22.06	-27.08
Feb	-30.76	-39.12	59.11	9.83	-24.78	-7.59	58.99	31.76	47.42	4.70	12.08	-24.76	15.98	8.78	-42.34	59.11	28.35	1.52
Mar	-30.43	-46.53	-28.86	-31.68	-12.55	-41.14	-10.63	-41.49	0.03	-25.70	5.26	-17.80	-35.38	-36.55	-51.05	5.26	13.42	-29.13
Apr	-16.50	-35.93	-21.28	-22.91	-10.75	-23.68	-1.11	-13.77	10.43	-5.96	17.69	1.11	-20.21	-20.89	-35.93	20.88	15.26	-10.15
May	-24.18	-26.83	-14.49	-15.03	-17.58	-15.12	-8.85	-16.57	-6.42	-20.37	2.72	-6.37	-20.29	-20.70	-40.57	6.49	10.94	-13.01
Jun	0.68	-0.34	3.28	4.29	1.78	-4.90	-3.37	-7.99	-1.80	-4.28	10.62	9.44	1.95	-2.67	-19.60	26.51	11.37	5.34
Jul	12.08	9.86	2.52	10.66	20.38	13.93	13.50	13.76	10.54	12.74	18.62	24.31	15.97	19.28	-10.98	27.26	9.41	13.16
Aug	4.45	-5.58	-15.67	-0.46	11.94	20.09	10.99	1.00	-4.65	-1.69	8.85	2.49	2.06	-7.20	-15.90	28.80	10.72	3.98
Sept	7.57	1.43	15.36	7.15	28.37	30.62	21.13	14.65	9.48	11.90	14.60	7.53	0.49	-1.83	-21.10	33.70	13.74	11.21
Oct	65.16	35.16	105.24	95.34	58.10	59.38	74.90	70.44	59.57	65.34	67.03	31.66	59.73	59.95	3.36	105.2	20.72	58.62
Nov	12.87	-7.69	72.38	49.54	-26.76	-33.48	37.52	29.94	49.43	35.09	35.17	4.70	15.75	37.57	-33.48	72.38	27.00	7.58
Dec	23.38	-5.91	77.72	95.70	2.20	-17.25	47.88	34.59	47.91	10.21	50.95	-16.43	22.72	3.92	-29.50	95.70	31.06	9.32

Table N-7a: Percentage change in evaporation at Observatory weather station between ERA-interim (1979-2012) and climate change model (2050-2100)

Month	Percentage change in evaporation at Observatory weather station between ERA-interim (1979-2012) and climate change model (2050-2100)														
	45	85	45	85	45	85	85	45	85	45	85	45	85	45	85
	bcc-csm 1-1-rcp45	bcc-csm1-1-rcp85	BNU-ESM-rcp45	BNU-ESM-rcp85	CanESM2-rcp45	CanESM2-rcp85	CMCC-CESMrcp85	CNRM-CM5-rcp45	CNRM-CM5-rcp85	GFDL-ESM2G-rcp45	GFDL-ESM2G-rcp85	GFDL-ESM2M-rcp45	GFDL-ESM2M-rcp85	HadGEM2-CC-rcp45	HadGEM2-CC-rcp85
Jan	2.42	5.73	3.31	7.40	4.79	8.77	6.93	5.97	8.10	2.04	4.83	0.85	3.52	1.05	7.17
Feb	4.35	7.49	4.00	9.44	5.23	8.50	8.16	7.41	10.89	3.23	6.97	2.98	4.73	2.43	7.81
Mar	5.07	9.48	6.56	11.45	7.44	10.25	11.06	8.75	10.30	3.94	8.25	4.62	7.83	3.74	10.46
Apr	8.94	13.17	9.77	14.55	8.67	13.34	12.77	9.87	13.97	4.06	9.67	7.20	9.46	6.47	14.08
May	7.13	11.06	10.06	15.53	7.62	12.56	9.53	8.17	10.87	-0.89	3.91	3.43	5.73	8.58	13.10
Jun	4.42	9.15	6.97	11.01	5.37	8.64	8.17	4.00	8.26	-1.44	1.28	-0.47	2.52	8.33	11.39
Jul	3.09	7.05	6.07	10.58	3.97	7.29	9.08	4.90	6.95	-1.69	0.94	0.71	1.88	7.06	8.91
Aug	4.84	7.62	6.06	9.92	4.41	7.39	9.03	3.22	5.27	-1.62	1.60	-0.07	3.30	6.05	6.98
Sept	3.68	7.07	6.41	11.12	5.26	7.39	9.03	2.15	5.20	-0.25	2.07	1.05	3.92	6.47	7.05
Oct	-0.82	2.95	2.41	6.87	3.08	6.51	6.17	-0.83	3.21	-2.03	2.09	-1.76	2.32	4.33	6.73
Nov	1.06	4.91	2.53	8.21	6.23	9.63	8.58	3.54	6.07	0.59	5.38	1.32	5.02	5.35	9.63
Dec	1.84	6.20	2.94	7.85	6.47	9.62	6.96	3.15	5.30	0.09	3.65	-0.29	3.33	0.88	6.50

Table N-7b: Percentage change in evaporation at Observatory weather station between ERA-interim (1979-2012) and climate change model (2050-2100) (Continued)

Month	Percentage change in evaporation at Observatory weather station between ERA-interim (1979-2012) and climate change model (2050-2100)															
	45	85	45	85	45	85	45	85	45	85	45	85	45	85	45	85
	inmcm4-rcp45	inmcm4-rcp85	IPSL-CM5A-MR-rcp45	IPSL-CM5A-MR-rcp85	IPSL-CM5B-LR-rcp45	IPSL-CM5B-LR-rcp85	MIROC5-rcp45	MIROC5-rcp85	MIROC-ESM-CHEM-rcp45	MIROC-ESM-CHEM-rcp85	MIROC-ESM-rcp45	MIROC-ESM-rcp85	MPI-ESM-LR-rcp45	MPI-ESM-LR-rcp85	MRI-CGCM3-rcp45	MRI-CGCM3-rcp85
Jan	3.71	3.71	4.78	11.78	-0.50	2.34	5.83	8.39	1.91	6.42	1.61	5.33	2.44	6.95	2.96	5.73
Feb	3.97	3.97	5.92	13.86	1.49	5.63	6.18	8.25	2.50	6.63	1.59	5.93	4.56	8.81	4.31	7.21
Mar	3.17	3.17	8.35	14.83	6.34	9.47	6.04	9.64	4.77	9.79	3.86	7.66	6.13	8.59	6.02	9.34
Apr	5.22	5.22	10.00	17.14	10.42	13.12	7.95	11.70	8.16	13.60	7.55	11.29	5.80	11.01	9.15	12.63
May	9.65	9.65	10.71	17.84	8.94	12.01	7.71	11.91	10.22	15.60	9.35	15.54	4.67	9.39	9.05	10.91
Jun	8.33	8.33	7.53	13.76	4.92	7.78	6.16	8.39	9.25	13.57	8.81	13.72	4.86	9.47	5.98	7.65
Jul	5.45	5.45	6.05	10.98	4.82	6.72	2.29	5.26	5.07	10.60	5.75	10.47	6.07	9.02	5.02	7.97
Aug	5.36	5.36	7.45	12.44	5.52	6.91	1.47	4.40	3.67	7.84	5.32	9.07	4.41	7.89	3.93	7.53
Sept	6.60	6.60	6.46	12.52	3.96	6.30	1.80	5.27	2.20	6.80	3.38	7.87	2.88	8.66	4.31	7.61
Oct	3.17	3.17	2.21	8.81	-2.64	0.07	2.48	3.95	-0.14	4.73	0.52	4.59	0.95	7.19	-1.21	2.02
Nov	4.31	4.31	3.88	10.61	-0.95	0.95	7.95	10.88	2.38	8.91	3.12	6.52	3.53	8.88	2.02	4.94
Dec	3.72	3.72	4.05	9.63	-2.14	0.78	4.01	7.82	1.70	6.36	1.02	5.69	0.80	6.20	-0.04	4.57

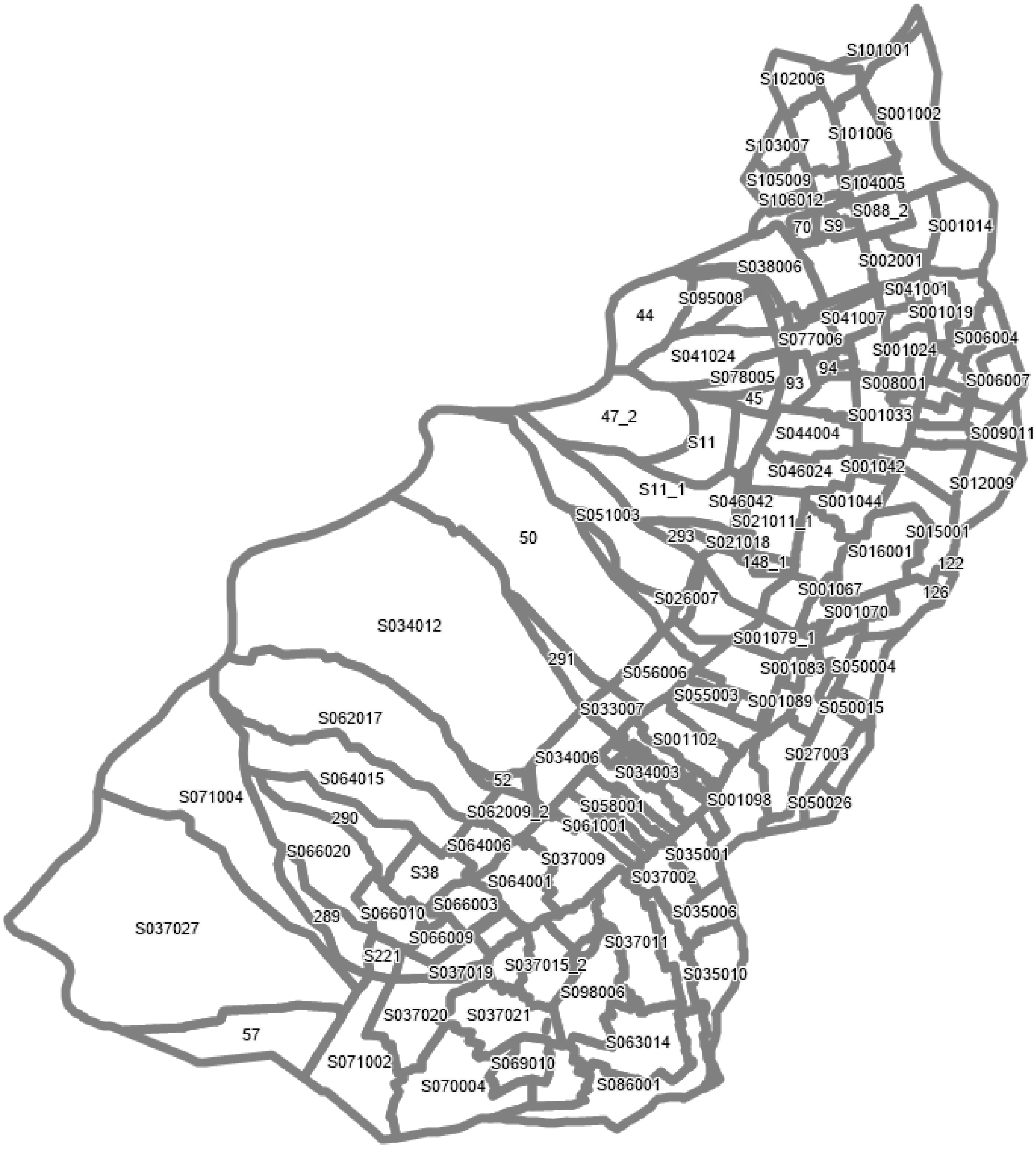
Table N-8a: Percentage change in rainfall at Observatory weather station between ERA-interim (1979-2012) and climate change model (2050-2100)

Month	Percentage change in rainfall at Observatory weather station between ERA-interim (1979-2012) and climate change model (2050-2100)														
	45	85	45	85	45	85	85	45	85	45	85	45	85	45	85
	bcc-csm1-1-rcp45	bcc-csm1-1-rcp85	BNU-ESM-rcp45	BNU-ESM-rcp85	CanESM2-rcp45	CanESM2-rcp85	CMCC-CESMrcp85	CNRM-CM5-rcp45	CNRM-CM5-rcp85	GFDL-ESM2G-rcp45	GFDL-ESM2G-rcp85	GFDL-ESM2M-rcp45	GFDL-ESM2M-rcp85	HadGEM2-CC-rcp45	HadGEM2-CC-rcp85
Jan	-2.63	-7.39	-9.34	-6.96	-25.82	-47.88	-36.74	-47.62	-38.59	-59.92	-43.36	-31.34	-37.76	-25.41	-30.45
Feb	-5.96	-38.34	1.34	3.05	-1.77	-33.20	-40.08	-24.01	-41.38	-38.60	-51.08	-40.06	-42.27	12.20	-35.72
Mar	-24.85	-45.09	-18.87	-22.20	-38.57	-35.94	-42.98	-33.02	-31.22	-47.91	-54.42	-47.16	-49.39	-5.54	-18.52
Apr	-29.07	-34.85	-36.33	-41.99	-19.45	-35.53	-8.97	-14.96	-28.97	-8.84	-38.13	-6.44	-37.96	-7.54	0.18
May	-12.10	-13.31	-4.46	-15.80	-10.66	-20.11	0.45	-19.09	-11.86	5.76	5.57	1.01	2.08	-15.35	-11.78
Jun	-4.10	1.10	-3.03	-4.26	0.48	1.93	11.21	-0.97	0.18	16.57	16.50	16.94	8.78	-4.67	-3.43
Jul	12.93	8.76	9.81	14.15	18.79	6.91	26.30	14.36	20.90	19.16	20.00	18.21	23.81	-5.31	6.21
Aug	-9.26	0.22	5.20	4.53	6.08	0.86	-0.17	4.08	0.70	12.97	21.54	27.62	11.22	-9.35	5.11
Sept	10.17	2.37	-5.46	1.07	4.24	16.07	-1.18	21.58	16.46	24.22	24.41	26.62	15.61	-6.06	12.71
Oct	56.79	44.32	27.29	45.75	36.12	33.71	37.19	51.13	36.47	52.54	31.75	64.78	38.78	27.67	45.70
Nov	-1.30	-6.63	3.16	-24.18	-32.44	-23.27	-19.65	-3.13	-3.41	-13.88	-41.93	-13.33	-31.40	-19.76	-22.08
Dec	14.00	-5.22	25.11	13.45	-20.98	-15.47	-29.17	-11.40	2.28	-15.29	-21.19	9.98	-21.13	9.48	-2.39

Table N-8b: Percentage change in rainfall at Observatory weather station between ERA-interim (1979-2012) and climate change model (2050-2100) (Continued)

Month	Percentage change in rainfall at Observatory weather station between ERA-interim (1979-2012) and climate change model (2050-2100)															
	45	85	45	85	45	85	45	85	45	85	45	85	45	85	45	85
	inmcm4-rcp45	inmcm4-rcp85	IPSL-CM5A-MR-rcp45	IPSL-CM5A-MR-rcp85	IPSL-CM5B-LR-rcp45	IPSL-CM5B-LR-rcp85	MIROC5-rcp45	MIROC5-rcp85	MIROC-ESM-CHEM-rcp45	MIROC-ESM-CHEM-rcp85	MIROC-ESM-rcp45	MIROC-ESM-rcp85	MPI-ESM-LR-rcp45	MPI-ESM-LR-rcp85	MRI-CGCM3-rcp45	MRI-CGCM3-rcp85
Jan	-65.63	-63.60	-26.81	-47.67	13.58	1.87	-57.38	-59.67	-8.50	-10.58	-11.69	-17.00	-21.91	-50.48	-26.88	-24.11
Feb	-45.67	-32.16	-28.33	-50.77	19.85	-7.09	-40.43	-31.28	21.17	-3.46	4.25	-25.81	-20.79	-48.80	-16.67	-12.98
Mar	-37.81	-39.26	-30.70	-48.36	-32.71	-31.21	-23.24	-41.15	-0.72	-39.16	17.04	-14.09	9.53	-11.49	-35.59	-26.05
Apr	-27.35	-30.15	-32.18	-45.58	-35.64	-32.50	-26.50	-31.09	-18.19	-19.85	3.34	-18.32	6.16	-11.50	-25.85	-26.95
May	-33.76	-33.75	-19.66	-24.86	-14.22	-17.51	-15.09	-13.56	-12.52	-16.34	-9.72	-25.58	3.38	-9.54	-22.49	-22.31
Jun	-13.76	-15.65	-1.09	-0.46	-1.23	-6.67	-5.51	-11.67	-5.00	-16.03	-7.20	-14.18	1.72	5.06	-4.89	-6.47
Jul	-8.61	-5.00	16.94	16.88	2.68	5.25	17.96	11.89	18.98	19.41	13.52	19.09	21.78	25.78	19.41	20.26
Aug	-11.99	-9.91	5.07	-3.57	-13.18	-3.16	8.00	14.30	2.88	2.25	-7.25	-0.70	2.52	-0.09	-1.83	-15.14
Sept	-18.04	-17.77	6.97	0.49	13.06	5.61	22.05	24.71	23.26	16.65	5.90	15.01	15.54	5.46	-2.30	-6.04
Oct	11.54	4.09	53.13	24.71	87.74	65.59	40.93	36.36	53.88	52.97	54.89	41.71	54.78	24.82	41.57	46.00
Nov	-45.79	-40.85	-10.59	-27.00	26.99	24.50	-45.63	-46.59	7.93	8.40	18.12	3.75	16.83	-9.39	-2.56	14.16
Dec	-27.60	-14.38	0.56	-18.28	50.00	87.54	-11.12	-33.61	45.16	36.02	50.20	4.44	32.47	-19.50	21.08	-4.31

Appendix O: Modelling subcatchments



Appendix P: Copyrights and disclaimers for figures

A number of the figures contained data which was either obtained from more than one source, or was compiled using original data and data obtained from an external source. Section R1 of this Appendix provides the relevant copyright, disclaimer and background information for the relevant sources. Section R2 provides a list of the relevant figures and what data was used from each source.

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'This map presents low-resolution imagery for the world and high-resolution imagery for the United States and other areas around the world. The map includes NASA Blue Marble: Next Generation 500m resolution imagery at small scales (above 1:1,000,000), i-cubed 15m eSAT imagery at medium-to-large scales (down to 1:70,000) for the world, and USGS 15m Landsat imagery for Antarctica. The map also includes i-cubed Nationwide Prime 1m or better resolution imagery for the contiguous United States, Getmapping 1m imagery for Great Britain, and GeoEye IKONOS 1m resolution imagery for Hawaii, parts of Alaska, and several hundred metropolitan areas around the world. I-cubed Nationwide Prime is a seamless, color mosaic of various commercial and government imagery sources, including Aerials Express 0.3 to 0.6m resolution imagery for metropolitan areas and the best available United States Department of Agriculture (USDA) National Agriculture Imagery Program (NAIP) imagery

and enhanced versions of United States Geological Survey (USGS) Digital Ortho Quarter Quad (DOQQ) imagery for other areas. For more information on this map, visit us online at http://goto.arcgisonline.com/maps/World_Imagery'

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Q2 Data sources for figures

Table R1 provides the data sources for figures where a reference has not been provided in text due to the raw data having been edited and/or more than one data source being used in a single image. Table R1 explains what data originates from each source.

Table R1: Data sources for figures where reference has not been provided

Figure	Source	Explanation
Figure 3-1	CoCT (2009d) StatsSA (2011) Esri et al., 2015)	Shapefile showing suburb boundary Shapefile showing city boundary Basemap
Figure 3-5	CoCT (2009a) Esri *et al.* (2015)	Shapefiles showing landuse, and road Basemap
Figure 3-6	CoCT (2009a) Stats SA. (2011) Esri *et al.* (2015)	Shapefiles showing landuse, and road Smal area layer and associated occupation data Basemap
Figure 4-1	CoCT (2009b)	Basemap
Figure 4-2	Esri *et al.* (2015)	Basemap
Figure 4-4	Esri *et al.* (2015)	Basemap
Figure 4-6	Esri *et al.* (2015)	Basemap
Figure 4-16	Esri *et al.* (2015) CoCT (2009e)	Basemap Shapefile of monitoring points
Figure 4-17	Esri *et al.* (2015)	Basemap
Figure 4-19	Esri *et al.* (2015)	Basemap
Figure 4-26	Esri *et al.* (2015)	Basemap
Figure 5-25	Esri *et al.* (2015)	Basemap
Figure 5-38	Esri *et al.* (2015)	Basemap